COLD IN-PLACE RECYCLING (CIR) STUDY

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U.S. Department of Transportation
Federal Highway Administration

CTIP

Federal Lands Highway
Commitment to Excellence

Central Federal Lands Highway Division
12300 West Dakota Avenue
Lakewood, CO 80228

FOREWORD

The Federal Lands Highway (FLH) promotes development and deployment of applied research and technology applicable to solving transportation related issues on Federal Lands. The FLH provides technology delivery, innovative solutions, recommended best practices, and related information and knowledge sharing to Federal agencies, Tribal governments, and other offices within the FHWA.

The objective of this study was to provide information to assist FLH with determination of sampling locations, sample curing protocols, mix design processes, reasonable target values for acceptance (QC/QA) testing, structural layer coefficients and quantify typical construction variability for cold in-place recycled (CIR) mixtures.

The study included a literature search on CIR and field sampling and testing on two CIR projects. Recommendations for improving FLH's mix design procedures and construction specifications were made.

The contributions and cooperation of the CFLHD personnel is gratefully acknowledged.

Notice

Quality Assurance Statement

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16. Abstract

Federal Lands Highway (FLH) utilizes cold in-place recycling (CIR) as a pavement rehabilitation technique and has a documented history of successful projects. FLH, with their policy of continual improvement, has expressed an interest completing a study evaluating current construction quality control and acceptance methods on CIR projects with the goal of improving these procedures. Current construction quality control and quality assurance on CIR projects is generally limited to checking for density, establishing a roller pattern, proof rolling, visual inspection, and/or checking for yields of emulsions and additives.

The objectives of this project were to provide information to assist FLH with determination of sampling locations, sample curing protocols, mix design processes, reasonable target values for acceptance (QC/QA) testing, structural layer coefficients and quantify typical construction variability by sampling and testing two CIR projects, CA PFH 123-1(1), Washington Road, and CA PFH 119-1(3), Quincy - Oroville Road.

Mix design methods using asphalt emulsions were recommended for CIR and full depth reclamation (FDR) mixes. Recommendations to current FLH CIR construction specifications were made and a new procedure for control of compacted CIR density was developed. The two projects evaluated in this study were not uniform and this lack of uniformity contributed to high standard deviations of CIR mix properties. Field mix sample age and temperature had a significant effect on mix properties. Compaction delay and mix temperature of CIR lab mixtures must be specified if field mix properties are used for construction control. An AASHTO structural layer coefficient of 0.32 – 0.34 appears reasonable based on dynamic modulus test results.

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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	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	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

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ACRONYMS & ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
A/E	architect/engineer
ARRA	Asphalt Recycling and Reclaiming Association
ASTM	American Society for Testing and Materials
BARM	Basic Asphalt Recycling Manual
CFLHD	Central Federal Lands Highway Division
CIR	cold in-place recycling
CSS	cationic slow set
DOT	Departments of Transportation
E*	dynamic modulus
FDR	full depth reclamation
FHWA	Federal Highway Administration
FLH	Federal Lands Highways
Gmm	maximum specific gravity
GSB	granular stabilized base
Gsb	bulk specific gravity
Gse	effective specific gravity
HMA	hot mix asphalt
ITS	indirect tensile strength
NCAT	National Center for Asphalt Technology
NCHRP	National Cooperative Highway Research Program
OMC	optimum moisture content
OSU	Oklahoma State University
PCC	portland cement concrete
PG	performance grade
PI	principal investigator
QC/QA	quality control / quality assurance
QO	Quincy-Oroville Road
RAP	reclaimed asphalt pavement
SE	sand equivalent
SGC	Superpave Gyratory Compactor
SS	slow set
SSD	saturated surface dry
TRB	Transportation Research Board
VTM	voids total mix
WR	Washington Road

EXECUTIVE SUMMARY

BACKGROUND

Federal Lands Highway (FLH) utilizes cold in-place recycling (CIR) as a pavement rehabilitation technique and has a documented history of successful projects. FLH, with their policy of continual improvement, has expressed an interest completing a study evaluating current construction quality control and acceptance methods on CIR projects with the goal of improving these procedures. Current construction quality control and quality assurance on CIR projects is generally limited to checking for density, establishing a roller pattern, proof rolling, visual inspection, and/or checking for yields of emulsions and additives. Achieving adequate density is a very important component for a quality construction project, but standardized methods for checking field density of CIR do not exist. As a result agencies have adopted and modified HMA-type procedures for density evaluation including the use of nuclear gauges, roller patterns, etc. There is a need for a study to evaluate options for developing a more robust and credible approach to evaluating density.

Another issue for CIR construction quality control has been the emergence and development of various proprietary asphalt emulsion products. The suppliers of these products tout the added benefits of increased durability, reduced opening to traffic time, stronger pavement layers, and other such performance increases. These proprietary asphalt emulsion products tend to cost more than conventional asphalt emulsions. In order to justify the additional costs, validate performance claims, and fairly advertise and award projects, agencies need methods to measure and assure performance characteristics during construction.

Finally, the increasing price of materials and shrinking transportation budgets has management focused on providing more optimal pavement designs. Higher structural coefficients for CIR layers may be established if greater consistency, stiffness, and durability of the CIR layer can be validated during construction. Establishing higher structural coefficients translates into thinner overlay requirements, which potentially provides for more cost effective designs.

OBJECTIVES

The objectives of this project were to provide information to assist FLH determine sampling locations, sample curing protocol, mix design processes, reasonable target values for acceptance (QC/QA), structural layer coefficients and quantify typical construction variability. In order to meet the objectives, a work plan was implemented that evaluated a sampling and testing protocol on two CIR projects, CA PFH 123-1(1), Washington Road, and project number CA PFH 119-1(3), Quincy - Oroville Road.

WORK PLAN

A work plan was developed that consisted of four parts with work tasks under each part. The following tasks were performed to meet the objectives of this project:

- Part A. Monitoring and Meetings
- Part B. Materials Testing
    - Task 1: Perform CIR Mix Designs
    - Task 2: Completion of Field QA/QC Testing
        - AASHTO T 283 (modified);
        - Retained Marshall stability, AASHTO T 245;
        - Bulk specific gravity, AASHTO T 166 and AASHTO T 331;
        - Gradation analysis, AASHTO T 27 & AASHTO T 30;
        - Maximum specific gravity, AASHTO T 209 and ASTM D6857;
        - Asphalt content, AASHTO T 308;
        - Dynamic modulus, AASHTO TP 62 and AASHTO TP 79.
    - Task 3: Compilation of FHWA provided data
- Part C. Literature Review
- Part D. Analysis and Report
    - Task 1: Analyze Data and Provide Recommendations for:
        - Construction quality control testing and acceptance processes/specifications,
        - Use of pay factors and/or statistical acceptance procedures,
        - Structural value to assign CIR layers (structural coefficient, E*, etc.),
        - Develop recommended mix design procedures for CIR and full depth reclamation (FDR) mixtures using asphalt emulsions.
    - Task 2: Submit Draft Report
    - Task 3: Submit Final Report
    - Task 4: Submit three PowerPoint Presentations
        - Summarizing the study,
        - Summarize the two mix design procedures.

REPORT ORGANIZATION

Part B Materials Testing

Summary results for the mix designs performed for Washington Road and Quincy-Oroville Road are found in Chapter 2. Test data from field and laboratory testing on the above two projects are found in Appendix A. Summarized test data is contained in Chapter 4 and summary statistics and analysis of project data are found in Chapters 5 and 6, respectively.

Part C Literature Review

Summary results for the literature review are found in Chapter 3. The complete literature is in Appendix D.

Part D Analysis and Report

Recommendations for construction quality control testing procedures are found in Chapters 5 and 6. Recommended construction specifications are found in Chapter 8 along with suggestions on the use of pay factors. Recommended AASHTO layer coefficients are discussed in Chapter 6. Mix design procedures are discussed in Chapter 7 and draft mix design methods for CIR and FDR are found in Appendix B and C, respectively. PowerPoint presentations were submitted under separate cover.

CONCLUSIONS

Major conclusions of this study are:

- Although still affected by water absorption, ASTM D6857 provided more reasonable maximum specific gravity results than AASHTO T 209 without the dry-back procedure.
- The existing pavements prior to in-place recycling were not uniform as shown by extracted aggregate gradations, back-calculated effective specific gravity values and extracted asphalt contents.
- The lack of uniformity of the existing pavements, contractor operations and construction temperatures contributed to the higher standard deviation of CIR mix properties compared to HMA values found in the literature. Standard deviations were 2-3 times higher for CIR compared to reported HMA values.
- The majority of 150 mm diameter molded CIR samples exceeded the threshold value of 2 percent water absorption of AASHTO T 166 and would require AASHTO T 331 or equivalent testing. A statistically significant difference was found between AASHTO T 166 and AASHTO T 331 bulk specific gravity values.
- The higher air voids calculated using AASHTO T 331 bulk specific gravities resulted in difficultly vacuum saturating samples to the 70-80 percent saturation level required in AASHTO T 283 as many of the AASHTO T 331 measured void spaces could not hold water for the SSD mass required for percent saturation calculations.
- Age and temperature of field produced mix had a significant effect on lab molded bulk specific gravity and many mix properties, excluding tensile strength ratio (TSR). If field produced mix properties are used for control of CIR mixtures then compaction delay and mix temperature must be specified.
- The FLH AASHTO structural layer coefficient for CIR of 0.28 appears conservative. A value of 0.32 – 0.34 appears reasonable based on dynamic modulus values at 68^{o}F and 0.1 – 1.0 Hz.
- Based on the limited dynamic modulus testing performed and data from the literature, the preliminary dynamic modulus values shown in Table 26 can be used for design until more data is available.
- Using percent relative compaction based on a target value of the maximum obtainable density from a control strip appears to adequately ensure average compaction to within 97-103 percent of the maximum obtainable density of the material.
- Additional test data, including cores from existing projects, would be required before recommendations could be made on the use of pay factors and statistical based specifications for control of CIR mixtures.

RECOMMENDATIONS

Based on the data obtained and the limits of the materials and test methods evaluated in this study, the following recommendations are warranted.

- Use AASHTO T 209 with the dry-back procedure or ASTM D6857 for determination of maximum specific gravity of CIR mixtures.
- Use of AASHTO T 166 will require the use of AASHTO T 331 in the majority of cases. When determining percent vacuum saturation for modified AASHTO T 283 testing use 55-75 percent saturation and AASHTO T 166 Method A and AASHTO T 331 when required, for determination of bulk specific gravity.
- For control of mix properties, compact field samples at in-situ temperatures within ± 30 minutes of the compaction delay between mixing and compaction used in the field. Keep field samples sealed and protected from excessive heat or cold prior to compaction.
- Use an AASHTO structural layer coefficient of 0.32-0.34 for structural design of CIR mixtures.
- Use the proposed CIR mix design method in Appendix B for CIR mixtures.
- Use the proposed FDR mix design method in Appendix C for FDR mixtures.
- Implement the specification changes and proposed compaction procedure outlined in Chapter 8.

CHAPTER 1 – STATEMENT OF WORK

BACKGROUND

Federal Lands Highway Division (FLH) utilizes cold in-place recycling (CIR) as a pavement rehabilitation technique and has a documented history of successful projects. FLH, with their policy of continual improvement, has expressed an interest completing a study evaluating current construction quality control and acceptance methods on CIR projects with the goal of improving these procedures. Current construction quality control and quality assurance on CIR projects is generally limited to checking for density, establishing a roller pattern, proof rolling, visual inspection, and/or checking for yields of emulsions. Achieving adequate density is a very important component for a quality construction project, but standardized methods for checking field density of CIR do not exist. As a result agencies have adopted and modified HMA-type procedures for density evaluation including the use of nuclear gauges, roller patterns, etc. There is a need for a study to evaluate options for developing a more robust and credible approach to evaluating density.

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Finally, the increasing price of materials and shrinking transportation budgets has management focused on providing more optimal pavement designs. Higher structural coefficients for CIR layers may be established if greater consistency, stiffness, and durability of the CIR layer can be validated during construction. Establishing higher structural coefficients translates into thinner overlay requirements, which potentially provides for more cost effective designs.

SCOPE

This project will pilot a sampling and testing protocol on two CIR projects, CA PFH 123-1(1), Washington Road, and project number CA PFH 119-1(3), Quincy - Oroville Road. State of the practice and state of the art test methods and equipment will be evaluated, including the use of the nuclear gauge, tensile strength ratio (TSR) test, falling weight deflectometer (FWD) data, dynamic modulus, and use of gyratory compactor test equipment.

OBJECTIVES

The objectives of this project are to provide information to assist FLH determine sampling locations, sample curing protocol, mix design processes, reasonable target values for acceptance (QC/QA), structural layer coefficients and quantify typical construction variability.

WORK TASKS

The above objectives will be met by performing the following tasks:

Part A. Monitoring and Meetings

Task 1. Kick-Off Teleconference

Within 45 days of award of the task order, the A/E in coordination with the COTR shall schedule a "kick-off" teleconference with the advisory panel to discuss the project schedule, review the project's objectives, answer questions, and ensure a complete understanding of the intent and administration of the project.

Task 2.Teleconference and Meetings

Periodic teleconferences to discuss project status and issues will be scheduled by the COTR throughout the duration of the project. One face-to-face meeting with the A/E at the CFLHD office and two project site visits by the A/E are expected.

Part B. Materials Testing

Task 1. Mix Designs

Completion of CIR mix design on up to two projects.

Task 2. Field QA/QC Testing

Completion of field QA/QC testing, including but not limited to the following tests: resistance to moisture induced damage, AASHTO T 283 (modified); retained Marshall stability, AASHTO T 245, bulk specific gravity, AASHTO T 166 and AASHTO T 331; gradation analysis, AASHTO T 27 & AASHTO T 30; maximum specific gravity, AASHTO T 209 and ASTM D6857; asphalt content, AASHTO T 308; raveling test, ASTM D7196; and dynamic modulus, AASHTO TP 62 and AASHTO TP 79.

Task 3. Compilation of Data

Compilation of FHWA provided data including construction quality control and acceptance test results.

Part C. Literature Review

Task 1. Review of Literature

Review research reports and case studies that have been completed and are related to the objectives of this study.

Task 2. Summary Report

Provide a summary of the documents reviewed and evaluate the literature for applicability, objectivity, and quality of content.

Part D. Analysis and Report

Task 1. Data Analysis

Analyze the data collected in Part B above. Evaluate trends and variability of the data. Provide recommendations for construction quality control testing and acceptance processes/specifications that will assure quality construction and long-term performance of CIR projects. Include recommendation of the applicability of using pay factors and/or statistical acceptance procedures. Provide recommendations for the structural value to assign CIR layers (structural coefficient, E*, etc.).

Analyze the data collected in Part B above along with the information obtained in Part C and develop recommended mix design procedures for CIR and full depth reclamation (FDR) mixtures using asphalt emulsions.

Task 2. Draft Report

Submit a draft report electronically which documents the purpose, scope, methodology, findings and recommendations of the study. The draft report will be reviewed by a Federal Lands Highway (FLH) advisory panel within 30 days of receipt. Reports shall be formatted according to the FHWA Turner-Fairbank Highway Research Center Communications Reference Guide available from the CFLHD's Technology Development website.

Task 3. Final Report

Address comments from the FLH advisory panel from task 2 above and submit a final report within 30 days of receipt of comments. The final report should be submitted in both MS Word and acrobat adobe (latest versions).

Task 4. PowerPoint Presentations

Submit a draft electronic copy of a PowerPoint presentation that summarizes the study in a 30 to 45 minute presentation. In addition, provide copies of PowerPoint presentations that summarize the two mix design procedures developed in task 2 of Part D above. The FLH advisory panel will review the drafts and provide comments within 2 weeks of receipt. The A/E will address the comments and provide a final copy of the PowerPoint Presentation within 2 weeks of receipt of comments.

CHAPTER 2 - MIX DESIGNS

MIX DESIGN PROCEDURES

As a part of this study, mix designs were completed on materials obtained from project number CA PFH 123-1(1), Washington Road, and project number CA PFH 119-1(3), Quincy - Oroville Road. Mix designs were performed in general accordance with the CIR mix design methods and procedures specified by the Kansas Department of Transportation and the requirements of CFLHD. The test procedures followed are shown in Table 1.

Table 1. Mix Design Test Procedures.

Test Procedures
Superpave gyratory compaction, AASHTO T 312, 35 gyrations, 100 mm mold. 75-blow Marshall compaction, AASHTO T 245.
Sample curing, 60°C to constant mass for no less than 16 hours and no more than 48 hours.
Bulk Specific Gravity, AASHTO T 166.
Maximum Theoretical Specific Gravity, AASHTO T 209 or ASTM D6857.
Marshall stability, cured specimen, AASHTO T 245, 40°C.
Retained Marshall stability, AASHTO T 245, 40°C, %. Based on moisture conditioning on cured specimen, vacuum saturation of 55 to 75 %, water bath at 25°C for 23 hours then 1 hour in 40°C water bath.
Moisture Sensitivity, AASHTO T 283, compaction, 35 gyrations, 100 mm mold. Based on moisture conditioning on cured specimen, vacuum saturation of 70 to 80 %, water bath at 25°C for 24 hours.

MIX DESIGN RESULTS

For project number CA PFH 123-1(1), Washington Road, the original mix design was completed on samples of pavement and aggregate base provided by CFLHD that were obtained from three stations or test pits from the above referenced project. Mix designs for test pits 1 and 2 contained 25% aggregate base mixed with 75% RAP. According to ARRA[(1)], this would be considered a full depth reclamation (FDR) project due to the inclusion of aggregate base. Test pit 3 used 100% RAP. Two different emulsions were used. The contractor requested a third emulsion, Pass R, and a revised mix design was performed using RAP only as aggregate base was depleted. The mix design for project number CA PFH 119-1(3), Quincy - Oroville Road used 100% RAP from cores obtained from the project.

RAP or RAP plus aggregate gradations used for the mix designs are shown in Table 2 along with the suggested CFLHD CIR gradation. Summary results at optimum emulsion content from the mix designs using Marshall stability from Washington Road are shown in Table 3. Results from the mix designs using a modified AASHTO T 283 are shown in Table 4. The results from Table 4 were used to construct the two projects.

Table 2. Mix Design Gradations.

	Washington Road				Quincy	Suggested CFLHD CIR Gradation
	TP 1	TP 2	TP 3	Comb.	Oroville	
% RAP	75	75	100	100	100	
% Agg.	25	25	0	0	0	
Sieve Size	Percent Passing					
1.5"	100	100	100	100	100	100
1"	95	99	98	95	100	90-100
3/4"	85	90	87	88	88	85-95
1/2"	75	79	77	78	78	75-85
3/8"	55	59	57	66	66	
No. 4	41	39	40	42	42	35-50
No. 8	20	23	15	25	24	
No. 16	13	15	8.8	12	10	5-15
No. 30	7.8	8.6	4.0	6.3	4.2	
No. 50	4.6	4.8	1.8	2.4	2	
No. 100	2.6	2.8	0.7	1.1	1.1	
No. 200	1.5	2.1	0.2	0.3	0.6	0-7.0

Table 3. Mix Design Results, Washington Road.

Route	Sample	Location	EAC	EAC (%)	Lime Slurry (%)	Mix Water (%)	Specific Gravity Max.	Specific Gravity Bulk	VTM (%)	Dry Stability (lbs)	Vac. Sat. (%)	Cond. Stability (lbs)	Retained Stability Ratio
WR	TP 1	Sta 85+00	Reflex	2.0	0.0	4.5	2.467	2.062	16.4	1605	71.9	1145	0.71
WR	TP 1	Sta 85+00	HFMS-2P	2.0	3.3	1.2	2.406	2.091	13.1	1276	78.5	1020	0.80
WR	TP 2	Sta 130+00	Reflex	2.0	0.0	4.5	2.421	2.039	15.8	1713	79.4	1245	0.73
WR	TP 2	Sta 130+00	HFMS-2P	2.0	3.3	1.2	2.410	2.075	13.9	1421	70.6	995	0.70
WR	TP 3	Sta 265+00	Reflex	2.0	0.0	3.0	2.405	2.013	16.3	1264	76.3	1250	0.99
WR	TP 3	Sta 265+00	HFMS-2P	1.5	3.3	1.7	2.417	2.035	15.8	1288	79.6	1146	0.89
WR	SGC		Pass R	2.0	0.0	3.0	2.394	2.032	15.1	1893	75.0	1363	0.72
WR	Marshall		Pass R	2.0	0.0	3.0	2.394	2.043	14.7	2200	72.5	1518	0.69

WR = Washington Road

Table 4. Mix Design Results Using Modified AASHTO T 283.

Route	EAC	Compaction	Optimum EAC (%)	Lime Slurry (%)	Mix Water (%)	Specific Gravity Max.	Specific Gravity Bulk	VTM (%)	Dry Tensile (psi)	Vac. Sat. (%)	Cond. Tensile (psi)	TSR
WR	Pass R	SGC	2.0	0.0	3.0	2.394	2.012	16.0	62.8	71.4	55.3	0.77
WR	Pass R	Marshall	2.0	0.0	3.0	2.394	2.040	14.8	75.7	77.4	53.1	0.69
QO	Albina	SGC	2.25	0.0	3.3	2.426	1.956	19.4	73.5	68.4	59.6	0.81

WR = Washington Road QO = Quincy - Oroville Road

CHAPTER 3 – SUMMARY OF LITERATURE

DEFINITIONS

Cold In-Place Recycling (CIR) is defined as an asphalt pavement rehabilitation technique that reuses existing pavement materials. CIR entails the processing and treatment, with bituminous and/or chemical additives, of an existing asphalt pavement without heat to produce a restored pavement layer. All work is completed on site and the transportation of materials, except for the additive being used, is not normally required. The depth of processing is typically 3 to 4 inches. The process is sometimes referred to as partial depth recycling because the base and or some of the bituminous materials are left intact. Cold in-place recycling that incorporates untreated base material with the bound material is referred to as full depth reclamation (FDR). The above definitions will be used herein; however, not everyone appears to follow these definitions.

ADDITIVES

Numerous recycling additives have been successfully utilized in CIR. Traditional additives include asphalt emulsions, rejuvenators and pozzolonic materials such as cement, lime and fly ash or blends of these materials. The most common recycling agents are anionic high float emulsions and cationic medium and slow setting emulsions. In recent years polymer modified high float emulsions appear to be used more frequently than traditional high float emulsions and slow set cationic emulsions are more prevalent that medium set.

Hydrated lime, added as slurry or more commonly as slaked quicklime, is often used to improve moisture sensitivity and increase early strength gain of CIR mixtures. Engineered emulsions are becoming popular as a recycling agent. Engineered emulsions are typically modified solventless emulsions that are formulated to break due to chemistry, providing enhanced coating and early strength gain. Expanded asphalt or foamed asphalt has been successfully used in CIR on a limited basis. Foam is generally said to require more fines, minus No. 200 material, to promote bonding or cohesion of the recycled material than is typically present in millings. However, several states have reported preliminary results indicating the procedure could be applicable to CIR. Little reference in the current literature was found indicating that uncoated aggregates were being added to the CIR process to improve aggregate gradation. The NYSDOT was the only agency found that routinely adds uncoated aggregates to CIR to improve the recycled mix gradation.

CONSTRUCTION

Project selection was commonly mentioned as a key factor in CIR success. Most agencies limit CIR to the rehabilitation of functional failures, not structural failures, as CIR will not address subbase and subgrade issues. Pavements with soft subgrades are usually avoided and drainage issues should always be addressed to ensure project success. Many agencies limit CIR to pavements with low to moderate traffic without heavy truck traffic. Most agencies with extensive experience with CIR do not have traffic restrictions on CIR with all routes being eligible for consideration. A NYSDOT study indicated base condition/ thickness was more important to CIR performance than traffic.

Proper construction and experienced contractors were mentioned as another key element in CIR project success. Many agencies require an experienced contractor representative be on site to adjust recycling additive contents as changing conditions dictate. Adequate compaction is a key factor to performance and most agencies require both heavy pneumatic rollers and heavy double drum vibratory rollers. Traffic is generally allowed on the compacted CIR mixture after a short cure time. The placement of the wearing surface is delayed for a minimum time, usually 10 days to two weeks and/or until the CIR mixture adequately cures. Adequate curing is usually defined as less than 1.5 - 2.0% moisture or less than 1.5% above residual moisture. In most cases, the contractor is responsible for repairs to the CIR mixture until placement of the wearing surface. Wearing surfaces for CIR mixtures range from single and double chip seals for low volume pavements to HMA mixtures for higher volume pavements.

Wearing surface thickness is designed based on structural requirements to handle anticipated traffic. AASHTO structural layer coefficients have been determined from laboratory compacted specimens and field cores of CIR mixtures. The range of calculated and measured structural numbers vary from 0.20 - 0.49. Most agencies reported using structural numbers for CIR of 0.26 to 0.35 with 0.30 being more common.

MIX DESIGN

Lack of a national mix design procedure is often cited as a barrier to CIR usage. However, some of the agencies that have very successful CIR programs do not require mix designs but use typical emulsion contents based on experience and adjust as field conditions dictate. Numerous mix design procedures exist and they all have similar elements. Most modern mix design procedures determine optimum recycling additive content based on both wet and dry Marshall stability testing or wet and dry indirect tensile strengths of samples compacted from crushed cores obtained from the project site. Minimum dry strengths and minimum wet to dry strength ratios are used to evaluate recycling contents and whether lime or a similar additive is required. Some of the newer mix design methods evaluate resistance to raveling and thermal cracking.

PERFORMANCE

Performance of CIR pavements has been reported as good. Most CIR pavement failures have been attributed to poor project selection with wet pavements and or subgrades being the most frequently reported cause of CIR failures. Agencies reported better performance on projects where the majority of existing pavement cracking can be removed by the CIR process than with CIR over thick HMA pavements or concrete pavements. However, these agencies typically reported better performance of CIR than conventional HMA overlays or mill and fill procedures in these instances.

Documented treatment lives of CIR pavements of 10-15 years are common. A few agencies have documented satisfactory pavement performance of 20-25 years. Agencies that have tried to predict pavement life indicate predicted lives of 20-30 years. Preventative maintenance activities are included in these pavement life reports and predictions. A few agencies have performed life-cycle cost analysis of rehabilitation procedures. CIR is reported to have an equal or lower net present worth than conventional HMA overlays or mill and fill procedures.

SURVEY OF PRACTICE

As part of a study on CIR performance for the NYSDOT, a survey of agency practice was undertaken. The survey (Task 2) has not been published by the NYSDOT. Thirteen agencies participated in the survey. Significant findings from the survey are presented below.

Specifications

Six agencies had specifications for CIR in their standard specifications with the other seven using special provisions or supplemental specifications. All but two agencies indicated a requirement for metering of all liquid additives. All 13 agencies indicated a maximum RAP size with one agency allowing greater than 2 inches, three requiring less than 1.5 inches, eight requiring less than 1.25 inches and one requiring less than 1 inch. Three agencies indicated they required pugmill mixing of RAP and additives, all agencies require the mix be homogenous.

Weather Requirements

All agency specifications reviewed had weather restrictions for CIR. All agencies, with the exception of Arizona, restricted CIR when the weather was rainy or foggy. Arizona restricted CIR when, in the opinion of the engineer, existing or predicted weather conditions could adversely affect operations. Seven agencies restricted CIR if freezing or cold (< 35^{o}F) weather was anticipated either overnight or within 48 hours. Seven of 13 agencies specified a minimum air temperature only. Two agencies specified a minimum pavement or material temperature only and four agencies specified both. Minimum air or ambient temperatures ranged from 50 to 65^{o}F. Minimum pavement or mixture temperatures varied from 40^{o}F to 70^{o}F. Five agencies also specified calendar restrictions for CIR.

Equipment Requirements

Equipment requirements varied but most agencies specified equipment requirements but not specific equipment. Most agencies require a closed loop system consisting of a crusher and scalper screen to control RAP size and a continuous weighing system with positive displacement pumps and automatic interlock system that shuts off pumps when the process stops or no RAP is present. All agencies required the RAP be mixed to a homogenous mixture with uniform coating. Nine agency specifications required a binder tolerance, seven required ± 0.2% and one each required ± 0.1% and ± 0.3%.

All agencies required the use of pneumatic rollers and double drum vibratory steel wheel rollers. A minimum tonnage was usually specified. For pneumatic rollers, four agencies required greater than 25 tons, five required a minimum of 25 tons, two required a minimum of 20 tons and one required less than 20 tons. All agencies required double drum vibratory steel wheel rollers and eight agencies required a minimum tonnage. Five agencies required a minimum 10 ton roller, two required a minimum 12 ton roller and one required a minimum 9 ton roller.

Field Construction Monitoring

All agencies performed some type of density or compaction monitoring. The majority of the agencies indicated a yield check of additives was performed. Most agencies monitor depth of milling, RAP maximum size and occasionally moisture content. Ohio reported testing moisture resistance of field produced mix.

Six agencies used test strips to control or monitor compaction, requiring a minimum compaction of 96 or 97 percent of control strip density. Four agencies required compaction based on a laboratory compacted sample. The requirements depended on the compaction procedure utilized which included standard proctor, 50-blow Marshall and Hveem compaction (AASHTO T 247). One agency used a method specification with a rolling pattern and one agency allowed all three procedures. Smoothness was checked by 11 agencies with nine using a straightedge, one using a profilograph and one using grade control on the paver.

Overlay Placement

Nine agencies had minimum moisture content requirements of the CIR mixture prior to overlay. Five agencies required less than 1.5% moisture, three less than 2.0% moisture and one less than 2.5% moisture. Two agencies also have provisions for residual moisture in the pavement, adjusting minimum moisture contents if the milled pavement had high residual moisture. Eight agencies had minimum CIR mixture cure times or required the mixture be overlaid within a limited time frame. Four agencies required additional rolling prior to placing the required overlay.

Mix Design

Nine of the thirteen agencies indicated they performed some form of preliminary pavement evaluation prior to CIR. Nine agencies required a mix design with seven of these agencies requiring the contractor provide the mix design. Three agencies have adopted the Road Science/SemMaterials mix design procedure for engineered emulsions. Of the four agencies that did not require mix designs, two indicated using typical recycling agent contents. Five agencies required either lime or an anti-strip agent with four agencies indicating their use only when required by the mix design.

Recycling Additives

Recycling additives varied from cationic slow and medium set emulsions to high float emulsions, with and without polymer modification, to engineered emulsions. Two agencies reported preferring expanded asphalt (foam) to asphalt emulsions. Only two agencies indicated they occasionally added virgin aggregates and one indicated this was only used in conjunction with lane widening.

Project Selection

Ten agencies indicated that they had no official traffic restrictions on the use of CIR. Of these ten agencies, four listed unofficial restrictions or qualifications. Two of these agencies said 15-16,000 ADT was the highest traffic they had cold recycled, one recommended no heavy truck traffic and all four said the majority of CIR had been on low to moderate trafficked pavements. Two agencies indicated they had traffic restrictions and reserved CIR for low to moderate traffic or < 4,000 ADT. Nevada, Kansas, Iowa, New Hampshire and New Mexico have all reported using CIR on Interstate pavements. Two agencies did not respond to the question. All agencies have a procedure for determining CIR eligibility. Most reported the procedure is a part of the agencies pavement management program. Most agencies indicated that CIR was reserved for pavements with functional, not structural, deficiencies. Other requirements included a minimum pavement structure to prevent pavement breakthrough of the equipment. All agencies indicated overlay thicknesses are designed based on traffic with chip seals being used for low volume roads and 1.5 to 3 inch HMA overlays reported as typical.

CHAPTER 4 - FIELD TEST DATA

SAMPLING AND TESTING

For Washington Road, sampling and compaction of field produced mix was performed by others. For the Quincy-Oroville Road project, samples were obtained by others but all field produced mix was compacted by the PI. Testing protocols were adjusted slightly after the Washington Road project based on preliminary results and ease/difficulty of testing. Therefore, the test plan for each project is presented separately, even though there is some duplication of procedures. Test results from samples obtained from the two CIR projects evaluated are presented in Appendix A.

CA PFH 123-1(1), WASHINGTON ROAD

CIR samples from CA PFH 123-1(1), Washington Road, were delivered in sealed 5-gallon plastic buckets to the bituminous laboratory at Oklahoma State University (OSU) for testing. Buckets were identified as to day of testing, station and lane (right or left). Samples consisted of loose mix (RAP) and compacted samples. Loose mix samples were obtained after the addition of asphalt emulsion.

Loose Mix Samples

Loose mix samples were oven dried at 140 ± 2°F (60 ± 1°C) to constant weight but no more than 48 hours and no less than 16 hours. Constant weight is defined as 0.05% change in weight in 2 hours. The samples were then reduced to testing size in accordance with AASHTO T 248. After reducing samples to the appropriate test size, the samples were tested for gradation (AASHTO T 27), maximum specific gravity (AASHTO T 209), asphalt content (AASHTO T 308) and gradation of recovered aggregate (AASHTO T 30).

RAP Gradation

Gradation of RAP was determined in accordance with AASHTO T 27. The results of the gradation analysis performed on the loose mix (RAP) samples are shown in Table A-1.

Maximum Specific Gravity

Two separate samples were obtained for determination of maximum theoretical specific gravity in accordance with AASHTO T 209. The dry-back procedure of AASHTO T 209 is usually performed on laboratory prepared specimens. However, this proved impractical for field samples due to the large amount of fine RAP present. During the dry-back procedure coarse particles dried to past a saturated surface dry (SSD) condition while the minus No. 8 materials remained considerably above an SSD condition. The results of AASHTO T 209 testing are shown in Table A-2.

Asphalt Content and Recovered Gradation

Where sufficient materials allowed, two separate samples were obtained for determination of asphalt content in accordance with AASHTO T 308. After asphalt content determination the gradation of recovered aggregate was determined in accordance with AASHTO T 30. The results are shown in Table A-3.

Field Compacted Samples

Field produced mix was returned to the field lab and compacted using either Marshall compaction, 75 blows per side using 4-inch (100 mm) mold, or with a Gyratory compactor to 35 gyrations using 6-inch (150 mm) mold. Samples were sealed in 5-gallon plastic buckets and sent to OSU. After arrival, compacted samples were cured at 140 ± 2°F (60 ± 1°C) to constant weight but no more than 48 hours and no less than 16 hours. Constant weight is defined as less than 0.05% change in weight in 2 hours. After curing, specimens were cooled at ambient temperature for a minimum of 12 hours. After curing and cooling at ambient temperatures, samples were tested for bulk specific gravity in accordance with AASHTO T 166 and AASHTO T 331 (CoreLok™). In order to prevent having to dry the samples after submersion in water, AASHTO T 331 was performed first and then AASHTO T 166. If the absorption from AASHTO T 166 sufficiently exceeded 2.0%, then the other replicates were tested using AASHTO T 331 only. Samples with absorptions near 2.0% were tested using both bulk specific gravity test procedures. Test results are presented with the corresponding performance tests.

AASHTO T 283 (Modified)

CIR samples are rarely tested in strict accordance with AASHTO T 283. Modifications are typically employed to account for cold mix samples. However, FLH wanted to evaluate the effect of AASHTO T 283 conditions on CIR mixtures so field mix samples were compacted at ambient temperatures using the SGC to 95 ± 5 mm and cured as described above. The remainder of the testing was performed in accordance with AASHTO T 283, excluding the optional freeze cycle. Bulk specific gravity of compacted and cured samples was determined in accordance with AASHTO T 166 unless the absorption exceeded 2.0% and then AASHTO T 331 was used. The results are shown in Table A-4. The results of the AASHTO T 283 testing are shown in Table A-5.

Retained Marshall Stability

Retained Marshall stability was determined on field mix samples using testing protocols usually employed for cold mix samples. Samples were compacted at ambient temperatures, 77 ± 4°F (25 ± 2°C), using 75-blow Marshall compaction and a 4-inch (100 mm) mold. Marshall samples were cured as described above. After curing, the specimens were cooled at ambient temperature for a minimum of 12 hours.

After curing and cooling, bulk specific gravity was determined in accordance with AASHTO T 166. Water absorption for Marshall samples was less than 2.0%; therefore, AASHTO T 331 testing was not required. After bulk specific gravity determination half of the samples from each location (station) were moisture conditioned by applying a vacuum of 13 to 67 kPa absolute pressures (254 to 660 mm of Hg partial pressure) for a time duration required to vacuum saturate samples to 55 to 75 percent. Saturation calculation was in accordance with AASHTO T 283. After vacuum saturation, moisture conditioned samples were soaked in a 77 ± 2°F (25 ± 1°C) water bath for 23 ± 1 hour, followed by a 30 to 40 min soak at 104 ± 2°F (40 ± 1°C). Dry specimens were double wrapped in leak proof bags and brought to test temperature by placing in a 104 ± 2°F (40 ± 1°C) water bath for 30 to 40 minutes.

Corrected Marshall Stability was determined in accordance with AASHTO T 245 at 104 ± 2°F (40 ± 1°C) after immersing in a water bath for 30 to 40 minutes. Dry specimens were not allowed to come in contact with water. Dry specimens were tested at the same time as moisture-conditioned specimens. Retained Marshall stability is the average moisture conditioned specimen strength divided by the average dry specimen strength. Test results are shown in Table A-6.

Dynamic Modulus (E)*

Dynamic modulus was determined on field mix samples compacted at ambient temperature using the SGC to a height of 160 to 175 mm. Field mix samples were cured as described above. After curing and cooling to ambient temperature, bulk specific gravity was determined in accordance with AASHTO T 166 and AASHTO T 331. If the absorption from AASHTO T 166 sufficiently exceeded 2.0%, then the other replicates were tested using AASHTO T 331 only. Samples with absorptions near 2.0% were tested using both procedures.

Dynamic modulus was performed in accordance with AASHTO TP 62 on samples sawed and cored to 100 mm diameter by 150 mm tall. After curing as described above, dynamic modulus samples were trimmed to test size. After sawing and coring to test size, samples were dried using vacuum drying (ASTM D7227). After drying, sawed and cored samples were tested for bulk specific gravity in accordance with AASHTO T 166 and AASHTO T 331. In order to prevent having to dry the samples after submersion in water, AASHTO T 331 was performed first and then AASHTO T 166. If the absorption from AASHTO T 166 sufficiently exceeded 2.0%, then the other replicates were tested using AASHTO T 331 only. Samples with absorptions near 2.0% were tested using both procedures. Submerged samples were vacuum dried (ASTM D7227) a second time prior to dynamic modulus testing in accordance with AASHTO TP 27. The results of bulk specific gravity and dynamic modulus testing are shown in Tables A-7 and A-8, respectively.

Lab Molded SGC Samples

Field mix samples were compacted to 35 gyrations in the SGC at ambient temperatures. Sample mass was selected to give a compacted height of 115 ± 5 mm. After compaction, samples were cured as described above. After cooling to ambient temperatures, samples were tested for bulk specific gravity in accordance with AASHTO T 166 and AASHTO T 331. In order to prevent having to dry the samples after submersion in water, AASHTO T 331 was performed first and then AASHTO T 166. If the absorption from AASHTO T 166 sufficiently exceeded 2.0%, then the other replicates were tested using AASHTO T 331 only. Test results are shown in Table A-9.

QA Density Testing

Quality assurance (QA) data performed by contractor's personnel and supplied by FLH consisted of nuclear density test results on the compacted CIR mat and moisture content determination. Moisture content was determined using a microwave oven and conventional oven. Moisture contents were obtained before and after initial curing. Nuclear density tests were performed in backscatter mode and although not specified, it is believed supplied data consisted of moist or total bulk specific gravity. Nuclear gauges determine moisture content based on the presence of hydrogen atoms. Hydrogen atoms are then correlated to water and the reported dry unit weight is calculated from wet unit weight and moisture content. However, CIR mixtures contain hydrogen atoms from sources other than water. Hydrogen atoms are present as existing asphalt cement, moisture from the existing pavement, asphalt emulsion (asphalt and water) and water added during the CIR process. Therefore, nuclear gauge dry unit weights and moisture contents have unclear meanings in construction testing involving asphalt cements and water. Asphalt content gauges only report one density or unit weight, a wet or total unit weight. Test results from Washington Road are shown in Table A-10.

CA PFH 119-1(3), QUINCY-OROVILLE ROAD

After completion of the Washington Road project, a meeting was held with FLH personnel and the test plan for Quincy-Oroville Road was modified slightly. Marshall testing was deleted because not all FLH offices have Marshall equipment and ideally FLH wants to use the most relevant equipment available and reduce their testing equipment needs. In addition, the use of ASTM D6857 (CoreLok™) maximum theoretical specific gravity as a possible replacement for the dry-back procedure of AASHTO T 209 was added.

CIR samples from CA PFH 119-1(3), Quincy-Oroville Road, were obtained by the prime contractor's quality control personnel and delivered in sealed 5-gallon plastic buckets to CFLHD's mobile field laboratory in Quincy, CA for testing. Buckets were identified as to day of sampling, station and lane (right or left). Samples consisted of loose mix (RAP) obtained from the windrow after discharge from the pugmill but prior to placement by the screed. Samples were generally collected twice per day, mornings and afternoons. Due to the distance from the mobile laboratory to the job site, samples obtained in the mornings were tested that afternoon and samples obtained in the afternoon were tested the next morning. Field samples were reduced to testing size in accordance with AASHTO T 248 when received. All samples were fabricated and tested on site by the PI with the exception of dynamic modulus testing.

Loose Mix Samples

After reducing loose mix samples to the appropriate test size, and prior to testing, samples were oven dried at 140 ± 2°F (60 ± 1°C) to constant weight as described for Washington Road. Samples were tested for gradation (AASHTO T 27), maximum specific gravity (AASHTO T 209 and ASTM D6857), asphalt content (AASHTO T 308) and gradation of recovered aggregate (AASHTO T 30).

RAP Gradation

Two samples were tested for RAP gradation analysis in accordance with AASHTO T 27. Results of the gradation analysis performed on the loose mix (RAP) samples are shown in Table A-11.

Maximum Specific Gravity

Following gradation analysis, the two samples were next tested for determination of maximum theoretical specific gravity in accordance with AASHTO T 209. The dry-back procedure of AASHTO T 209 is usually performed on laboratory prepared CIR specimens due to the possibility of water absorption under vacuum. However, this proved impractical for field samples due to the time required and the amount of scheduled testing. Therefore, ASTM D6857 was performed on the samples first followed by AASHTO T 209 testing as water absorption under vacuum is not as significant an issue with ASTM D6857. Test results are shown in Table A-12.

Asphalt Content and Recovered Gradation

After maximum specific gravity testing, the samples were oven dried and then tested for asphalt content in accordance with AASHTO T 308. After asphalt content determination gradation of the recovered aggregate was determined in accordance with AASHTO T 30. Test results are shown in Table A-13.

Field Compacted Samples

After reducing loose mix samples to the appropriate test size, samples were compacted to 35 gyrations in the SGC for lab molded air voids, modified AASHTO T 283 testing, and dynamic modulus testing. Samples were compacted in general accordance with AASHTO T 312 except at ambient temperatures with no oven-aging. Samples obtained in the afternoons were compacted the next morning and sample temperatures were usually in the 70-75^{o}F range. Samples obtained in the mornings were tested in the afternoon at ambient temperatures, typically 80-90^{o}F.

Compacted samples were cured to constant weight as previously described. After curing, specimens were cooled at ambient temperatures for a minimum of 12 hours and then tested for bulk specific gravity in accordance with AASHTO T 166 and AASHTO T 331. Test results are presented with the corresponding performance tests.

Lab Molded Voids

Two field mix samples were compacted at ambient temperatures to 35 gyrations in the SGC. Sample mass was selected to give a compacted height of 115 ± 5 mm. After compaction, samples were cured as described above. After curing and cooling at ambient temperatures, samples were tested for bulk specific gravity in accordance with AASHTO T 166 and AASHTO T 331 as the water absorption exceeded 2.0%. In order to prevent having to dry samples after submersion in water, AASHTO T 331 was performed first and then AASHTO T 166. Test results are shown in Table A-14.

AASHTO T 283 (Modified)

CIR samples are rarely tested in strict accordance with AASHTO T 283. A modified AASHTO T 283 procedure is typically used.

Field mix samples were compacted at ambient temperatures to 35 gyrations using the SGC. Sample mass was selected to give a compacted height of 95 ± 5 mm. After compaction samples were cured as described above. Bulk specific gravity of compacted and cured samples was determined in accordance with AASHTO T 331 as the water absorption determined in accordance with AASHTO T166 exceeded 2.0%. It was originally desired to condition samples to 70-80% vacuum saturation. However, due to the high air void content and the use of AASHTO T 331 bulk specific gravities, this proved impractical as too much water escaped from the sample between vacuum saturation and SSD measurements to achieve greater than 70% saturation. The difference between AASHTO T 166 and AASHTO T 331 are interconnected voids that cannot hold water. When lifting a sample from the water bath, water from these pores runs out before it can be measured. Therefore, saturation levels were reduced and conditioned samples were vacuum saturated to between 55 and 75 percent saturation. Fifty-five to 75 percent saturation levels are used in most published CIR mix design procedures.

After vacuum saturation, conditioned samples were soaked in a 25^{o}C water bath for 24 ± 1 hour and tested for indirect tensile strength in accordance with AASHTO T 283. The optional freeze cycle was omitted. Dry or unconditioned samples were placed in leak-proof bags and placed in a 25^{o}C water bath for 2 hours prior to indirect tensile strength testing. The results of the modified AASHTO T 283 testing are shown in Table A-15.

Dynamic Modulus (E)*

Dynamic modulus was determined on field mix samples compacted to 35 gyrations in the SGC at ambient temperatures. Sample mass was selected to give a compacted height of 160 to 175 mm. After compaction the samples were wrapped and sealed in 5-gallon buckets and sent to OSU for further sample preparation and testing. Several samples were damaged during shipment and could not be tested.

After arrival at OSU, the field mixed dynamic modulus samples were cured and tested for bulk specific gravity as described for Washington Road. Samples were then sawed and cored to 100 mm diameter by 150 mm tall. After sawing and coring to test size, the samples were dried using vacuum drying (ASTM D7227). After drying the sawed and cored samples were tested for bulk specific gravity in accordance with AASHTO T 166 and AASHTO T 331 as previously described. Results are shown in Table A-16. After bulk specific gravity testing, any samples that underwent AASHTO T 166 testing were vacuum dried and then all samples were vacuum sealed and sent to CFLHD laboratories where dynamic modulus testing was performed in accordance with AASHTO TP 79, the AMPT procedure. The results are shown in Table A-17.

QA Density Testing

Quality assurance (QA) data performed by contractor's personnel and supplied by FLH consisted of nuclear density test results on the compacted CIR mat. Nuclear density tests were performed in backscatter mode and although not specified, it is believed supplied data consisted of moist or total bulk specific gravity. As stated for Washington Road, dry unit weights and moisture contents have unclear meanings in construction testing involving asphalt cements and water. Test results from Quincy-Oroville Road are shown in Table A-18.

In-Place Unit Weight

A limited number of samples were saw-cut from the compacted CIR mat by the contractor's QC personnel after the mat had cured. CIR mat density was measured at the same locations using a nuclear density meter in backscatter mode. Results were supplied as unit weight and pounds moisture. It is unclear whether the unit weight is a moist or dry unit weight. The saw-cut samples were supplied for bulk specific gravity testing. The samples were tested for bulk specific gravity in accordance with AASHTO T 331 as the water absorption exceeded 2.0%. It was unclear whether Nuclear gauge results are "moist densities;" therefore, bulk specific gravities were determined "moist" and "dry;" however, the saw-cut samples were basically dry when tested. Results are shown in Table A-19.

CHAPTER 5 - SUMMARY STATISTICS

INTRODUCTION

Summary statistics of average and standard deviation were calculated for test properties evaluated on Washington Road (WR) and Quincy-Oroville Road (QO). Most test properties evaluated are the average of two or more observations (Gmm is the average of two samples); therefore, averages and standard deviations are calculated on average values, not individual test results. This has no effect on the average but does have an effect on the standard deviation. Summary statistics for test properties evaluated for WR and QO are shown in Table 5. Each test property is discussed below.

Table 5. Summary Statistics.

	Washington Road			Quincy-Oroville Road		
Property	Average	Standard Deviation	Coef. Variation (%)	Average	Standard Deviation	Coef. Variation (%)
RAP Gradation						
3/4" Sieve	93.4	5.1	5.4	89.9	2.2	2.5
No. 4 Sieve	44.2	13.3	30.0	30.7	5.2	16.9
No. 30 Sieve	8.0	3.5	44.2	0.59	0.27	46.1
AC (%)	6.05	0.5	7.9	8.65	1.09	12.5
Aggregate Gradation						
3/4" Sieve	99.2	0.8	0.8	99.7	0.6	0.6
No. 4 Sieve	69.9	4.0	5.8	65.6	6.0	9.2
No. 30 Sieve	28.0	2.4	8.6	25.7	3.0	11.8
No. 200 Sieve	7.75	0.69	8.9	11.24	0.96	8.5
Lab Molded Voids (115 mm Samples)						
VTM (%)	16.9	2.9	17.2	17.1	3.8	22.2
Bulk Specific Gravity (95 mm Samples)						
AASHTO T 166	2.093	0.0240	1.1	2.093	0.081	3.9
CoreLok	2.072	0.0280	1.4	2.026	0.111	5.5
Air Voids (95 mm Samples)						
AASHTO T 166	15.3	1.6	10.1	14.3	2.5	17.3
AASHTO T 331	16.2	1.7	10.6	17.1	3.8	22.3
Marshall Stability						
Conditioned	1466	384	26.2	N/T	N/T	N/T
Dry	2868	1057	36.9	N/T	N/T	N/T
Retained Ratio	0.54	0.13	23.4	N/T	N/T	N/T

Table 5 (Con't.). Summary Statistics.

Property	Washington Road Average	Washington Road Standard Deviation	Washington Road Coef. Variation (%)	Quincy-Oroville Road Average	Quincy-Oroville Road Standard Deviation	Quincy-Oroville Road Coef. Variation (%)
AASHTO T 283						
Conditioned ITS	24.0	6.0	24.9	*60.7*	*14.8*	*24.4*
Dry ITS	50.7	11.6	22.8	*73.1*	*17.8*	*24.3*
TSR	0.48	0.09	19.5	*0.84*	*0.12*	*14.2*
Maximum Specific Gravity						
T 209 Gmm	2.474	0.023	0.9	2.452	0.035	1.4
T 209 Gse	2.728	0.020	0.7	2.837	0.061	2.2
D 6857 Gmm	N/T	N/T	N/T	2.440	0.039	1.6
D 6857 Gse	N/T	N/T	N/T	2.817	0.050	1.8
Dynamic Modulus (10 Hz.)						
4 C	1,247,418	225,295	18.1	1,089,464	239,467	22.0
20 C	525,137	53,713	10.2	599,705	115,627	19.3
35 C	285,916	42,708	14.9	273,877	49,425	18.0
Dynamic Modulus (1 Hz.)						
4 C	836,640	147,571	17.6	883,444	190,889	21.6
20 C	321,950	53,202	16.5	401,155	73,676	18.4
35 C	148,783	35,457	23.8	138,333	27,297	19.7
Dynamic Modulus (0.1 Hz.)						
4 C	567,245	93,557	16.5	675,579	144,740	21.4
20 C	188,692	55,096	29.2	237,262	45,990	19.4
35 C	90,321	24,039	26.6	61,905	14,225	23.0
QA Field Data						
Unit Wt. (pcf)	115.5	4.4	3.8	119.5	4.7	3.9
% Compaction	90.7	3.5	3.9	94.8	2.9	3.1

Represents modified AASHTO T 283, no hot soak.

RAP Gradations

RAP gradations were performed in accordance with AASHTO T 27 and are dry, unwashed gradations. Results presented are average gradations of two split samples from the CIR mixture sampled from the windrow after addition of asphalt emulsion. Variation in RAP and aggregate gradations along WR and QO are shown in Figures 1 and 2, respectively.

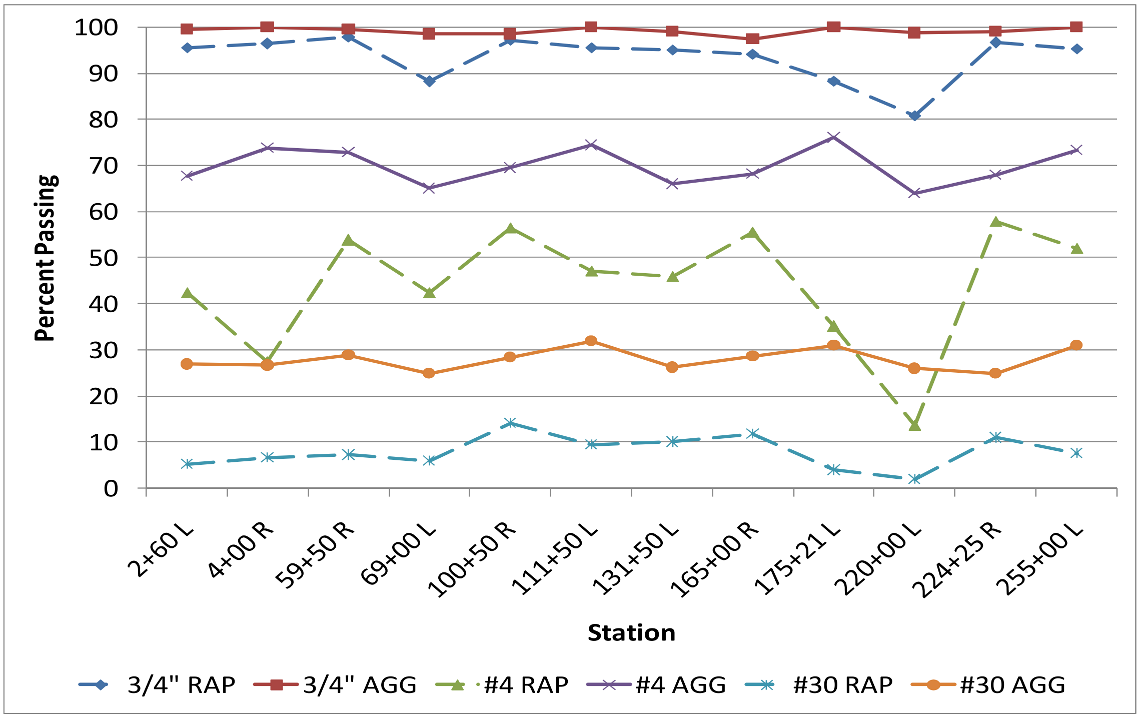

Figure 1. Chart. RAP and Aggregate Gradation, Washington Road.

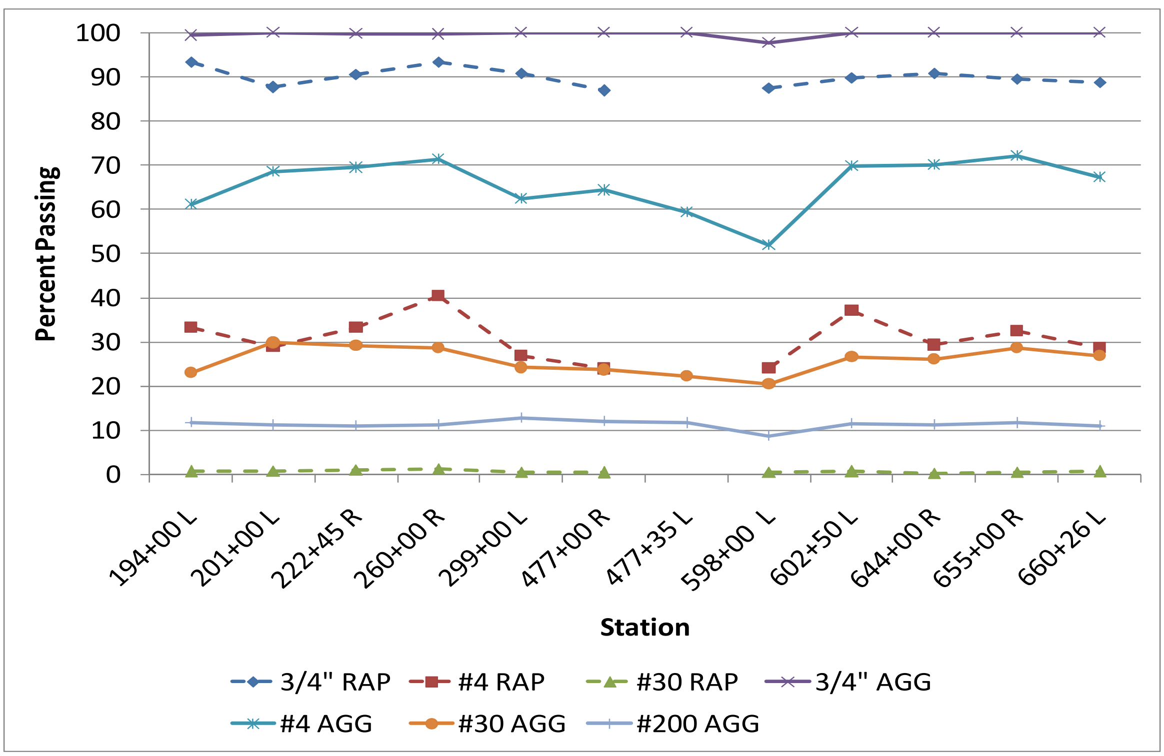

Figure 2. Chart. RAP and Aggregate Gradation, Quincy-Oroville Road.

As shown in Table 5, RAP gradation for QO was slightly finer on the 3/4-inch sieve but coarser on the No. 4 and 30 sieves than RAP from WR. There was a minimal amount of material finer than the No. 200 sieve for both projects. This would be expected for materials mixed with emulsion and from a dry or unwashed sieve analysis.

Standard deviations were higher for WR than QO, indicating a less uniform RAP gradation for WR. According to design report for WR [(2)], planned CIR milling depths were deeper than the existing HMA thickness in some places, especially from the beginning of the project to sta. 180+00. Differences in gradation and standard deviations could be attributed to the presence of base material that appears to have been incorporated into the CIR mix on WR in some locations.

Maximum Specific Gravity

After determining RAP gradation, maximum specific gravity was determined on the same samples. Results are the average of two samples. For WR, maximum specific gravity was determined in accordance with AASHTO T 209. However, for RAP materials, the dry back procedure of AASHTO T 209 is often required due to the presence of uncoated particles. AASHTO T 209 is generally performed on HMA where all particles are coated with asphalt and there is little to no fines that are not attached to larger particles or present as conglomerates.

This was not the case for WR where aggregate base was incorporated into the CIR mixture in some areas. Performance of the dry-back procedure proved impractical for field samples from WR due to the large amount of fine aggregate size particles present. Coarse particles dried to past a saturated surface dry (SSD) condition while the minus No. 8 materials remained considerably above the SSD condition. Because of the presence of fines and uncoated materials, and the impracticality of performing the dry back procedure, maximum specific gravity values for WR are probably high because uncoated aggregates absorb water, reducing the measured volume of the mix, increasing maximum specific gravity.

Because of the issues with determination of Gmm on WR, the use of the CoreLok™ procedure, ASTM D6857, was investigated on QO. Samples were tested for Gmm using ASTM D6857 and then tested using AASHTO T 209. However, the dry back procedure of AASHTO T 209 was not employed as the time required was not practical based on the amount of additional testing required.

Maximum specific gravity of RAP is a function of asphalt content, specific gravity of aggregate and gradation of RAP particles. Variability of Gmm along a project would be an indication of variability in one or more of these variables. Effective specific gravity of the aggregate (Gse) can be calculated from Gmm results if the asphalt content and specific gravity of the asphalt cement are known. Total asphalt content was available from the test results of AASHTO T 308. An assumed value of the asphalt cement specific gravity was used. Changes in Gse of the aggregate would be an indication of a change in the source of aggregates. Variation in Gmm and Gse are shown in Figures 3 and 4 for WR and QO, respectively.

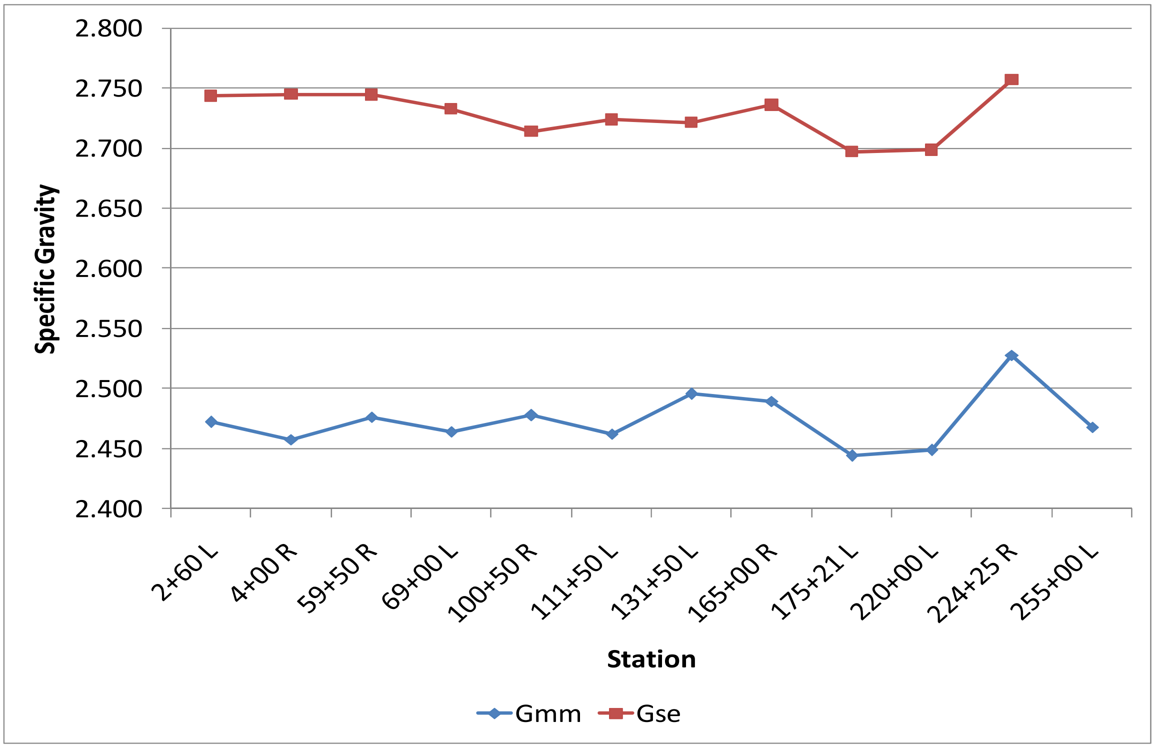

Figure 3. Chart. Maximum and Effective Specific Gravity, Washington Road.

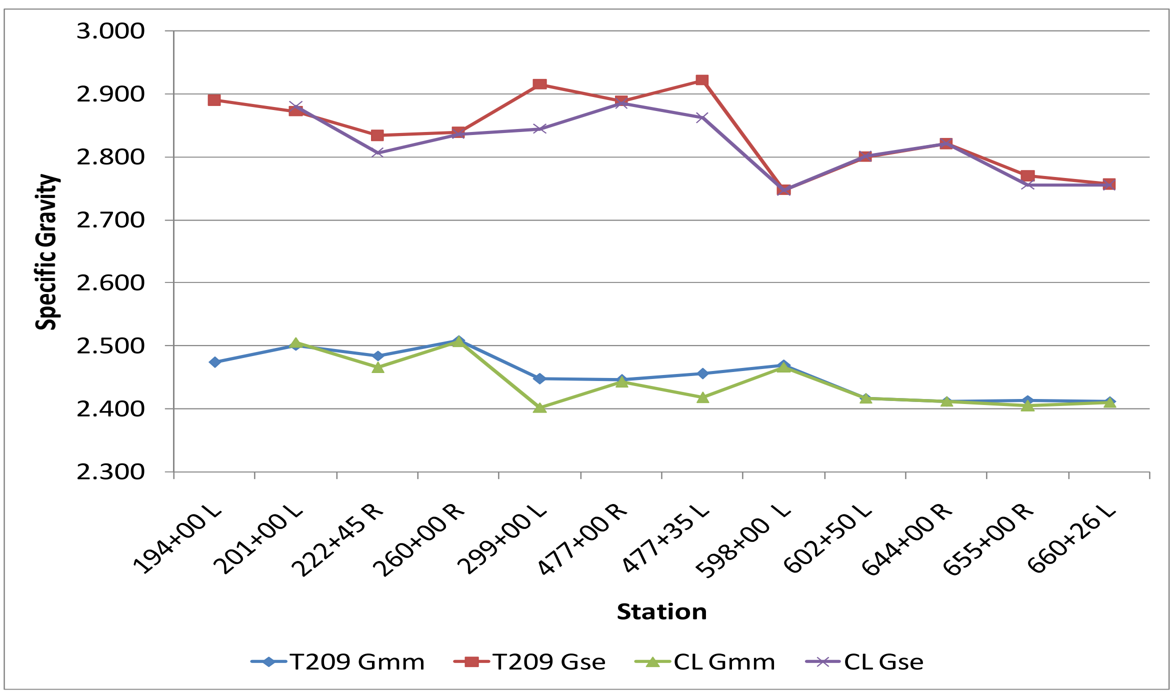

Figure 4. Chart. Maximum and Effective Specific Gravity From AASHTO T 209 and ASTM D6857, Quincy-Oroville Road.

The standard deviation of AASHTO T 209 Gmm and Gse was slightly higher for QO compared to WR. This indicates that there were possibly more different mixtures encountered along QO than WR. A comparison of Gmm values shows that AASHTO T 209 resulted in an average Gmm 0.012 higher than ASTM D6857. Standard deviation for AASHTO T 209 was 0.035 compared to 0.039 for ASTM D6857. With a little practice, ASTM D6857 is as easy to perform as AASHTO T 209. Although ASTM D6857 is affected by uncoated aggregate, in instances where the dry back procedure of AASHTO T 209 is not practical to perform, ASTM D6857 could be an attractive alternative.

Asphalt Content

After Gmm testing, the samples were dried to a constant mass and tested for asphalt content in accordance with AASHTO T 308. An aggregate correction factor of 0.0 was used (no correction) as one could not be easily determined from the materials available. The variations in asphalt content for WR and QO are shown in Figures 5 and 6, respectively. The overall average asphalt content was lower for WR compared to QR. This is likely an indication of different asphalt emulsion application rates and uncoated aggregate base in the CIR mixture as much as an indication of HMA mixes with finer gradations and higher asphalt contents. The standard deviation was lower for WR, 0.5 to 1.1 percent, respectively, indicating a more consistent total asphalt content and possibly a more uniform section for WR compared to QO.

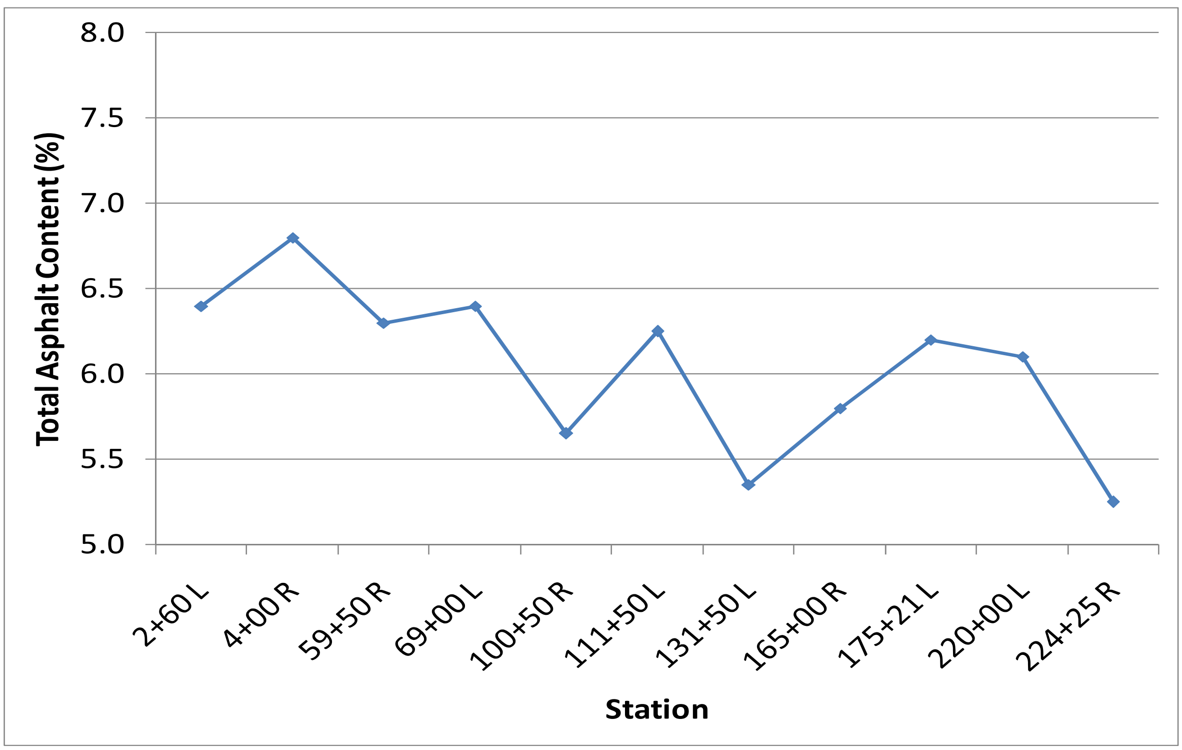

Figure 5. Chart. Total Asphalt Content, Washington Road.

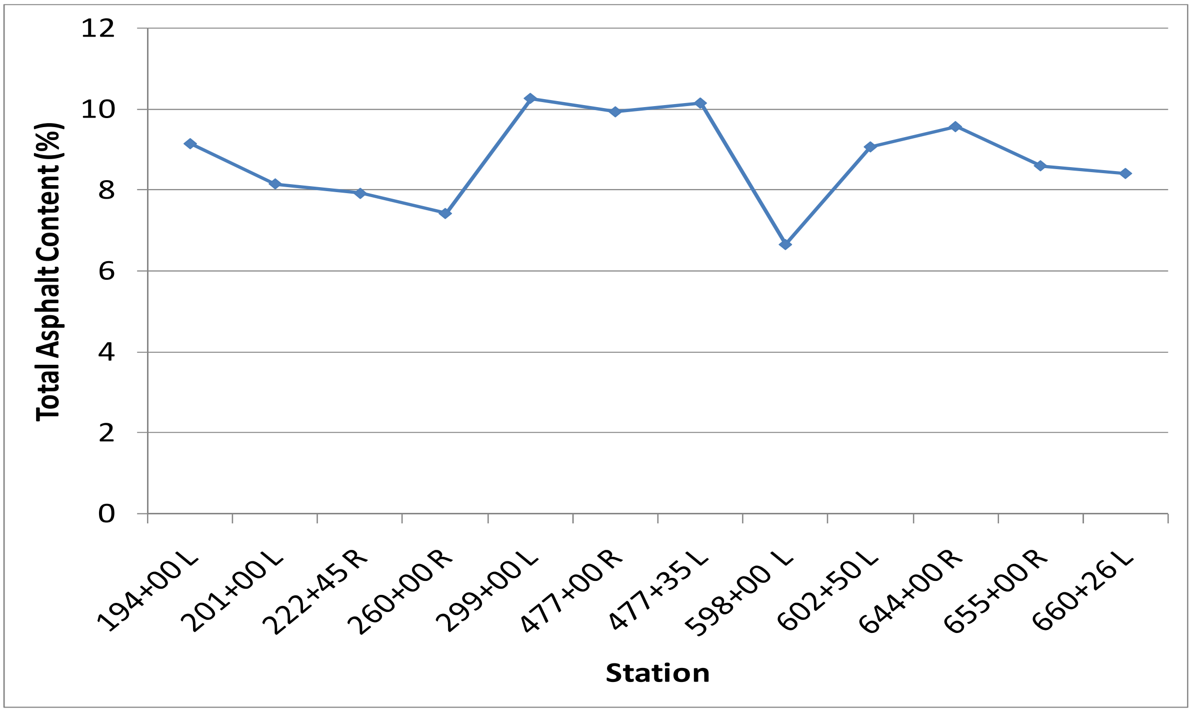

Figure 6. Chart. Total Asphalt Content, Quincy-Oroville Road.

Asphalt contents were higher along the upper end of WR where the project design report (2) indicated thicker HMA sections and hence less presence of uncoated aggregate in the CIR. One section of QR, sta. 598+00 L, had a lower asphalt content than surrounding areas. There is a possibility that sta. 598+00 L was in an area that contained an aggregate patch prior to recycling.

Aggregate Gradations

Aggregates recovered from the ignition furnace were tested for gradation analysis in accordance with AASHTO T 30. Variations in aggregate gradation were shown in Figures 1 and 2 for WR and QO, respectively. Average gradations and standard deviations were similar with the exception of percent passing the No. 200 sieve. There was considerably more material passing the No. 200 sieve for QO than WR. Inclusion of aggregate base in some areas of WR, which according to the project engineering report (2) was very clean, could have affected the results. It is worth noting that changes in aggregate gradation do not necessarily follow changes in RAP gradation.

Cores obtained for the mix design for QO indicated different HMA mixes present as well as did color of recovered aggregate from the upper to lower end of the project.

Lab Molded Voids

Lab molded density is used to check void properties in HMA and has been used in the past to help control compaction of HMA. Samples of field produced mix were compacted to 35 gyrations in the SGC at ambient temperatures. Sample mass was selected to give a compacted specimen height of 115 ± 5 mm. After compaction the samples were cured, cooled and bulk specific gravity determined in strict accordance with AASHTO T 166. For samples where water absorption exceeded 2.0% by volume, bulk specific gravity was determined in accordance with AASHTO T 331. Voids total mix (VTM) was calculated using the appropriate Gmm. Reported voids are the average of three samples for WR and two samples for QO and are shown in Table 6. Results are presented graphically in Figures 7 and 8 for WR and QO, respectively.

Table 6. Lab Molded Voids.

Washington Road		Quincy-Oroville Road		
Sta.	VTM (%)	Sta.	VTM (%)	Compacted
2+60 L	17.4	194+00 L	24.05	AM
4+00 R	16.8	201+00 L	10.66	PM
59+50 R	17.5	222+45 R	14.16	PM
69+00 L	19.2	260+00 R	15.215	PM
100+50 R	15.6	299+00 L	17.89	AM
111+50 L	18.3	477+00 R	18.98	AM
131+50 L	14.8	477+35 L	19.14	AM
175+21 L	14.2	598+00 L	12.12	PM
220+00 L	17.4	602+50 L	19.61	AM
225+00 R	23.2	644+00 R	20.01	AM
255+00 L	12.0	655+00 R	19.15	AM
		660+26 L	14.44	PM
Average	16.9		17.1	
Std. Dev.	2.9		3.8	

All samples for QO exceeded 2.0% absorption and bulk specific gravity was determined in accordance with AASHTO T 331. For WR, all but three stations had absorptions above 2.0% and used AASHTO T 331, stations 100+50 R, 131+50 L and 225+00 R. For station 100+50 R, all three samples had less than 2.0% absorption and used AASHTO T 166 to calculate the average. For stations 131+50 L and 225+00 R, two of the three samples had adsorptions less than 2.0%. If AASHTO T 166 adopts a maximum adsorption of < 1.0%, all samples would have required AASHTO T 331 testing.

The overall average VTM was 16.9 and 17.1 percent for WR and QO, respectively. The standard deviation for QO was 3.8% compared to 2.9% for WR, a difference of 0.9%.

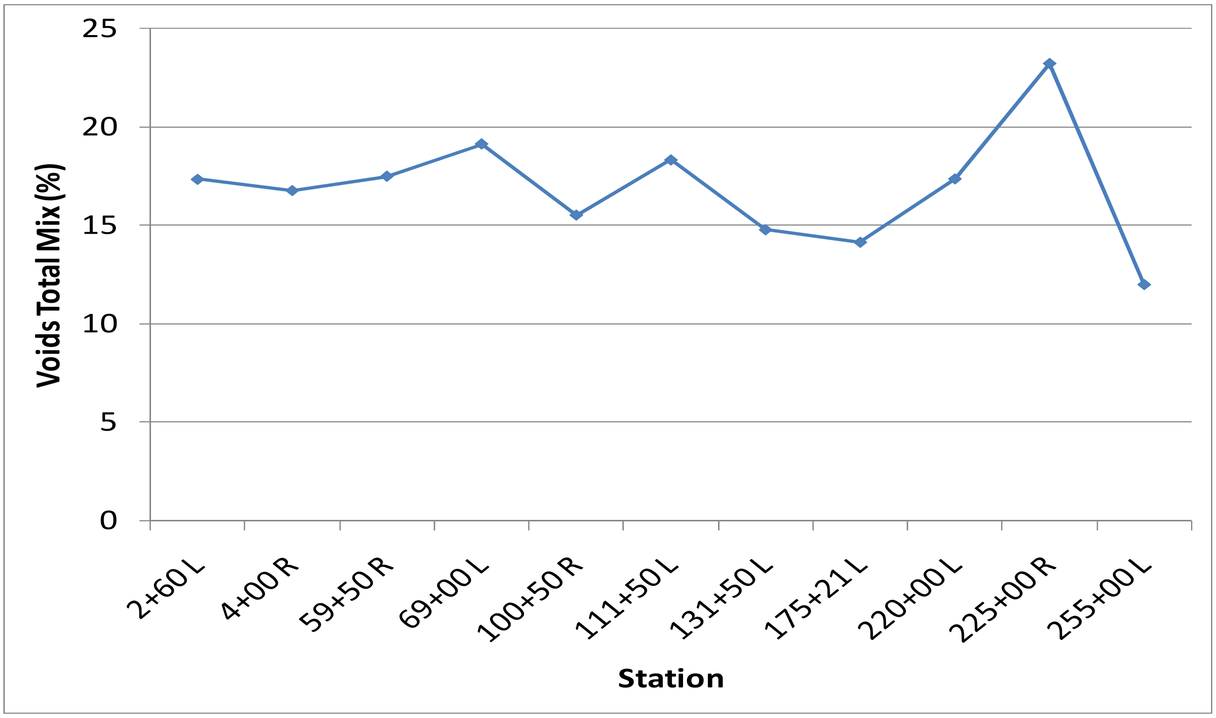

Figure 7. Chart. Lab Molded Voids Total Mix, Washington Road.

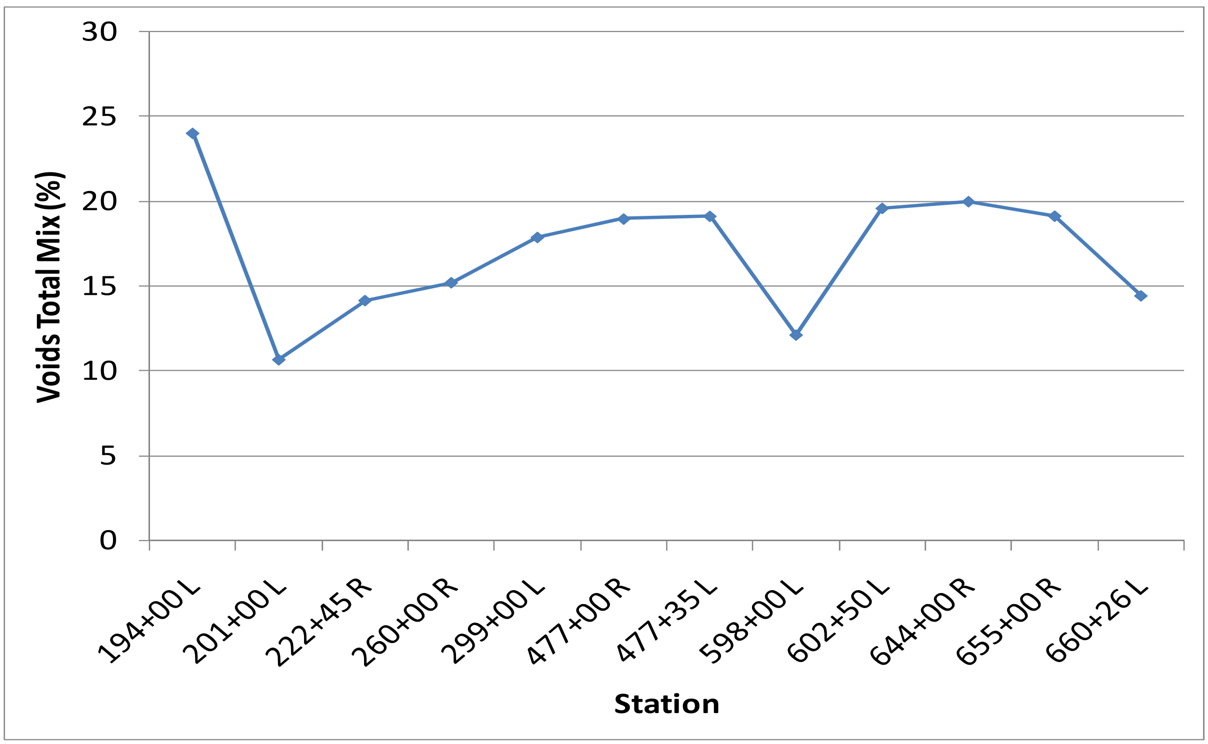

Figure 8. Chart. Lab Molded Voids Total Mix, Quincy-Oroville.

AASHTO T 283 and Retained Marshall Stability

Marshall Stability

A modified Marshall stability and retained stability test is often used for evaluation of cold mixes. The procedure was performed for the mix designs for WR and during field testing but was not used for QO because FLH ideally wants to use the most relevant equipment and mix design methodology and reduce their testing equipment needs. Marshall stability is determined on samples at 40°C with conditioned samples undergoing vacuum saturation and a 23 hour soak at 25°C. Summary statistics for Marshall stability testing were shown in Table 5. Variation in Marshall stability and conditioned Marshall stability for WR is shown in Figure 9 and retained stability ratio is shown in Figure 10.

It is interesting to note that there was considerably more variability in dry rather that conditioned Marshall stability values. Dry Marshall stability averaged 2,868 lbs with a standard deviation of 1,057 lbs and conditioned samples had an average Marshall stability of 1,466 lbs with a standard deviation of 384 lbs. The coefficient of variation was 37% for dry stability and 26% for conditioned stability. Average retained Marshall stability ratio was 0.54 with a standard deviation of 0.13.

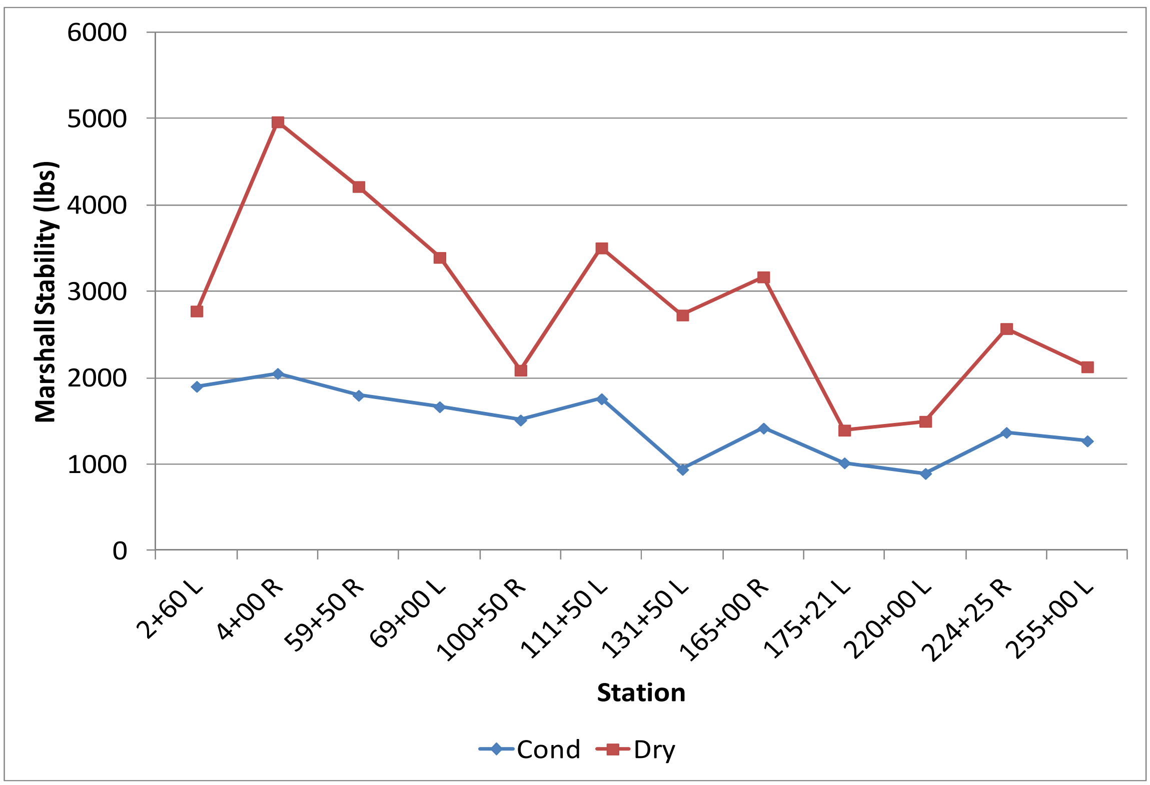

Figure 9. Chart. Marshall Stability, Washington Road.

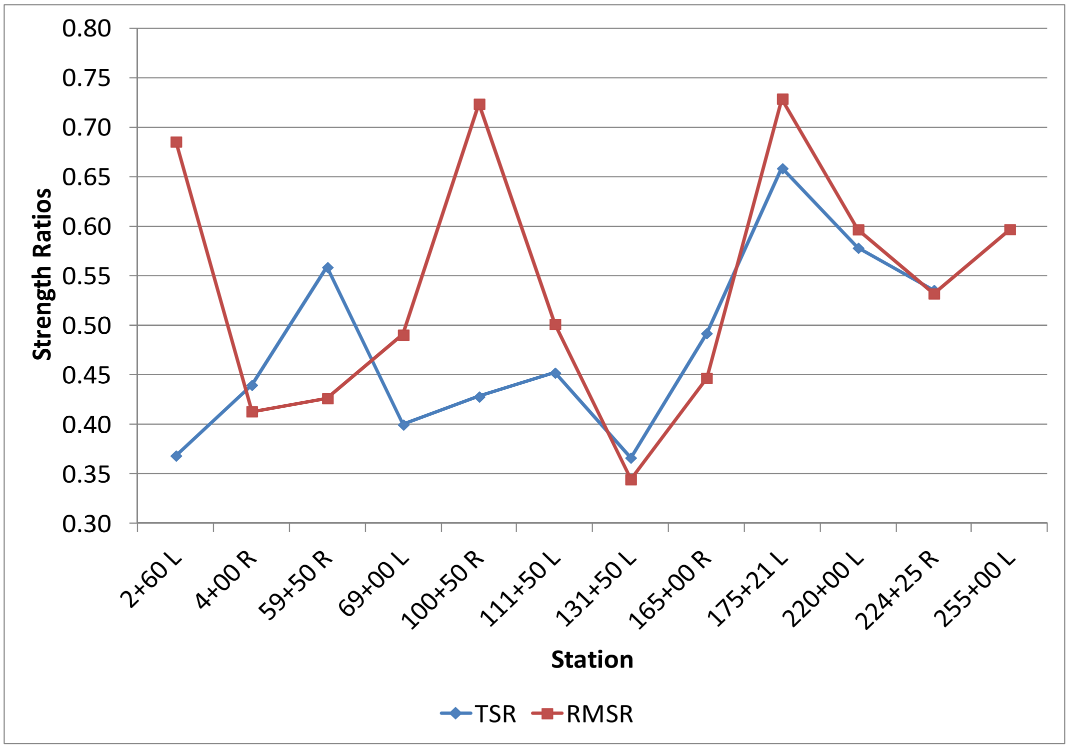

Figure 10. Chart. Retained Marshall Stability and TSR, Washington Road.

AASHTO T 283

For WR, tensile strength ratio (TSR) tests were performed in accordance with AASHTO T 283 without the optional freeze cycle but including the hot soak. Samples were compacted and cured as previously described. This is not the customary method for testing cold mixes but there was a desire expressed by FLH to determine the effect of the hot soak typically used for HMA on CIR mixtures. Mix designs were performed using a modified AASHTO T 283 without the optional freeze cycle and hot soak. Summary statistics are shown in Table 5. Tensile strengths are presented graphically in Figure 11; TSRs were shown in Figure 10 with retained stability ratios for comparison.

The hot soak appears to have had a significant impact on conditioned tensile strength and TSRs. Dry tensile strengths averaged 50.7 psi with a standard deviation of 11.6 psi and conditioned samples had an average tensile strength of 24.0 psi with a standard deviation of 6.0 psi. The average TSR was 0.48 with a standard deviation of 0.09. The low TSR and retained Marshall stability ratio could be a function of a lower than designed emulsion content and/or untreated aggregate base incorporated into the CIR mixture.

Due to the severity of the hot soak, AASHTO T 283 samples for QO were tested using the more conventional procedure for cold mixes, replacing the 23 hour hot soak with a 24 hour soak at 25°C. Summary statistics were shown in Table 5. Tensile strengths and TSRs for QO are shown in Figures 12 and 13, respectively. Dry tensile strengths averaged 73.1 psi with a standard deviation of 17.8 psi and conditioned samples had an average tensile strength of 60.7 psi with a standard deviation of 14.8 psi. Coefficients of variation were similar for dry and conditioned tensile strengths, 24% each, lower than for Marshall stability. Average TSR was 0.84 with a standard deviation of 0.12.

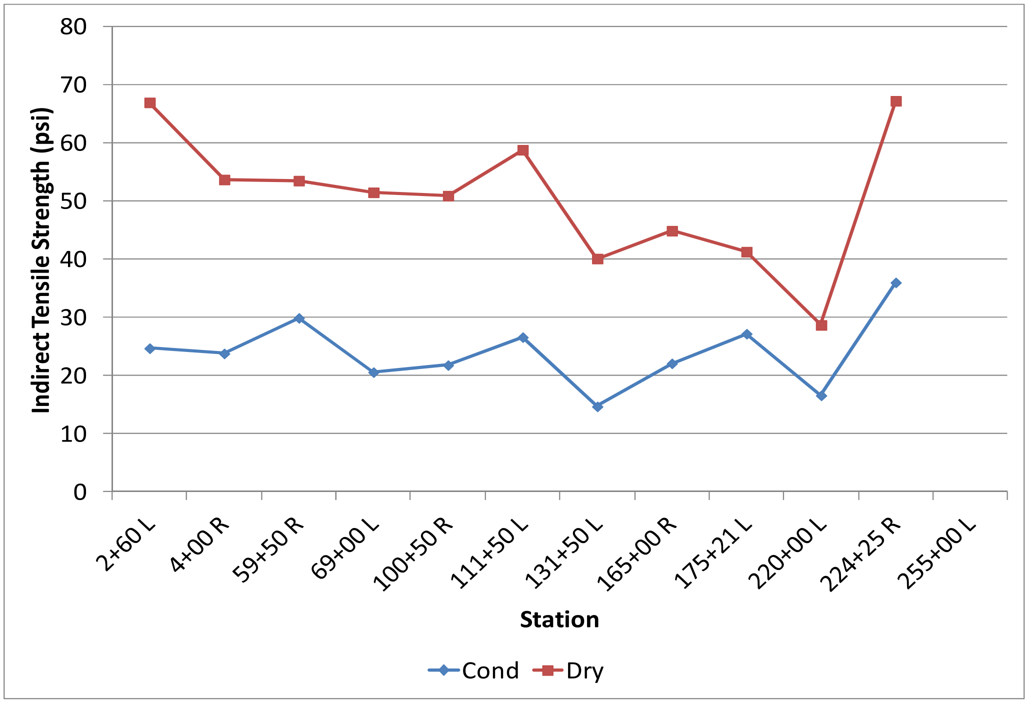

Figure 11. Chart. AASHTO T 283 Indirect Tensile Strengths, Washington Road.

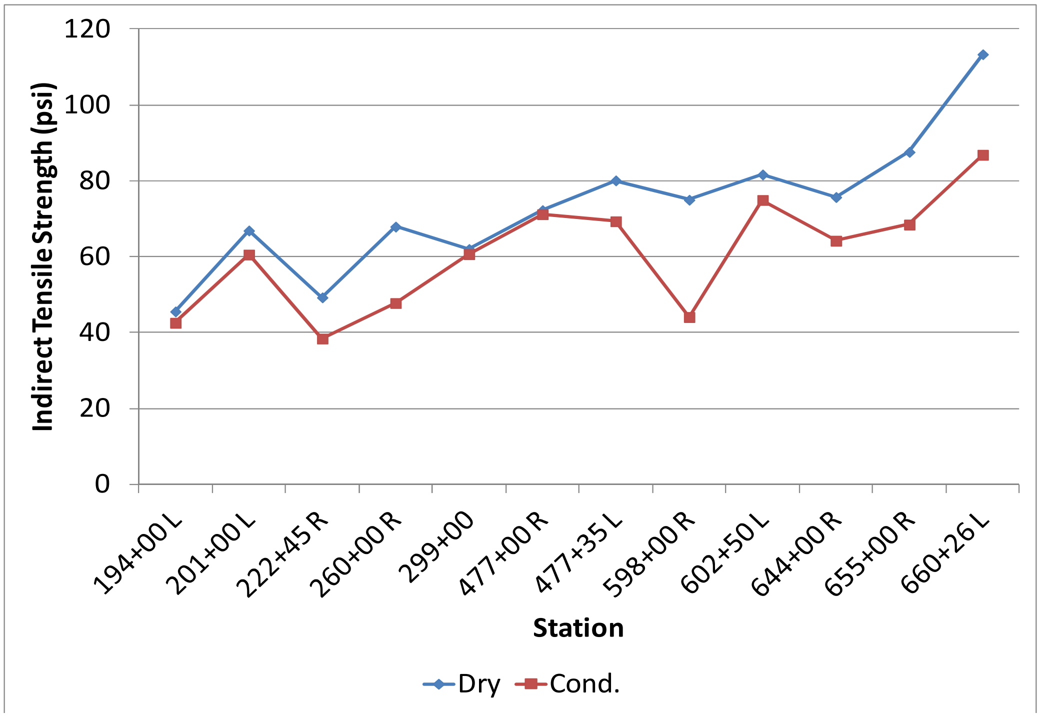

Figure 12. Chart. AASHTO T 283 Indirect Tensile Strengths, Quincy-Oroville Road.

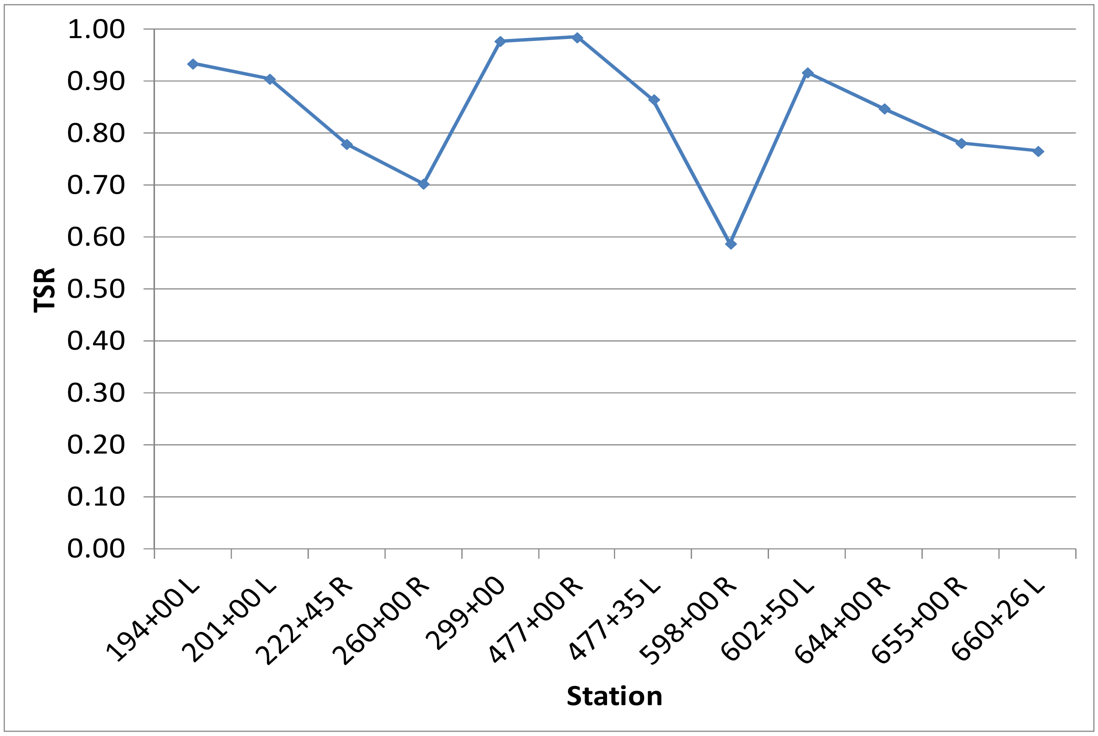

Figure 13. Chart. AASHTO T 283 TSR, Quincy-Oroville Road.

Dynamic Modulus

Dynamic modulus results for WR and QO are shown in the appendix in Tables A-8 and A-17, respectively. Summary statistics at 1 and 0.5 Hz are shown in Table 5. One of the objectives of this study was to recommend an AASHTO structural layer (a) coefficient for use with CIR in the *1993 AASHTO Design Guide for Design of Pavement Structures*. AASHTO structural layer (a) coefficients are based on resilient modulus at 20°C (68°F); therefore, dynamic modulus at the same temperature was selected to determine a frequency that would give similar results to resilient modulus. Resilient modulus testing typically applies a load pulse at 1 Hz with a 0.9 second rest. Dynamic modulus at 1 Hz is not the same load pulse as there is no rest period. Kim et al.[3] developed a procedure to calculate resilient modulus from dynamic modulus test data but the procedure is complex and all of the required data was not available. Dynamic modulus values at 5 Hz were over 500,000 psi and at 0.1 Hz averaged less than 200,000 psi, above and below typical reported values for CIR, respectively. A frequency between 0.5 and 1 Hz were selected as giving dynamic modulus values similar to conventional resilient modulus values.

Variations in dynamic modulus at 1 Hz along the projects are shown in Figures 14 and 15 for WR and QO, respectively. Values are lower for WR compared to QO. This can be explained by the presence of uncoated aggregate base in some of the CIR mixture samples for WR. Values are between those typically seen for FDR, 150,000 – 200,000 psi [4] and CIR. Standard deviations were similar between the two projects.

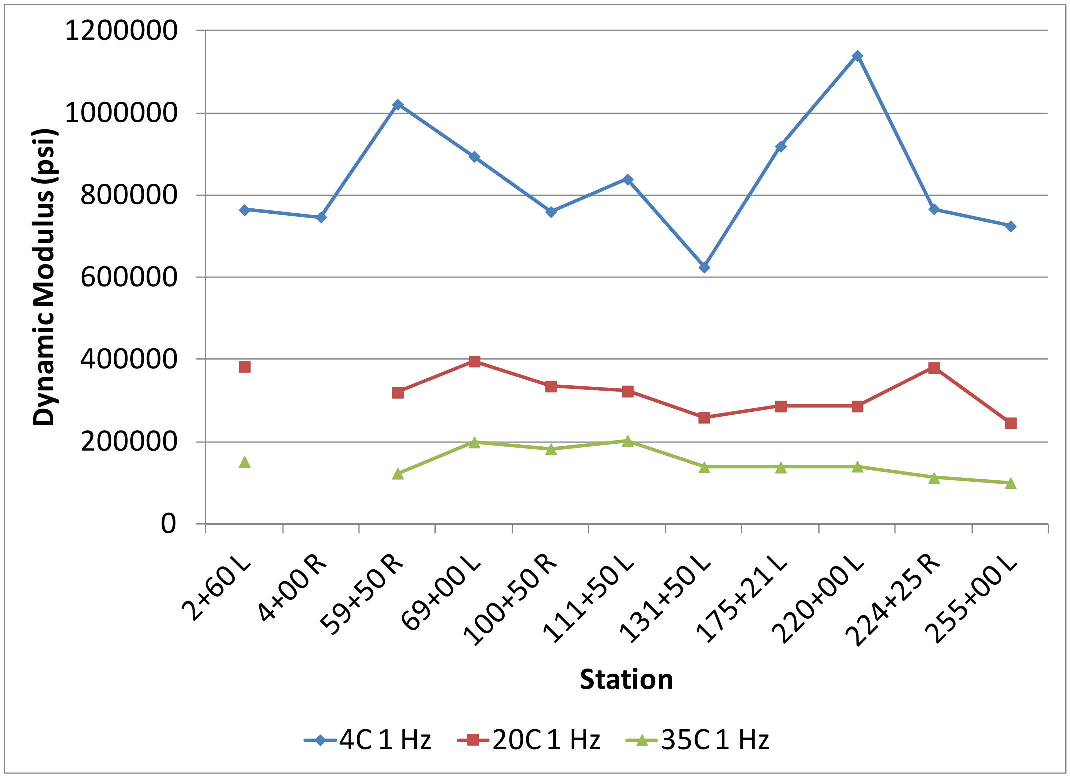

Figure 14. Chart. Dynamic Modulus, 1 Hz., Washington Road.

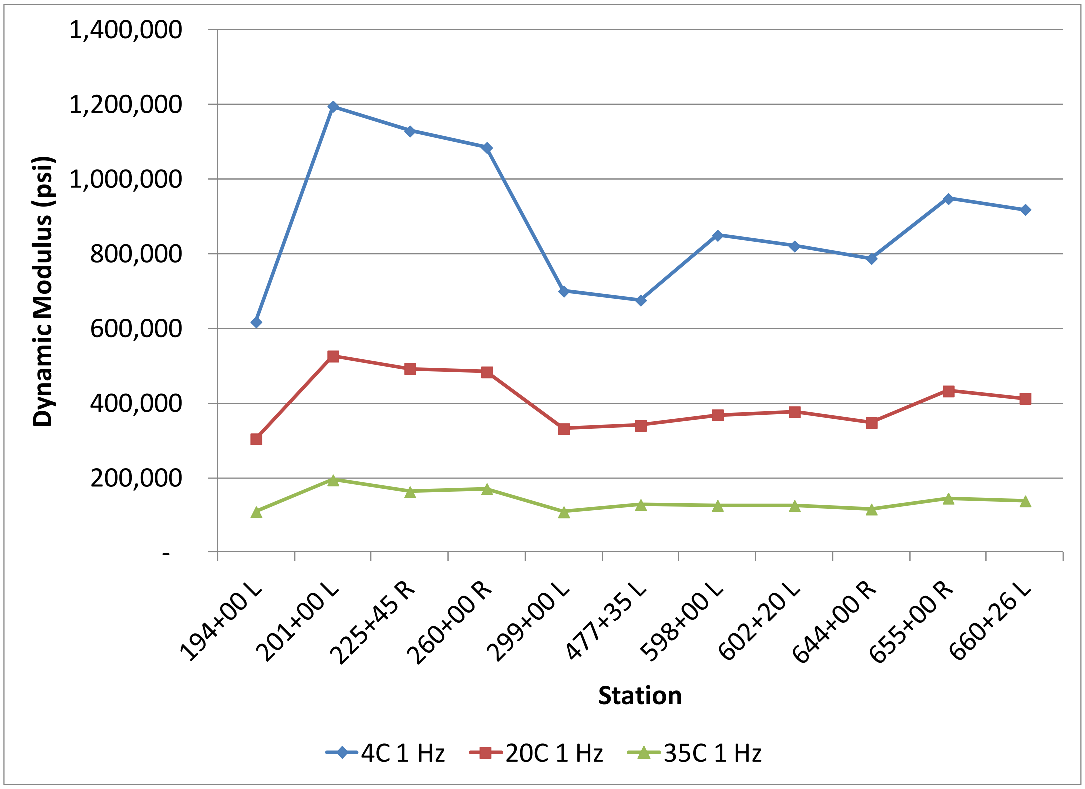

Figure 15. Chart. Dynamic Modulus, 1 Hz., Quincy-Oroville Road.

QA Density Data

Density test results and percent compaction for WR and QO are shown in Tables A-10 and A-18, respectively. Summary statistics were performed on percent compaction to normalize the data as unit weight is partially a function of aggregate specific gravity. For WR, percent compaction was based on a reported bulk specific gravity of 2.039 (127.2 pcf) from the mix design. For QO, percent compaction was based on a unit weight of 125.8 pcf.

Percent compaction along WR is shown in Figure 16. Average percent compaction was 90.7% with a standard deviation of 3.5%. The QO project consisted of an upper portion from sta. 0+00 to approximately 300+00 and a lower section from approximately sta. 450+00 to 700+00. Percent compaction for the upper section is shown in Figure 17 and the lower section in Figure 18. Average percent compaction for the upper section was 95.5% with a standard deviation of 3.3% compared to an average percent compaction of 93.9% and a standard deviation of 1.8% for the lower section. The overall average percent compaction for QO was 94.8% with a standard deviation of 2.9%.

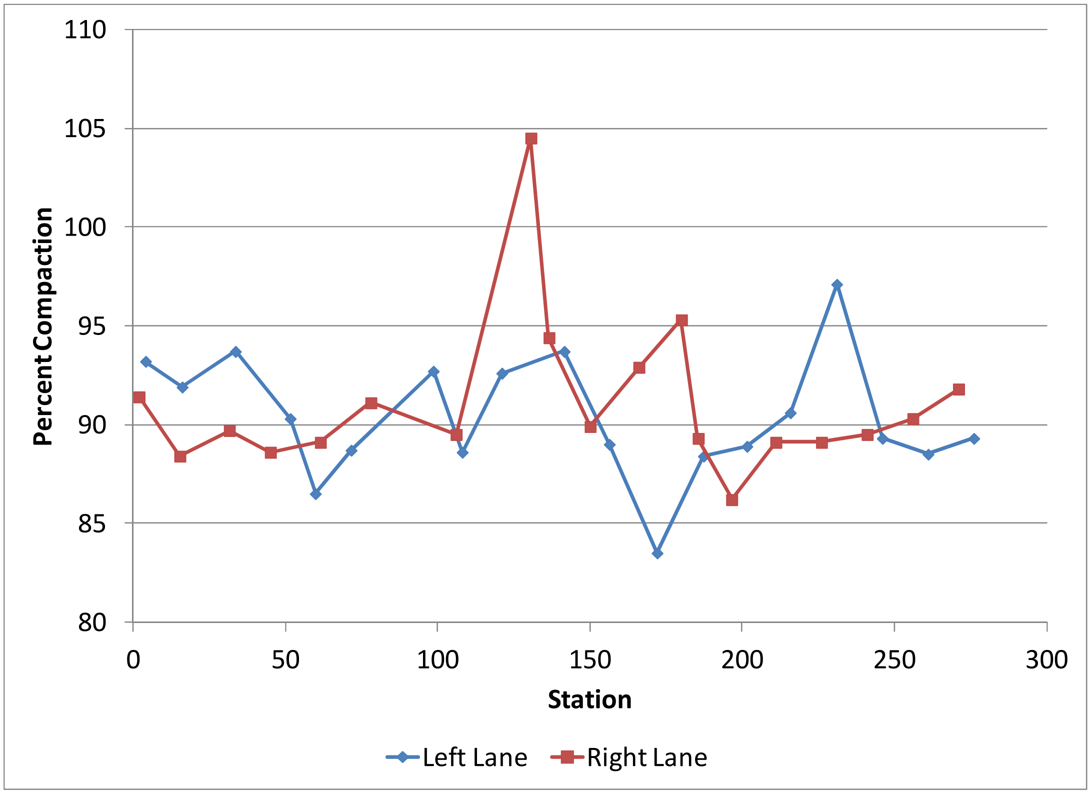

Figure 16. Chart. Percent Compaction, Washington Road.

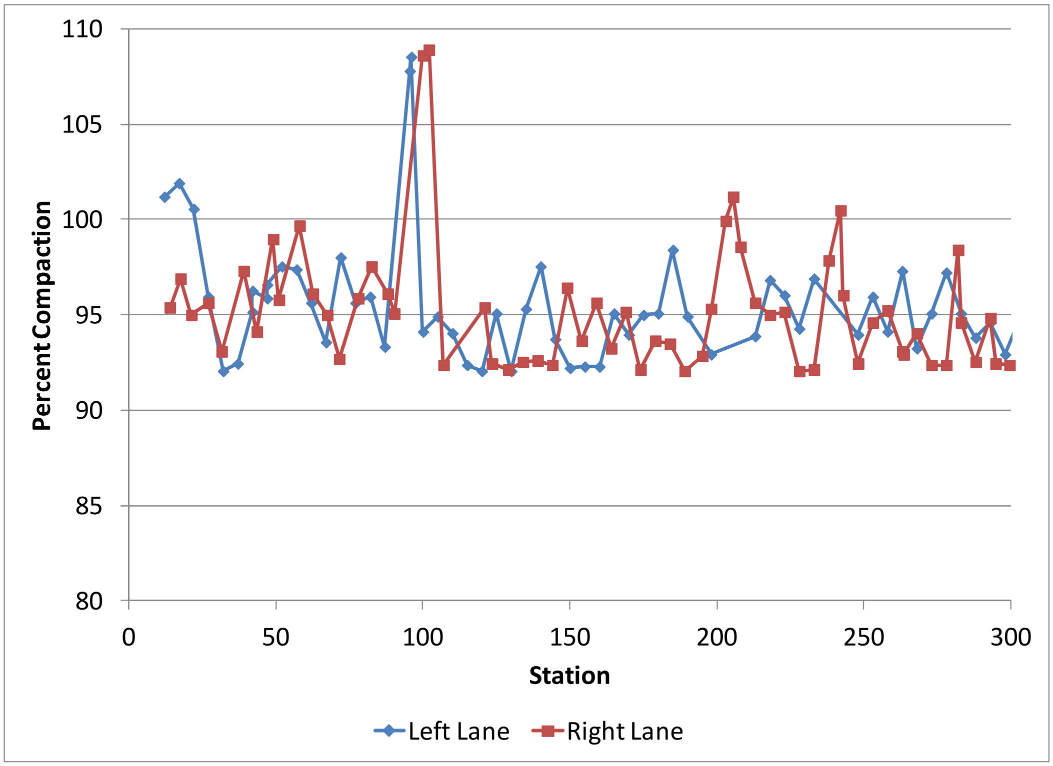

Figure 17. Chart. Percent Compaction, Upper End, Quincy-Oroville Road.

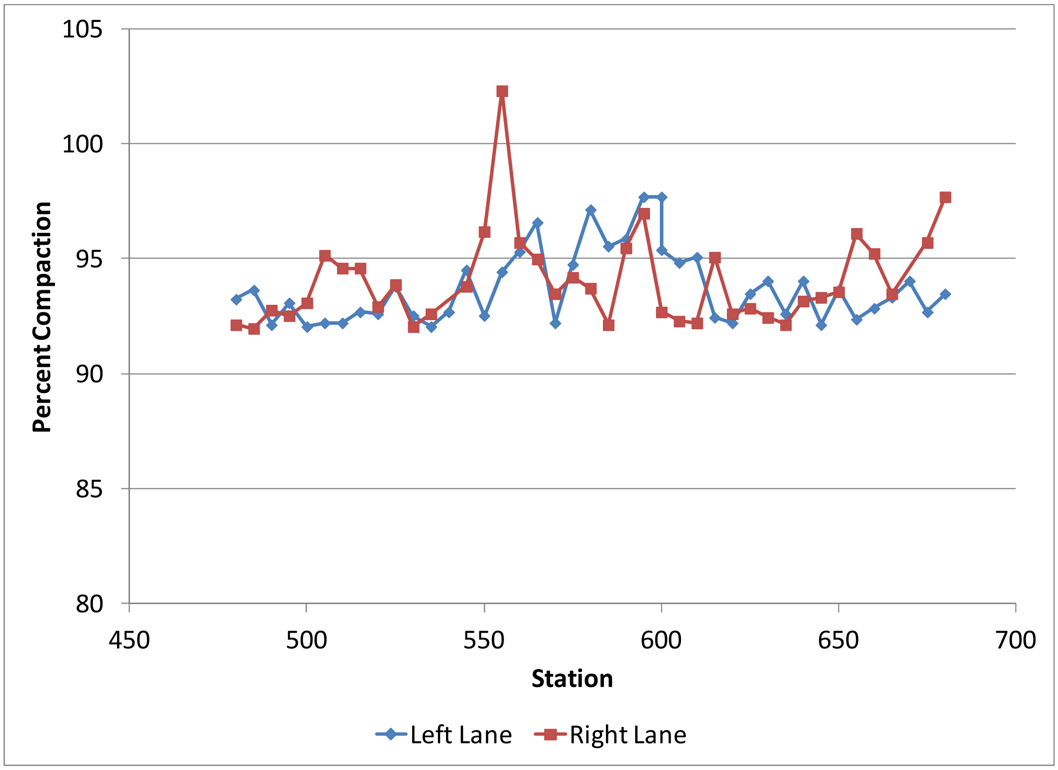

Figure 18. Chart. Percent Compaction, Lower End, Quincy-Oroville Road.

CHAPTER 6 - ANALYSIS OF PROJECT DATA

EFFECT OF MIXTURE AGE ON MIX PROPERTIES

CIR mixture properties can be affected by temperature of the mix when compacted and age of the samples. Due to testing schedules and distance from the field lab and project site, samples for QO were compacted at different temperatures and ages. Samples were obtained in the mornings and afternoons. Samples obtained in the mornings were obtained by the PI and brought to the field lab around noon. They were compacted that afternoon at ambient temperatures, which could be quite warm. Samples obtained in the afternoons were brought to the field lab by the contractor's testing agency after completion of work on the site. Samples often did not arrive until well after 8 pm, too late for testing that day. Afternoon samples were sealed in 5-gallon plastic buckets and left in the field lab overnight. These samples were tested the next morning and were not as warm and were at least 12 hours older than samples obtained in the morning and compacted that afternoon. Some breaking and curing of the emulsion could have occurred for these samples. It is believed that samples from WR were compacted shortly after sampling and would not be as affected by compaction delay. Samples compacted in the mornings (AM) and afternoons (PM) were shown in Table 6. Table 7 shows summary statistics for RAP gradation and asphalt content; and mix properties that could be affected by compaction delay, lab molded voids, AASHTO T 283 results, and dynamic modulus.

Table 7. Summary Statics AM/PM Samples, Quincy-Oroville Road.

	AM Samples			PM Samples		
Property	Average	Standard Deviation	Coef. Variation (%)	Average	Standard Deviation	Coef. Variation (%)
RAP Gradation						
3/4" Sieve	90.2	2.1	2.3	89.5	2.5	2.8
No. 4 Sieve	30.4	4.8	15.8	31	6.2	20.0
No. 30 Sieve	0.4	0.1	32.6	0.8	0.3	37.5
AC (%)	9.40	0.6	6.5	7.7	0.69	9.0
Lab Molded Voids (115 mm Samples)						
VTM (%)	19.8	2.0	10.1	13.3	1.9	14.3
AASHTO T 283						
Voids	20.2	1.7	8.4	14.7	2.4	16.3
Conditioned ITS	64.5	10.7	16.6	55.6	19.3	34.7
Dry ITS	72.1	14.2	19.7	74.5	23.7	31.8
TSR	0.90	0.07	7.8	0.75	0.12	16.0
Dynamic Modulus (20^{o}C)						
1 Hz.	355,173	44,418	12.5	456,332	64,243	14.1

Lab Molded Voids

To determine the affect of compaction delay and temperature on CIR mix properties, the time (AM or PM) samples were compacted was noted for QO samples. Average lab molded voids are shown in Table 7. Average lab molded voids for AM compacted samples was 19.8% compared to 13.3% for PM compacted samples. Standard deviations were 2.0 and 1.9%, respectively. It is apparent that age and temperature has a pronounced affect on lab molded voids. To determine if the difference in means was statistically significant, a t-test was performed on bulk specific gravity (Gmb) and lab molded voids (VTM). The results of the t-test are shown in Table 8. The difference in Gmb and VTM is statistically significant at over a 99% level of significance (α=0.01).

Table 8. T-test on Lab Molded Gmb and VTM.

Variable	Time	Mean	Degrees of Freedom	t-Value	Prob. > |tcr|
Gmb	AM	1.943	22	-9.65	<0.0001
	PM	2.142			
VTM	AM	19.8	22	7.96	<0.0001
	PM	13.3			

RAP Gradation and Asphalt Content

RAP gradation can be affected by temperature of the mat. Higher mat temperatures during milling typically results in finer RAP gradation and colder mat temperatures coarser RAP gradation. However, oversize RAP millings are sent to a crushing unit and coarser RAP means more material is sent to the crusher than with finer RAP millings. Because of this, processed RAP gradations could be similar, regardless of mat temperature.

Table 7 shows average and standard deviations for RAP gradations on the 3/4 inch, No. 4 and No. 30 sieves for AM and PM samples. To determine if the difference in means was statistically significant, a t-test was performed on the percent passing the 3/4 inch, No. 4 and No. 30 sieves. The results are shown in Table 9.

Table 9 shows that the difference in means for the 3/4 inch and No. 4 sieves are not significantly different. There was a statistically significant difference in means on the No. 30 sieve. However, with the low percent passing, less than 1 %, the difference has little engineering significance.

Average and standard deviation for asphalt is shown in Table 7 as well. It is interesting to note that the difference in total asphalt content was statistically significant at a level of significance of over 99% (alpha = 0.01), as shown in Table 9. Contractors often reduce the emulsion content slightly as the pavement heats up to prevent over-asphalting the CIR mix. However, this slight change in emulsion content (generally 0.5%) does not account for the 1.7% difference in asphalt content observed between morning and afternoon samples.

Table 9. T-test on RAP Gradation and Total Asphalt Content.

Variable	Time	Mean	Degrees of Freedom	t-Value	Prob. > |tcr|
3/4" Sieve	AM	90.2	9	0.46	0.6537
	PM	89.5			
No. 4 Sieve	AM	30.4	9	-0.17	0.8678
	PM	31			
No. 30 Sieve	AM	0.43	9	-2.71	0.0241
	PM	0.78			
%AC	AM	9.4	9	4.37	0.0018
	PM	7.7			

Modified AASHTO T 283

Modified AASHTO T 283 samples from QO were compacted at different temperatures and ages (AM and PM samples). To determine the effect on modified AASHTO T 283 results, the results from QO were analyzed by compaction time. The results are shown in Table 10.

Table 10. T-test on AASHTO T 283 Results.

Variable	Time	Mean	Degrees of Freedom	t-Value	Prob. > |tcr|
VTM	AM	20.2	10	4.65	0.0009
	PM	14.6			
Dry ITS	AM	72.1	10	-0.22	0.8331
	PM	74.5			
Cond. ITS	AM	64.5	10	1.03	0.3252
	PM	55.5			
TSR	AM	0.90	10	2.77	0.0196
	PM	0.75			

As shown in Table 10, there was a statistical difference in compacted VTM between samples compacted in the afternoon (PM) and samples that sat sealed overnight and were compacted in the morning (AM), the same as for lab molded voids samples. There was no statistical difference in dry or conditioned indirect tensile strengths between AM samples and PM samples. There was a statistically significant difference in TSR between AM and PM samples at a level of significance exceeding 95% (alpha = 0.05). Samples that sat overnight had a significantly higher TSR than those that were compacted the same day.

It is interesting to note that PM samples, with their lower VTMs, did not have higher dry tensile strengths than AM samples but did have lower conditioned tensile strengths, resulting in lower TSR values. This is not what was expected because higher VTM in HMA generally results in lower TSR values. CIR void contents were two to three times higher than what is seen in HMA and CIR is generally considered an open-graded mix whereas HMA is a dense-graded mix. The effect of these considerably higher voids is not well documented.

Dynamic Modulus

The effect of sample age on dynamic modulus was evaluated using samples tested at 20^{o}C and 1 Hz. The results of the t-test to determine if differences in means were statistically significant are shown in Table 11. As shown in Table 11, there was a statistical difference, at a level of significance exceeding 98% (alpha = 0.02), in VTM and dynamic modulus (E*) values between samples compacted in the afternoon (PM) and samples that sat sealed overnight and were compacted in the morning (AM). Samples compacted in the AM had higher VTMs and lower E* values than those compacted in the PM, the same air void trend as seen for lab molded and TSR samples. Compacted air voids are known to have an effect on dynamic modulus with higher air voids producing lower E* values.

Table 11. T-test on Dynamic Modulus Results.

Variable	Time	Mean	Degrees of Freedom	t-Value	Prob. > |tcr|
VTM	AM	18.0	9	3.58	0.0059
	PM	13.7			
E*	AM	355,173	9	-3.09	0.0130
	PM	456,332			

COMPARISON OF VARIABILITY (STANDARD DEVIATION)

One of the objectives of this study was to evaluate CIR mixture variability. Standard deviations for CIR mix properties evaluated were shown in Table 5 for WR and QO and in Table 7 for QO separated by compaction age. There is little published data on variability of CIR mixtures; therefore, values of HMA mixture variability obtained from published literature were used for comparison. Table 12 shows HMA mix property variability (standard deviations) from the literature. The two Oklahoma projects represent standard deviations from one lot of 5 sublots of 1,000 tons of materials. The remaining data are from numerous mixes and represent overall agency expected values.

Table 12. Reported Standard Deviations.

Material Property	Oklahoma (5)					Hall & Williams (6)			Parker (7)	NCHRP #409 (8)
	NHIY 35-4 (169) 177			STPY-125 C (69)		Quality Level				
	S-2 R	S-3 R	S-4	S-3 R	S-4	High	Medium	Low		
Aggregate Gradation										
3/4" Sieve	2.2	0.9	*	0.7	*					1.4
No. 4 Sieve	2.0	1.8	1.3	1.5	2.7					2.7
No. 30 Sieve	1.5	0.8	0.5	0.8	0.9					1.1
No. 200 Sieve	1.7	0.4	0.3	0.5	0.4					0.9
AC	0.11	0.07	0.07	0.11	0.11	0.18	0.25	0.41	0.25	0.27
Gmm	0.014	0.010	0.005	0.013	0.006					0.011
Gse	0.015	0.012	0.008	0.016	0.005					0.012
Lab Molded Voids	0.36	0.43	0.23	0.58	0.72	0.65	0.77	2.10	0.90	0.90
Compaction	0.59	0.40	0.51	1.32	0.68	0.79	0.96	1.31	1.13	

It must be recognized that HMA variability from the literature represents variability from single HMA mixtures normalized to represent typical HMA mixtures. HMA mixtures are placed with stringent controls on aggregate gradation, asphalt content and mix properties whereas CIR mixtures utilize in-place mixtures. HMA variability would represent a minimum baseline or minimum inherent variability if uniform HMA mixtures were being recycled. However, this is often not the case as many CIR projects consist of different HMA mixes along the project as well as maintenance mixes and patching. Each test property is discussed below.

Maximum Specific Gravity

Maximum specific gravity values were only available from the two ODOT projects [5]. The standard deviation of Gmm for the ODOT projects ranged from a low of 0.005 to a high of 0.014. Maximum specific gravity of CIR is a function of asphalt content, specific gravity of aggregate and gradation of RAP particles. Standard deviation of Gmm was 0.020 and 0.035 using AASHTO T 209 for WR and QO, respectively. The standard deviation was 0.039 for QO using ASTM D6857. Standard deviations were 2.5 to 3 times larger for CIR than HMA.

Asphalt Content

As shown in table 12, the literature [5,6,7,8] reported average standard deviations of 0.25 with Hall [6] reporting as low as 0.18 for high control and 0.41 for low levels of control. The two ODOT projects varied from 0.07 to 0.11. Average standard deviation for asphalt content for the CIR mixes on WR and QO were 0.50 and 1.09 percent, respectively. Standard deviations for CIR are based on total asphalt content, not the amount of emulsion added. WR had more uniform base asphalt contents in the RAP than QO, thus lower overall standard deviations. The standard deviation for WR was twice as high as the reported national averages while QO was four times higher. According to the project report [9], QO had several different mixtures and several large patches along the project.

Aggregate Gradations

Differences in standard deviations between normalized data reported in the literature and standard deviations would be an indication of existing variability of the mixtures cold recycled along each project. The NCHRP report [8] and the two ODOT projects [5] were the only reference found that reported standard deviations of extracted aggregate gradations. These aggregates were not milled as CIR aggregates would be, although the affect on gradation is usually minor, depending upon the hardness of the aggregate. The overall effect of milling would be a finer aggregate gradation. Standard deviations are shown in Table 13.

Standard deviations were 2-3 times larger on the No. 4 and No. 30 sieves for WR and QO compared to reported [8] HMA variability. Standard deviations were similar on the No. 200 sieve and were less that HMA on the 3/4 inch sieve.

Table 13. Reported Gradation Standard Deviation.

Aggregate Gradation	WR	QO	Oklahoma [5] NHIY 35 S-3 R	Oklahoma [5] STPY-125 S-3 R	NCHRP #409 [8]
3/4" Sieve	0.8	0.6	0.9	0.7	1.4
No. 4 Sieve	4.0	6.0	1.8	1.5	2.7
No. 30 Sieve	2.4	3.0	0.8	0.8	1.1
No. 200 Sieve	0.7	1.0	0.4	0.5	0.9

Lab Molded Voids

Standard deviations for lab molded air voids were 2.9 and 3.8 percent for WR and QO, respectively. When QO was separated by time of compaction, standard deviations dropped to 2.0 for AM and 1.9 percent for PM samples. Reported literature (5,6,7,8) HMA lab molded voids ranged from a low of 0.65 for high level of control in Arkansas (6) to an average of approximately 0.9 percent to a high of 2.1 percent for low levels of control. The two ODOT projects (5) showed lower standard deviations, ranging from 0.23 to 0.72 percent. Lab molded air voids for the CIR projects were approximately 2-3 times higher than HMA when adjusted for time or age at compaction. There is also much less temperature control of CIR compared to HMA. It should be noted that lab molded voids are determined from samples compacted with strict temperature control whereas CIR samples are compacted at ambient temperatures.

COMPARISON OF AASHTO T 166 TO AASHTO T 331

To compare AASHTO T 166 to AASHTO T 331 bulk specific gravity, modified AASHTO T 283 samples were used for WR and lab molded samples were used from QO. All samples were compacted to 35 gyrations using the SGC. Sample mass was adjusted to give a compacted height of 95 ± 5 mm for WR and 115 ± 5 mm for QO. Average and standard deviations were shown in Table 5 and the data is shown in Table A-4 for WR and Table A-14 for QO. A 1-way ANOVA was performed on Gmb, by site and with the data pooled, to determine if the difference in means between AASHTO T 166 and AASHTO T 331 was statistically significant. The results of the ANOVA are shown in Table 14.

The ANOVA results indicate a statistically significant difference in test methods, at a confidence limit exceeding 95% (alpha = 0.05). Although not necessary because there were only two levels of the main effect, Duncan's Multiple Range Test was performed on bulk specific gravity to show the rankings of means. The results are shown in Table 15. Means with the same letter are not significantly different at a confidence limit of 95% (alpha = 0.05). There was a statistically significant difference between AASHTO T 166 Gmb and AASHTO T 331 Gmb with AASHTO T 331 producing lower Gmb values in all cases.

Table 14. Results of ANOVA on Gmb, by Project.

Source	Degrees Freedom	Sum of Squares	Mean Square	F Value	Prob. > Fcr
All (Pooled)					
Method	1	0.0447	0.0447	10.12	0.0019
Error	109	0.4818	0.0044		
Total	110	0.5265			
WR					
Method	1	0.0060	0.0060	16.41	0.0001
Error	61	0.0223	0.0004		
Total	62	0.0283			
QO					
Method	1	0.0547	0.0547	5.76	0.0205
Error	46	0.4371	0.0095		
Total	47	0.4918			

Table 15. Duncan's Multiple Range Test on Gmb.

Grouping*	Gmb	n	Test Method
All (Pooled)			
A	2.088	55	T 166
B	2.048	56	T 331
WR			
A	2.085	31	T 166
B	2.065	32	T 331
QO			
A	2.093	24	T 166
B	2.026	24	T 331

*Means with the same letter are not significantly different.

AASHTO LAYER COEFFICIENT

An objective of this study was to evaluate AASHTO structural layer (a) coefficients FLH uses for structural design of CIR mixtures. FLH currently uses a structural layer (a) coefficient of 0.28 for CIR. AASHTO structural layer (a) coefficients are either assigned by an agency based on experience, based on resilient modulus at 20°C, or more typically, based on a combination of the two. If resilient modulus values are used, *Figure 2.9 Variation in a_2 for Bituminous Treated*

Bases with Base Strength Parameter [9] could be used. However, CIR mixtures are not aggregate base coated with asphalt emulsion, they are RAP particles coated with emulsion. Although not dense graded HMA, some argue that CIR has more load carrying capacity than Figure 2.9 indicates. Therefore, *Figure 2.5 for Dense Graded Asphalt Concrete* [9] has been consulted as well.

Resilient modulus is no longer used to characterize HMA and CIR; dynamic modulus is now recommended. Resilient modulus testing typically applies a load pulse at 1 Hz with a 0.9 second rest. Dynamic modulus at 1 Hz is not the same load pulse as there is no rest period. Dynamic modulus values at 1, 0.5 and 0.1 Hz at 20^{o}C were selected for analysis as they provided values similar to typical resilient modulus values and were used with figures 2.5 and 2.9 of the 1993 AASHTO Design Guide [9] to determine structural layer (a) coefficients. Dynamic modulus values and corresponding structural layer (a) coefficients are shown in Table 16. The *1993 AASHTO Design Guide* is reliability based and requires the use of average input parameters [9]; therefore, structural layer coefficients are calculated using average dynamic modulus values. Use of other than average values with the 1993 Design Guide, such as the 75th or 85th percentile, would result in designs exceeding the selected reliability.

Table 16. AASHTO a Coefficients from Dynamic Modulus Results.

Route	E* (psi)	Fig 2.5 a_1 Coefficient	Fig 2.9 a_2 Coefficient
	1 Hz		
WR	321,950	0.38	0.29
QO	401,155	0.41	0.32
QO-AM	355,173	0.39	0.30
QO-PM	456,332	0.43	0.34
	0.5 Hz		
WR	294,619	0.36	0.28
QO	358,154	0.39	0.30
QO-AM	326,750	0.38	0.29
QO-PM	387,013	0.41	0.31
	0.1 Hz		
WR	188,692	0.29	0.22
QO	237,262	0.33	0.25
QO-AM	209,870	0.31	0.23
QO-PM	270,133	0.35	0.27

Average AASHTO a_1 coefficients ranged from a high 0.43 to a low of 0.38 for 1 Hz loading rate approaching dense-graded HMA. However, a study by NCAT[11] reported calculated a_1 coefficients of greater than 0.50 based on test track results. However, these values are higher than values reported in the literature[1] for CIR that ranged from 0.25 to 0.35. Average AASHTO a_1 coefficients ranged from 0.41 to 0.36 for 0.5 Hz loading rate and from 0.35 to 0.29 for 0.1 Hz loading rate. Average AASHTO a_1 coefficients for WR and QO were 0.38 and 0.41 at 1 Hz, 0.36 and 0.39 at 0.5 Hz and 0.29 and 0.33 at 0.1 Hz, respectively. When QO was separated by age of samples when compacted, a_1 coefficients increased to 0.43, 0.41 and 0.35 at 1, 0.5 and 0.1 Hz, respectively, for samples compacted soon after sampling (PM samples).

Average AASHTO a_2 coefficients ranged from a high 0.34 to a low of 0.29 for 1 Hz loading rate, from 0.31 to a low of 0.28 for 0.5 Hz loading rate and from 0.27 to 0.22 for 0.1 Hz loading rate. Average AASHTO a_2 coefficients for WR and QO were 0.29 and 0.32 at 1 Hz, 0.28 and 0.30 at 0.5 Hz and 0.22 and 0.25 at 0.1 Hz, respectively. When QO was separated by age of samples when compacted, a_2 coefficients increased to 0.34, 0.31 and 0.27 at 1, 0.5 and 0.1 Hz, respectively, for samples compacted soon after sampling (PM samples).

In some areas, WR samples contained aggregate base and would be considered FDR by ARRA [1]. FDR generally has lower load bearing capacity due to the increased amount of fines and uncoated materials associated with incorporation of aggregate base. Although not the case throughout the project, the aggregate base would lower the dynamic modulus and WR could be considered on the low end of expected load bearing capacity of CIR projects.

Based on the results shown in Table 16, a structural layer a coefficient of 0.32-0.34 appears reasonable for most CIR projects. The FLH a coefficient of 0.28 for CIR appears slightly conservative. The PM compacted data from QO supports a structural layer coefficient in the 0.34 to 0.36 range. There are agencies that use structural layer coefficients as high as 0.35 for CIR. Based on the data obtained from QO, a value of 0.34 for a structural layer coefficient for engineered emulsions could be easily supported. The use of additives such as lime or cement could increase the structural layer coefficient to over 0.35. However, more study would be necessary before a recommend value over 0.35 for CIR could be made.

CHAPTER 7 - MIX DESIGN RECOMMENDATIONS

BACKGROUND

A mix design is a formulation that defines percent and grade of recycling agent, recommended water content, and additives for the planned CIR or FDR mixture. The data is used to develop mix properties that will ensure that the mix will exhibit adequate initial strength, resistance to moisture-induced damage, resistance to thermal cracking and resistance to raveling. A formal mix design and a mix design report documenting the design formulation introduces additional quality control that helps to ensure that the pavement will meet desired specifications and performance expectations. It is recommended for all CIR and FDR applications. Recommended mix design procedures are described and presented in the form of a draft FLH test method in Appendix B and C for CIR and FDR, respectively.

COMPONENT MATERIALS

Recycling Agent (Asphalt Emulsion)

The proposed mix designs will assist the designer with selection of the appropriate amount of recycling agent and additives. However, more than one recycling agent (asphalt emulsion) could meet the design requirements and many different types of recycling agents are available for use with CIR and FDR. Asphalt emulsions used for CIR and FDR are similar; however, specific formulations can be different due to different requirements for coating, mixing, curing and breaking conditions. The most popular recycling agents are engineered emulsions, which are typically modified cationic slow set emulsions, polymer and non-polymer high float emulsions and cationic slow set emulsions. The following is a brief discussion of the properties of the commonly available asphalt emulsions used in CIR and FDR.

Engineered Emulsions

Emulsions can be “engineered” to provide selective properties for a given project. Properties that are engineered include mixing and coating ability, breaking times, curing times, moisture resistance, softening ability of the emulsion and stiffness properties of the residual binder. Properties are adjusted by numerous techniques including varying the residual binder content, stiffness of the residual binder, polymer modification, pH, and adding a fluxing agent, to name a few. There are limits, however, as to how much modification can be accomplished with a given grade or classification of recycling agent.

High Float Emulsions

High float emulsions are often selected for their ability to soften old aged binder and their ability to coat coarser aggregates. High float emulsions are manufactured with a small amount of fluxing agent to promote coating and consequently, soften the old aged binder. Coating of dense graded material with high float emulsions tends to be selective with the smaller particles coated with a thick film of asphalt while the larger particles are partially coated [12].

Cationic Slow Set Emulsions

Slow setting emulsions have long workability times to ensure good mixing with dense-graded materials. Cationic slow set emulsions contain little to no solvents and are often preferred because solvents, if trapped in a CIR or FDR mixture, can lead to performance issues. Cationic slow set emulsions tend to coat more of the fine portion of the mix with a more uniform, thinner film thickness and, as for all bituminous additives, the coated fine material acts as a mortar that binds the material together. Pozzolonic material, lime or cement, can be added to cationic materials to act as a catalyst to accelerate the buildup of cohesion, increasing initial strength and moisture resistance, and reducing curing time[(12)].

Polymer Modification

Polymer modification can enhance positive characteristics of emulsions resulting in higher cohesion of the binder and more rapid strength gain. Other advantages are increased resistance to moisture damage, reduced raveling and reduced cracking. Polymer modification allows the use of softer residual binders that are better able to soften the aged binder in the RAP [(13)] and will increase resistance to thermal cracking.

Recycling Additives

The most common recycling additives are lime and portland cement. Additional aggregates are sometimes used as well.

Lime & Portland Cement

Due to the higher in-place voids, CIR and FDR mixtures can be susceptible to moisture-induced damage (stripping). Because of this fact, some agencies require that either lime or an anti-strip agent be incorporated into the mix. Other agencies require that these supplemental additives be added only when required by the mix design.

Mixes that fail the moisture susceptibility test generally benefit greatly from the addition of 1.0-1.5 % hydrated lime or 0.25 - 0.5% portland cement. Lime or portland cement added to cationic materials act as a catalyst accelerating the buildup of cohesion, and increased cohesion improves moisture resistance [(12)].

Use of lime and portland cement, or polymer modification, could also assist in improving dry tensile strength (or Marshal stability) of the mix. Benefits of lime are well documented, including improved resistance to moisture induced damage, rapid strength gain, improved resistance to permanent deformation and improved stability [(14,15,16,17,18)].

Care should be taken when using additives (portland cement and lime) because they will affect the mixture breaking and curing times. Portland cement should be added in limited quantities to prevent the mixture from exhibiting brittle characteristics. A minimum ratio of emulsion residual asphalt content to cement of 3.0 to 1.0 should be maintained to prevent brittle behavior. Additives should be evaluated in the mix design and then with test strips in the field before final inclusion in the mix.

Add-Stone

Recycled asphalt pavement (RAP) milled from the existing surface comprises the primary aggregate in CIR mixes. RAP and base course materials are the primary aggregates in FDR mixes. Uncoated aggregates (add-stone) are occasionally added to CIR and FDR mixtures to improve aggregate gradation. The addition of add-stone is a useful practice if the mix design shows a quantifiable improvement in measured mix properties. CIR and FDR mixtures may be designed with or without additional aggregate (add-stone) as long as the mixture meets the mix design requirements.

CIR MIX DESIGNS

Background

In 2001, ARRA published the Basic Asphalt Recycling Manual (BARM), which summarized the available mix design procedures and provided recommended mix design steps for CIR mixtures. Two currently popular methods are the procedures developed by Road Science, LLC and their predecessors, and a procedure recently adopted by the Pacific Coast Conference on Asphalt Specifications (PCCAS) [19]. Both procedures follow the recommended steps outlined in the BARM [1].

Many State DOTs have adopted the Road Science mix design procedure, which uses Superpave principals including specimen compaction using the Superpave Gyratory Compactor (SGC). The procedure also includes testing samples for resistance to thermal cracking using AASHTO T 322, and resistance to raveling using ASTM D7196. AASHTO T 322 is a complex test that few agencies have the capability to perform.

The PCCAS procedure is a simpler version of the Road Science procedure and allows the use of 75-blow Marshall compaction as well as SGC compaction for those agencies that cannot compact 100 mm diameter (4-inch) SGC samples. The PCCAS procedure removes the thermal cracking test requirement (AASHTO T 322) and replaces it with a requirement that the asphalt binder used to make the asphalt emulsion meet the Bending Beam requirements of *AASHTO M 320 Standard Specification for Performance-Graded Asphalt Binder*, the Performance Graded (PG) Asphalt Binder for the project location. This modification makes the procedure much less costly and greatly increases the number of agencies/firms that can perform the mix design. Both procedures utilize the raveling test (ASTM D7196).

ARRA has recommended a CIR mix design procedure and it is available on their web page.[20] The procedure has a basic mix design procedure and options for a more robust mix design which includes provisions for testing binder recovered from pavement cores for penetration and viscosity, performing the raveling tests at different compaction and curing temperatures and determining TSR or retained Marshall stability on samples compacted at 104^{o}F as well as the conventional 77^{o}F.
The Colorado DOT has a new mix design procedure for CIR, *Standard Specification 406 Cold Bituminous Pavement Recycle (Special)*, which uses the Hamburg Rut Tester and appears to be the only procedure that includes loaded wheel rut testing. Use of the Hamburg Rut tester is an

interesting development but it is not recommended for adoption at this time due to a lack of data to support a threshold or specification value.

Procedures

Establishing a mix design for a CIR project requires collection of field samples of the existing pavement to be recycled and subsequent laboratory (mix) testing to establish a target formulation of the materials (asphalt emulsion, water, RAP, add-stone and additives, if needed) that will be used during construction.

Sample Collection

Coring the pavement to be recycled is the preferred method for collection of representative samples of the target pavement. This is undertaken to establish whether properties of the pavement are consistent along its length, width, and depth and to obtain materials for the mix design. Cores should be obtained from edge to edge in both lanes to ensure sufficient thickness is present. Obtain samples near the centerline, between wheel paths, at the pavement edge and in the paved shoulders if they are to be recycled.

A minimum of three cores per lane mile should be obtained to check for pavement consistency with additional cores where visual differences in the pavement are noticed. These cores are also used for determination of asphalt content and gradation analysis of the existing pavement (to the specified milling depth). If cores show significant differences between areas, such as different type or thickness of layers between cores, then separate mix designs are recommended for each of these pavement segments.

More material, above the minimum coring rate, may be required for mix design. Approximately 350 pounds of usable pavement (RAP) is required if 150 mm diameter samples are used in the mix design and 200 lbs if 100 mm diameter samples are used. Usable RAP refers to RAP from the section to be recycled only. Pavement cores can be either 6 inches in diameter (recommended) or 4 inches in diameter. Materials for the mix design should be separated from the rest of the core by collecting (sawing off) the material to be recycled. The collected samples are then processed using a laboratory jaw crusher or other technique that will yield a material similar to material manufactured during actual milling operations.

Batching Samples

RAP gradation will vary throughout the project due to daily temperature changes of the milled pavement and due to changes in mix composition of the pavement and normal mixture variation. To account for these changes, many mix design procedures recommend performing two mix designs on RAP samples batched to a “coarse” and “medium” gradation. The Road Science mix design procedures originally had three gradations, coarse, medium and fine, and two of three were selected for the design. It appears that the fine gradation was rarely used and newer mix design methods dropped the fine gradation leaving the coarse and medium. However, these coarse and medium gradations appear to represent the extremes of RAP milling gradations that are typically encountered and could be considered coarse and fine gradations.

The coarse and medium gradations from the Road Science and PCCAS procedures are shown in Table 17 along with the suggested FLH CIR RAP gradation used for the mix designs. As shown in Figures 19 and 20, the suggested FLH gradation falls between the Road Science coarse and medium gradations. The PCCAS gradations fall within the existing FLH gradation, as shown in Figure 21.

Table 17. Cold Recycling Gradation Requirements.

Sieve Size	Road Science		PCCAS		Suggested FLH Gradation
	Medium	Coarse	Medium	Coarse	
	Percent Passing				
1.5 inch					100
1.25 inch		100			
1 inch	100	85-100	100	100	90-100
3/4 inch	85-96	75-92	95 ± 2	85 ± 2	85-95
1/2 inch					75-85
No. 4	40-55	30-45	50 ± 2	40 ± 2	35-50
No. 16					5-16
No. 30	4-14	1-7	10 ± 2	5 ± 2	
No. 200	0.6-3.0	0.0-3.0	0.8 ± 0.3	0.3 ± 0.3	0-7

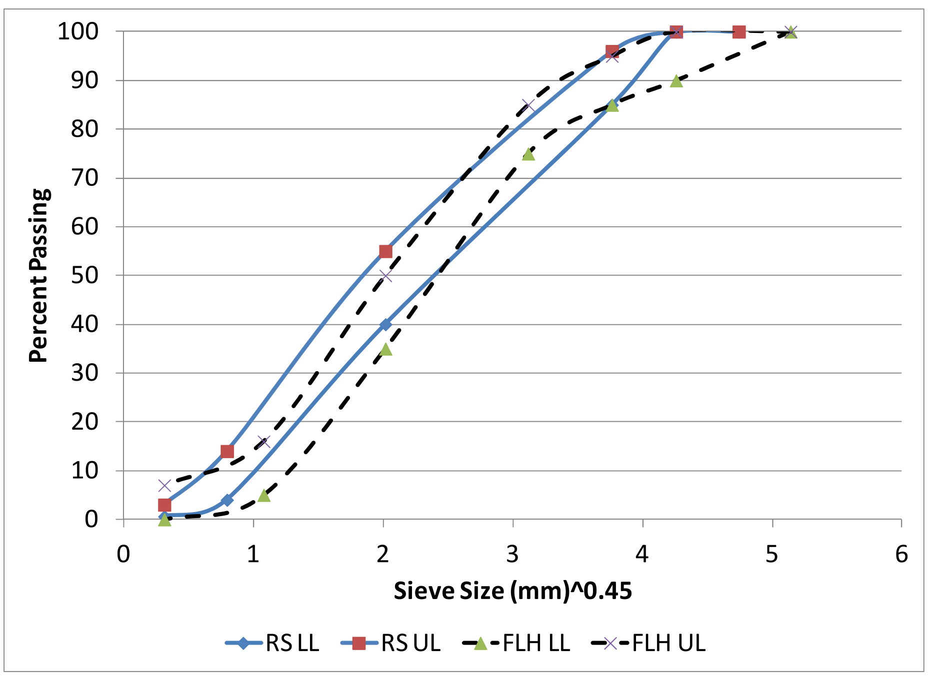

Figure 19. Plot. FLH vs. Road Science Medium CIR Mix Design Gradation Bands.

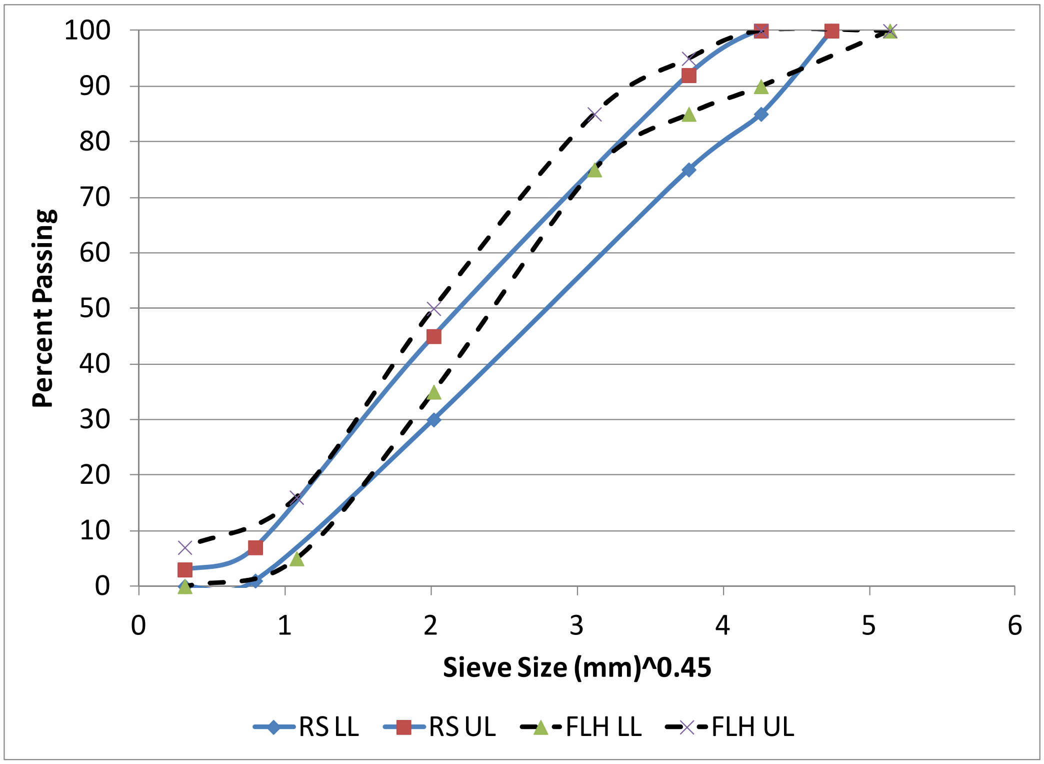

Figure 20. Plot. FLH vs. Road Science Coarse CIR Mix Design Gradation Bands.

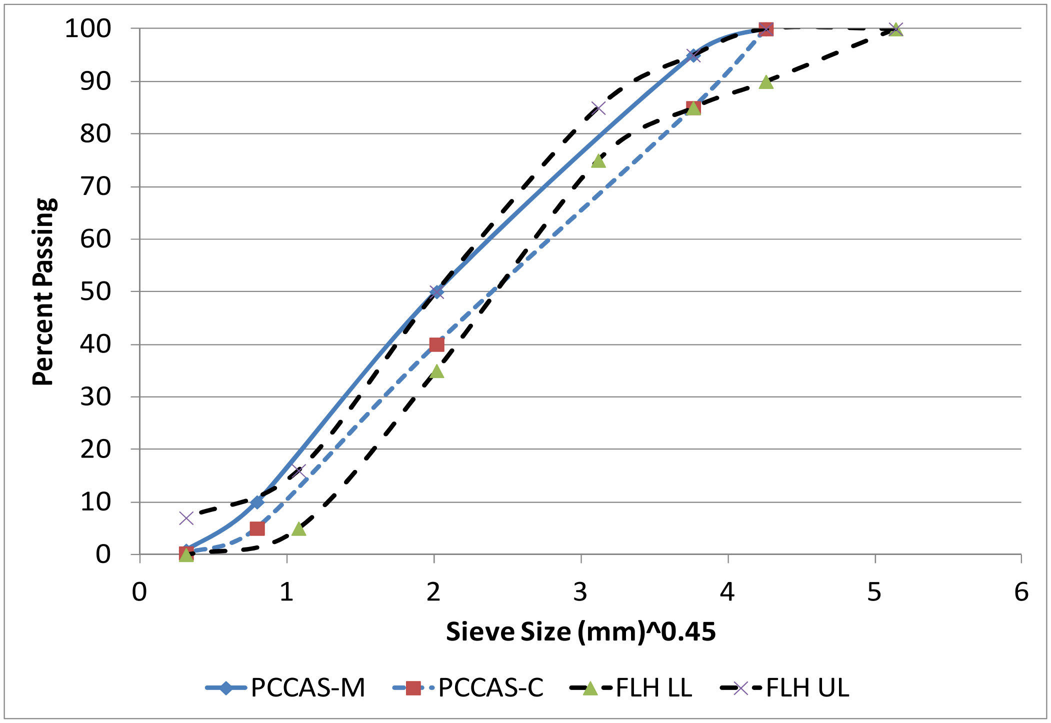

Figure 21. Plot. FLH CIR Mix Design Gradation Bands vs. PCCAS Coarse and Medium Gradations.

Table 18 shows average CIR RAP gradations from WR and QO along with the suggested FLH gradation. There is a difference in gradations. The suggested FLH gradation is for RAP or millings, prior to mixing with emulsion. The WR and QO gradations were determined on samples obtained after addition of emulsion. The addition of emulsion will bind up many of the fines resulting in a coarser gradation. As shown in Figure 22, QO is considerably coarser than the mix design gradation used and the suggested FLH gradation, as expected. WR is slightly finer than the mix design gradation and within the suggested FLH gradation. The addition of the relatively clean aggregate base into the WR mix resulted in a considerably finer gradation than what is typically seen for CIR and for QO. The finer gradation of WR could help account for some of the raveling reported on WR.

The suggested FLH RAP gradation should be sufficient and can be used for CIR mix designs. As an option, mix designs can be performed on both the coarse and medium gradations from either agency listed in Table 17. This provides an indication of the range of expected water and emulsion contents that might be needed in the field. However, one mix design gradation should be sufficient in most instances as long as inspectors realize that changes to the mix design emulsion content will be necessary if RAP gradation changes significantly.

Table 18. Project Gradations.

Sieve Size	WR		Q-O		Suggested FLH Gradation
	Mix Design	Field	Mix Design	Field	
	Percent Passing				
1.5 inch	100	100	100	100	100
1 inch	95	98	95	96	90-100
3/4 inch	88	93	88	90	85-95
1/2 inch	75	81	75	75	75-85
3/8 inch	66	71	66	62	
No. 4	42	44	42	31	35-50
No. 8	25	28	25	11	
No. 16	12	16	12	2.6	5-16
No. 30	6.3	8.0	6.3	0.6	
No. 50	2.4	3.2	2.4	0.2	
No. 100	1.1	1.3	1.1	0.1	
No. 200	0.3	0.6	0.3	0.1	0-7

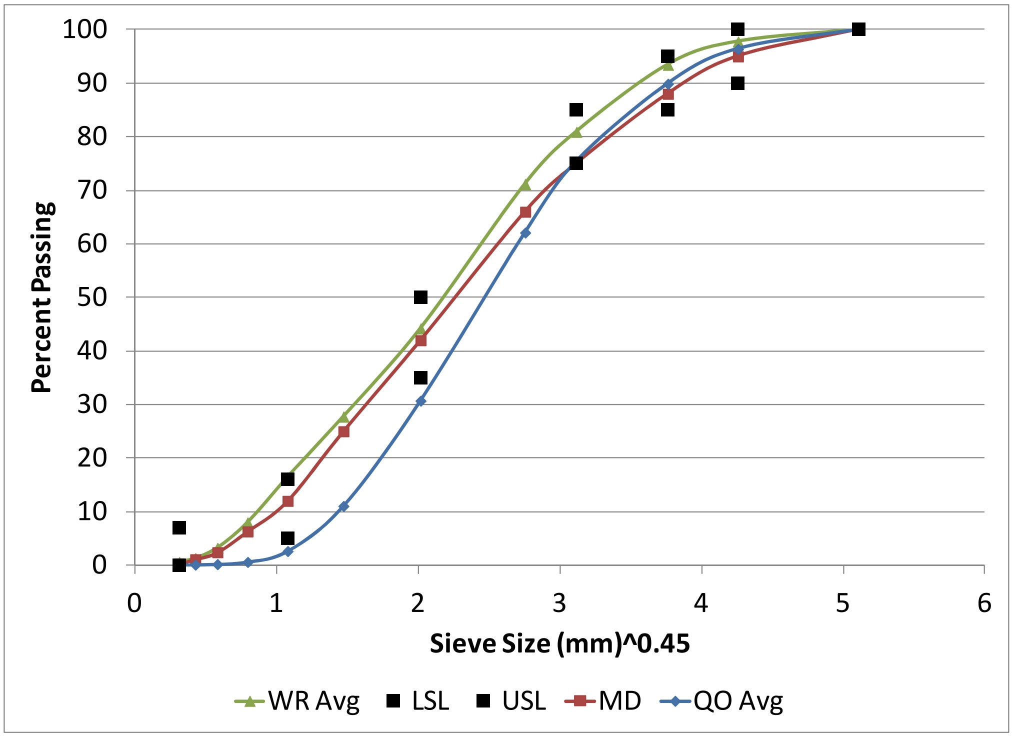

Figure 22. Plot. Field Gradations vs. Mix Design and Specification Limits.

Mixing

Specimen size should be selected to produce a 95 ± 5 mm tall specimen when compacting 150-mm (6-inch) diameter samples or to produce a 63.5 ± 2.5 mm (2.5 ± 0.1 in.) tall specimen when compacting 100-mm (4-inch) diameter specimens. Plus 1-inch material should be excluded from 100-mm (4-inch) molds and replaced with an equal amount of minus 1-inch material.

A minimum of three emulsion contents, in either 0.5% or 1.0% increments which bracket the estimated recommended emulsion content, are selected. Emulsion contents typically cover a range between 1.0% and 4.0% by dry weight of RAP. Six samples are compacted at each emulsion content, three for dry tensile strength or Marshall stability testing on cured samples and three for conditioned tensile strength or conditioned Marshall stability on cured samples.

Two specimens are prepared for theoretical maximum specific gravity according to AASHTO T 209 or ASTM D6857. Loose RAP mixtures are cured as described below under Curing. RAP agglomerates that will not easily reduce with a flexible spatula should not be broken apart. Both specimens are tested at the highest emulsion content in the design and the maximum specific gravity back calculated for the lower emulsion contents.

Mixing of test specimens should be performed manually, with a mechanical bucket mixer or a laboratory sized pugmill mixer or combination of the two. For larger specimens, 150 mm diameter, mechanical or pugmill mixing is preferred. Samples are mixed thoroughly with the amount of water that is expected to be added at the milling head, typically 1.5 to 2.5 percent. If any additives are in the mixture, they are usually introduced next but should be introduced in a similar manner that they will be added during field production. Emulsion is then added and the sample mixed. One specimen is mixed at a time and mixing time with emulsion should not exceed 60 seconds.

Compaction

Compactive Effort: Two methods of compacting mix design test specimens have been used, Marshall compaction and Superpave gyratory compaction (SGC). Marshall compaction can be either 50 or 75 blows per side with most modern CIR Marshall mix designs employing 75-blow compaction. Compactive effort for SGC compaction was evaluated by Cross [(21)] and 30 to 35 gyrations were recommended. Most published mix design procedure us 30 gyrations. Compaction is in accordance with AASHTO T 245 for Marshall and AASHTO T 312 for SGC with the exception that samples are typically compacted at ambient temperatures.

Mix designs for WR were made using both 75-blow Marshall and SGC compaction using 35 gyrations. QO used 35-gyration SGC compaction. Comparisons between mix design compacted voids and lab molded field voids are shown in Figure 23. Mix design voids between 75-blow Marshall and SGC compaction were similar for WR. Field lab molded voids were approximately 1.5% higher than mix design voids at the same compactive effort. WR could have had aggregate base incorporated in the mix where the mix design samples shown in Figure 23 did not. Mix design voids for QO were approximately 2 percent higher than field lab molded voids. QO samples were finer than mix design samples, possibly accounting for the higher density and lower voids of the field samples.

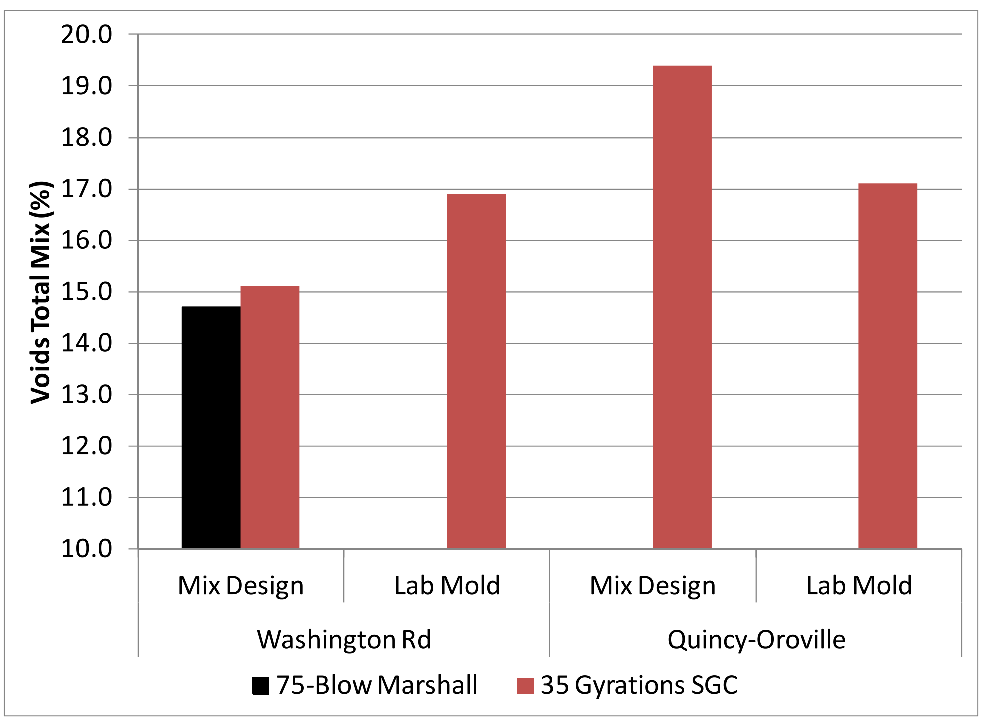

Figure 23. Bar Chart. Comparison of Mix Design and Lab Molded Voids.

For a volumetric based mix design the laboratory compactive effort should produce a sample with the same density as the mix would achieve in the field. CIR mix designs are not completely volumetric based, but the concept is still considered valid. Nine cores from the compacted CIR mat were available from sta. 106+00 to 251+10 on QO. The average dry unit weight, as calculated from the results shown in Table A-19, was 124.4 pcf. The mix design unit weight at optimum emulsion content was 122.1 pcf. Lab molded unit weights were available from four samples in the vicinity of where the cores were obtained. Three of these samples were compacted in the PM and one in the AM after sitting overnight. The average lab molded unit weights for all four locations, AM samples, average PM samples and an average of the AM and average PM samples are shown in Table 19 along with the average core and mix design unit weights.

Table 19. Unit Weight Results from Quincy-Oroville Road.

		Lab Molds				
	Cores	Avg. All	AM	Avg. PM	Avg. AM & PM	Mix Design
Unit Weight (pcf)	124.4	130.4	117.2	134.8	126.0	122.1

Cores and lab molded samples are compacted at in situ temperatures not necessarily at room temperature as for mix design samples. The average of all lab molds and the PM lab molds had higher unit weights than the cores. The mix design and average of the AM sample and PM samples had similar unit weights to the cores. The data indicate that temperature and age have an impact on lab molded unit weights. However, there is insufficient data to draw a definitive conclusion on mix design compactive effort. Either 35 gyrations, or the more customary 30 gyrations, in the SGC appear as reasonable mix design compactive efforts.

Compaction Temperature: At high pavement temperatures the CIR mat will compact denser and not reducing the emulsion content could result in an over asphalted and unstable mix, especially for higher trafficked pavements. Therefore, it is recommended that contractors be allowed to adjust the emulsion content ± 0.5% without requiring a new mix design. Many agencies are reluctant to allow a contractor to reduce emulsion content as the pavement heats up during the day. To combat this, newer mix designs are specifying compaction at 104°F at 0.5% less emulsion that optimum from the mix design. The purpose of this increased temperature and reduced emulsion content is to verify that there will be no adverse effect on CIR density, strength or moisture sensitivity.

Before mix designs became relatively standardized, mix designs for projects that were planned for construction during high summertime temperatures (nearing 100°F) were often performed by compacting samples at 104°F. Compaction at this elevated temperature reportedly better matched field asphalt emulsion temperatures and tested mix properties at more realistic conditions. Testing at 77°F is closer to a worst case scenario. Rather than requiring compaction at two test temperatures, the mix design procedure could call for compaction at 77°F with the statement added "or as directed by the engineer" for instances when compaction at 104°F would better simulate field conditions. If this is not followed, then agency personnel would need to allow adjustments (lowering) of the emulsion content when conditions warrant, such as high pavement temperatures.

Curing

Samples are cured before testing. The only exception is for raveling test samples. Samples are cured in a 140°F (60°C) forced draft oven to a constant mass but no more than 48 hours and no less than 16 hours. Constant mass is defined as no more than 0.05% change in mass in 2 hours. Samples should be placed in small specimen containers to account for material loss from specimens. After curing, samples should cool at ambient temperatures for 18 ± 6 hours before testing.

Laboratory Testing

Table 20 presents a list of laboratory tests to be performed as part of the mix design. The design emulsion content is the emulsion content(s) such that the cold mix requirements listed in Table 20 are met for the design gradation.

Bulk Specific Gravity of Compacted Samples

Bulk specific gravity of compacted samples can be determined using either AASHTO T 166 Method A or AASHTO T 331. Due to the high air voids, AASHTO T 331 will most likely be required. Bulk specific gravity is necessary to determine air void contents for use in determining percent saturation of retained Marshall stability or TSR samples. Calculations for AASHTO T 283 percent saturation use SSD weights and recommends saturation levels of 70-80 percent saturation. If AASHTO T 331 is used, and due to the inherent high void content of CIR mixtures, it might not be possible to saturate samples to the recommended 70-80 percent levels without damaging samples. The difference between AASHTO T 331 and AASHTO T 166 specific gravity are void spaces that are too large to hold water during SSD determination. Results in Chapter 5 indicated AASHTO T 331 voids were, on average, 2.1 percent higher than AASHTO T 166 voids. Using AASHTO T 331 based air voids could require the partial filling of void spaces that cannot hold water, making obtaining 70-80% saturation difficult is not impossible in some cases. Therefore, it is recommended that AASHTO T 166, and AASHTO T 331 when required, be used to determine specific gravity and subsequent air void calculations for CIR mixtures and that 55-75 percent saturation levels be required for moisture sensitivity testing.

Maximum Theoretical Specific Gravity

Maximum theoretical specific gravity can be determined in accordance with AASHTO T 209. However, *Section 11 Supplemental Procedure for Mixtures Containing Porous Aggregate,* or the dry-back procedure, is often necessary due to uncoated RAP particles. An alternative method, when the dry-back procedure is not practical, is *ASTM D6857 Standard Test Method for Maximum Specific Gravity and Density of Bituminous Paving Mixtures Using Automatic Vacuum Sealing Method.* ASTM D6857 does not require the dry-back procedure but results will be affected, to a lesser extent, than AASHTO T 209 without the dry-back procedure. Use of either method should be allowed.

Table 20. CIR Laboratory Mix Design Tests.

Design Parameters	Objective	Requirement
Gradation of Design Reclaimed Asphalt Pavement (RAP), AASHTO T 27	To ensure that the mix design meets the gradation specification.	Suggested FLH Gradation or Coarse and Medium Gradations of Table 2
Asphalt Content of RAP, AASHTO T 308	General information only.	Report[1]
Bulk Specific Gravity of Compacted Samples, AASHTO T 166 Method A or AASHTO T 331	To determine air voids of compacted specimens.	Report[1]
Maximum Theoretical Specific Gravity, AASHTO T 209 or ASTM D6857	To determine air voids of compacted specimens.	Report[1]
Air Voids of Compacted and Cured Specimens, AASHTO T 269	Information only: Typical values are 8-16% and higher. CIR should not be designed, nor the asphalt emulsion content altered, to meet a specific air void content.	Report[1]
Marshall Stability, Cured Specimen, AASHTO T 245, 104°F (40°C)	To evaluate cured strength.	1,250 lb. minimum
Marshall Retained Stability, AASHTO T 245, 104°F (40°C) Based on Moisture Conditioning on Cured Specimen	To evaluate resistance to moisture induced damage.	70% minimum
Indirect Tensile Strength, Cured Specimen, AASHTO T 283, 77°F (25°C)	To evaluate cured strength.	70 psi minimum
Tensile Strength Ratio, AASHTO T 283, 77°F (25°C) Based on Moisture Conditioning on Cured Specimen	To evaluate resistance to moisture induced damage.	0.70 minimum
Raveling Test, ASTM D7196, 50°F (10°C) or 77°F (25°C), 50% Humidity	To determine mixtures resistance to raveling and evaluate curing.	7 % loss, maximum
RAP Coating Test, AASHTO T 59, using RAP from mix design and emulsion, water and additive rates at optimum from mix design	To evaluate coating of binder.	Minimum good

[1]These items are reported by convention and are necessary for mix design calculations and to assess the overall quality of the mix design.

AASHTO T 283

A modified AASHTO T 283 to account for cold mixes can be used to determine cured mixture strength and resistance to moisture induced damage. Samples should be mixed, compacted and cured as described in the mix design procedure in Appendix B. Dry or unconditioned tensile strength can be used to evaluate cured strength. Samples are conditioned by vacuum saturation of 55-75 percent, as determined in AASHTO T 283. AASHTO T 166 Method A bulk specific gravity, or AASHTO T 331 bulk specific gravity when required, should be used to determine volume of air and volume of sample for saturation calculations. Vacuum saturated samples are soaked in a 77°F (25°C) water bath for 24 hours and the indirect tensile strength determined. Unconditioned or dry samples should be placed in leak proof bags and placed in a 77°F (25°C) water bath for a minimum of 45 minutes and tested for indirect tensile strength.

Other mix design procedures have used saturation levels of 55-75% with vacuum saturated samples soaked in a 77°F (25°C) water bath for 23 hours followed by a 30-45 minute soak at 104°F (40°C). Indirect tensile strength tests for conditioned and dry samples are performed at 104°F (40°C). Indirect tensile strength testing at these elevated temperatures is not recommended for cold mixes.

Minimum Marshall stability values for CIR mixtures are well established. Minimum dry tensile strengths are not well established. Values in the 45-50 psi range have been reported; however, these values are for tensile strengths at 40°C. Correlations between Marshall stability and tensile strength are not well established either; however, relationships between Marshall stability and tensile strength are available from field data from Washington Road. A Marshall stability of 1,250 lbs correlates to a tensile strength of 46 psi; however, the coefficient of determination (R^2) is very low.

Indirect tensile strengths and TSRs are available from the mix design and field produced samples from QO, all together and separated by age of samples. The average results are shown in Table 21. Dry indirect tensile strengths exceeded 70 psi and the average TSR was 0.84. Both values were similar to mix design values. A minimum tensile strength of 70 psi is recommended for samples saturated to 55-75% saturation and tested at 77°F. If field samples are used for verification, samples should be compacted as soon as possible. A minimum TSR of 0.70 is recommended.

Table 21. Modified AASHTO T 283 Results from Quincy-Oroville Road.

	Mix Design	QO All samples	QO AM Samples	QO PM Samples
Dry Tensile Strength	73.5 psi	73.1 psi	72.1 psi	74.5 psi
Conditioned Tensile Strength	59.6 psi	60.7 psi	64.5 psi	55.6 psi
TSR	0.81	0.84	0.89	0.75

Marshall Stability

As an option, AASHTO T 245 can be used to determine cured mixture strength and resistance to moisture induced damage. Samples should be mixed, compacted and cured as described in the mix design procedure in Appendix B. After curing, samples are tested for Marshall stability and retained Marshall stability with modifications to the testing procedures to account for cold mix. Marshall stability is used to evaluate cured strength and a minimum Marshall stability of 1,250 lbs is a well established minimum. For retained Marshall stability samples are conditioned by vacuum saturation of 55-75 percent, as determined in AASHTO T 283. AASHTO T 166 Method A or AASHTO T 331 bulk specific gravity could be used to determine volume of air and volume of sample. At the lower saturation level of 55-75 percent AASHTO T 331 could be used but the test would be more severe than if using AASHTO T 166 method A.

Vacuum saturated samples are soaked in a 77^{o}F (25^{o}C) water bath for 23 hours and then placed in a 104^{o}F (40^{o}F) water bath for 30-40 minutes and the conditioned Marshall stability determined. Unconditioned or dry samples can be placed in a 104^{o}F (40^{o}F) forced draft oven for 2 hours or placed in leak proof bags and placed in a 104^{o}F (40^{o}F) water bath for a minimum of 45 minutes. Samples are tested dry for unconditioned Marshall stability. Retained Marshall stability is the conditioned stability divided by the unconditioned stability.

Marshall stability results are available from field produced samples from WR. Mix design samples were made without aggregate base and some portions of WR included aggregate base in the CIR mix. Average Marshall stability was 2,868 lbs and the conditioned stability was 1,466 lbs for a retained stability ratio of 0.54. Well established values for CIR are Marshall stability of 1,250 lbs and a minimum retained stability ratio of 0.70. Stability ratios of 0.60 are usually considered acceptable if the conditioned Marshall stability exceeds 1,250 lbs. A minimum Marshall stability of 1250 lbs and a retained Marshall stability ratio of 0.70 is recommended for CIR mixtures with 100% RAP.

Raveling Test

The raveling test (ASTM D7196) was developed by Road Science and their predecessors for their Reflex® emulsion, which is a solventless engineered emulsion that utilizes a chemical induced break rather than simple water evaporation. The test is a modified slurry seal wet track abrasion test that measures the resistance of a partially cured and compacted CIR test specimen to abrasion from a weighted rubber hose. After curing for the designated time at specified temperature and relative humidity, the samples are abraded in a modified slurry seal wet track abrasion device and the weight loss that occurs is measured against specified standards to assess raveling susceptibility potential.

The raveling test should be performed on samples compacted at optimum emulsion and mix water contents. ASTM D7196 does not specify curing temperatures or relative humidity levels. The test has traditionally been performed at 50^{o}F (10^{o}C) at 50% relative humidity. Field temperature and relative humidity affect when an emulsion will break and how fast it will cure. Construction difficulties have been encountered with some emulsion formulations when field conditions were considerably warmer and drier than test conditions. ARRA [(20)] has recommended performing the raveling test at a second compaction temperature, 40^{o}C (104^{o}F), at

0.25-0.5 percent less emulsion, to evaluate the effect of increased construction temperatures on test results.

It is recommended that the raveling test be made a part of FLH mix designs to ensure adequate initial strength and resistance to raveling. Test conditions should be selected to match anticipated construction conditions (pavement temperatures and anticipated relative humidity). Minimum test conditions of 50°F (10°C) at 50% relative humidity are recommended for most circumstances. For projects being constructed in hot conditions, testing at 75°F (25°C) at 50% relative humidity should be considered. For extreme hot conditions, such as desert climates, consideration of compacting and testing specimens at 104°F (40°C) at 30% relative could give better results. A maximum percent loss of 5-7 percent has been successfully used. A maximum percent loss of 7 percent is recommended. If a mix design compactive effort of 35 gyrations is used as recommended, then it is recommended that compaction for the raveling test be increased from 20 gyrations to 25 gyrations. It is recommended that the mix design specify testing conditions of 50°F at 50% relative humidity and contain the clause “unless directed by the engineer” for instances when testing at higher temperature and lower relative humidity levels would provide more realistic conditions and better field performance. An alternative would be to follow the ARRA recommendations [20].

Mix Design Report

The findings of a mix design should be presented in a mix design report. A good report should include the following data: 1) gradation of millings, 2) recommended water content range as a percentage of dry millings, 3) optimum emulsion content as a percentage of dry millings, 4) amount of additive as a percentage of dry millings, 5) corresponding density, 6) air void level, 7) tensile strength (or Marshall Stability), 8) tensile strength ratio (or retained stability) and, 9) raveling at recommended moisture and emulsion contents. The recommended mix design procedure for CIR mixtures is found in Appendix B.

FDR MIX DESIGNS

Background

In 2001 ARRA published the BARM which summarized the available mix design procedures and provided recommended mix design steps for FDR mixtures. There is no nationally accepted method for design of FDR mixtures. Most agencies that use FDR have their own mix design procedures and these have ranged from simple empirical formulas to more complex methods with performance based testing. The definitions between FDR and soil stabilization (often called granular base stabilization or GBS) can overlap. Therefore, the majority of these mix design methods have provisions for materials with less than 8-10 percent passing No. 200 sieve (FDR or Type 1 FDR) and for materials containing more than 8-10 percent and less than 20 percent passing No. 200 sieve (GBS or Type 2 FDR). A currently popular method is the procedure developed by Road Science, LLC and their predecessors. Many State DOTs have adopted the Road Science mix design procedure, which uses Superpave principals including specimen compaction using the SGC. The procedure follows the recommended steps outlined in the

BARM [1] and includes testing samples for initial strength development, resilient modulus and resistance to thermal cracking.

The recommended mix design procedure for FDR is presented in detail in Appendix C. The procedures are based on methods found in the MODOT, NCDOT and ALDOT Supplemental Specifications [22,23,24] and the South Dakota FDR study [25]. The recommended procedure eliminates initial strength testing due to reported difficulties with the procedure and FLH's stated desire to move away from Marshall and Hveem testing procedures, and AASHTO T 322 testing, thereby increasing the number of agencies/contractors that will be able to perform the procedure.

Procedures

Establishing a mix design for an FDR project requires the collection of field samples of the existing pavement to be recycled and subsequent laboratory (mix) testing to establish a target formulation of the materials (asphalt emulsion, water, RAP, add-stone and additives, if needed) that will be used during construction.

Sample Collection

Auger borings, test pits and coring the pavement to be recycled are the preferred methods for collection of representative samples of the target pavement. This is undertaken to establish whether the properties of the pavement are consistent along its length, width, and depth and to obtain materials for the mix design.

A minimum of three cores per lane mile should be obtained to check for pavement consistency with additional cores where visual differences in the pavement are noticed. These cores are also used for determination of asphalt content and gradation analysis of the existing pavement. If cores show more than a 2-inch (50 mm) difference in bituminous surface between sections, then separate mix designs are recommended for each of these pavement segments.

Approximately 350 lbs of usable material is required for each mix design. Pavement cores should be at least 6 inches in diameter if auger borings will be used to obtain base materials. The collected samples are then processed using a laboratory jaw crusher or other technique that will yield a material similar to material manufactured during actual reclaiming operations. Base material should be obtained from core locations by auger boring or from test pits.

RAP Processing

Cores/slabs of asphalt pavement need to be crushed to produce RAP of a gradation similar to what is expected during reconstruction. RAP gradation will vary throughout the project due to daily temperature changes of the reclaimed pavement and due to changes in mix composition of the pavement and normal mixture variation. The Road Science recommended FDR RAP gradation and the suggested FLH CIR RAP gradation are shown in Table 22 and presented graphically in Figure 24. As shown in Figure 24 the suggested FLH gradation is finer than the Road Science gradation. Because of the difference in equipment used with FDR, the Road Science gradation is recommended for use with the modification of 100 percent passing the 1.5 inch sieve rather than the 1.25 inch sieve.

Table 22. Cold Recycling Gradation Requirements.

Sieve Size	Road Science FDR	Suggested FLH CIR Gradation
1.5 inch		100
1.25 inch	100	
1 inch	85-95	90-100
3/4 inch	75-85	85-95
1/2 inch		75-85
No. 4	30-40	35-50
No. 16		5-16
No. 30	1-5	
No. 200		0-7

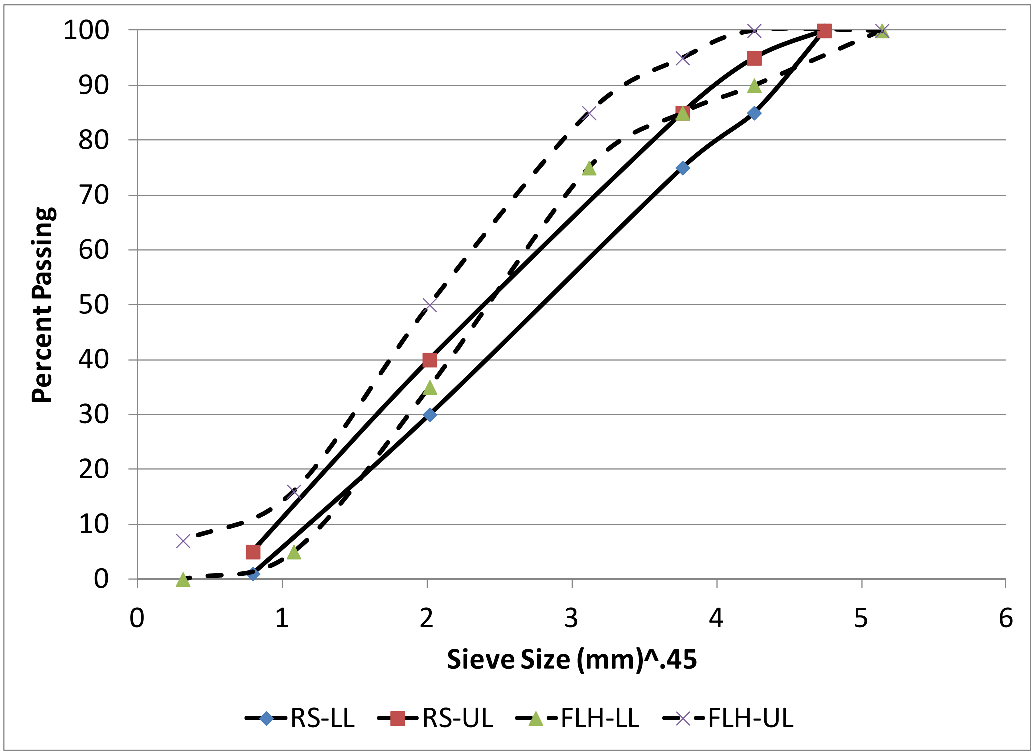

Figure 24. Scatter Plot. Road Science FDR and FLH CIR RAP Gradations.

RAP and Base Material Evaluation

A washed sieve analysis, (AASHTO T 11 and T 27) should be performed on the aggregate base. The processed RAP should have a dry sieve analysis test (AASHTO T 27) only performed. Gradation of the combined RAP and base sample is determined by combining gradations of both materials to the planned percentages. A sand equivalent (SE) test (ASHTO T 176) should be performed on the RAP and base material to determine if they are suitable for stabilization with asphalt emulsion. Ideally, combined RAP and base material for stabilization with asphalt emulsions should have less than 8-10 percent passing the No. 200 sieve and a SE of 30 or higher. Materials with lower SE and higher fines, up to 20 percent, have been successfully stabilized with asphalt emulsions. Strength will be less and comparisons with other stabilizing agents should be considered to ensure an optimized design. Many existing FDR mix design procedures(22,23) distinguish between FDR (< 10% passing No. 200 sieve) and granular base stabilization (GSB) (10 - 20% passing No. 200 sieve). These are often referred to as type 1 and type 2 designs, respectively, and specification requirements are different. One procedure (24) differentiated based on average rainfall.

Optimum Moisture Content

Samples of RAP and aggregate base should be combined to the desired percentages and the optimum moisture content (OMC) of combined material determined using modified Proctor compaction in accordance with AASHTO T 180 Method D (6-inch diameter mold). Materials should be mixed with the target moisture, sealed and set aside for a minimum of 3 hours. Materials with a significant amount of RAP or coarse material may not produce a well defined OMC curve. If so, then an OMC of 2-3 percent is recommended. If the combined materials contain less than 4 percent passing the No. 200, testing for OMC is not required because there will not be a well defined peak and an OMC of 2-3 percent should be used.

Mixing and Compaction

FDR materials behave like aggregate base when compacted. Therefore, FDR materials are compacted at or near the OMC. Asphalt emulsions are liquid when applied and act like additional moisture. Therefore, samples are usually mixed with a percentage of the OMC with asphalt emulsion making up the remainder of the required liquid. Water contents, not including water in the emulsion, is typically around 65 percent of OMC but is often based on anticipated average annual rainfall and SE as indicated below:

Average annual rainfall ≥ 20 inches:

- 60 to 75 percent of OMC if SE ≤ 30
- 45 to 65 percent of OMC if SE > 30.

Average annual rainfall < 20 inches:

- 50 to 75 percent of OMC if SE ≤ 30
- 40 to 65 percent of OMC if SE > 30.

SE values are based on combined materials.

Compacting samples at 60-65% of OMC will satisfy the above recommendations and is recommended.

Specimen size should be selected to produce a 95 ± 5 mm tall, 150-mm diameter sample. A minimum of three and preferably four emulsion contents in 1.0% increments that bracket the estimated recommended emulsion content are selected. Emulsion contents typically cover a range between 2.0% and 6.0% by dry weight of material. Six samples are compacted at each emulsion content, three for indirect tensile strength testing on cured samples and three for conditioned tensile strength on cured samples for moisture conditioning.

Two specimens are prepared for theoretical maximum specific gravity according to AASHTO T 209 or ASTM D6857. The dry-back procedure of AASHTO T 209 will be required. Loose RAP mixtures are cured as described below under Curing. RAP agglomerates that will not easily reduce with a flexible spatula should not be broken apart. Both specimens are tested at the highest emulsion content in the design and the maximum specific gravity back calculated for the lower emulsion contents.

Mixing of test specimens should be with a mechanical bucket mixer or laboratory sizes pugmill mixer or a combination of the two. Samples are mixed thoroughly with the recommended percentage of the OMC water for 60 seconds. If any additives are in the mixture, they are usually introduced next but should be introduced in a similar manner that they will be added during field production. Emulsion is then added and the sample mixed. One specimen is mixed at a time and mixing time with emulsion should not exceed 60 seconds.

Curing Before Compaction

Specimens are usually cured before compaction to mimic field conditions. Samples should be placed in sealed containers and allowed to sit for a minimum of 3 hours. An alternate procedure is to place samples in individual plastic containers, 4-7 inches high and 6 inches in diameter. The specimens are cured in a 104°F (40°C) forced draft oven for 30 ± 3 minutes prior to compaction.

Compaction

Samples are compacted in a SGC. Gyrations range from 30 to 40 gyrations in accordance with AASHTO T 312 except molds are not heated. Samples typically compacted immediately after initial curing. Asphalt emulsion can be heated to the expected delivery temperature as directed by the emulsion supplier.

Curing After Compaction

Samples are cured after compaction and before testing. The most common curing conditions found in the literature were 72 hours in a 104°F (40°C) forced draft oven or until dry in a 140°F (60°C) forced draft oven, or a maximum of 48 hours. The bottoms of the specimens should rest on racks with slots or holes for air circulation. After curing, samples should cool at ambient temperatures for 18 ±6 hours before testing. Specimens for maximum specific gravity testing are cured at the same conditions as compacted specimens. The 140°F (60°C) cure is recommended to match CIR curing requirements.

Laboratory Testing

Table 23 presents a list of laboratory tests that are typically performed as part of a mix design. The design emulsion content is the emulsion content(s) such that the FDR mix requirements listed in Table 23 are met.

Table 23. FDR Laboratory Mix Design Tests.

Design Parameters	Objective	Requirement
Gradation of Design Reclaimed Asphalt Pavement (RAP), AASHTO T 27	To ensure that the mix design meets the gradation specification	Modified Road Science Gradation, Table 7
Asphalt Content of RAP, AASHTO T 308	General information only	Report[1]
Gradation of Aggregate Base, AASHTO T 11 and T 27	General information only	Report[1]
Sand Equivalent of Combined RAP and Aggregate Base, AASHTO T 176	General information only	Report[1]
Maximum Dry Density and Optimum Moisture Content, Combined RAP and Aggregate Base, AASHTO T 180, Method D	To determine optimum moisture content for sample fabrication and for field compaction control.	Report[1]
Bulk Specific Gravity of Compacted Samples, AASHTO T 331 or AASHTO T 166 Method A	To determine air voids of compacted specimens	Report[1]
Maximum Theoretical Specific Gravity, AASHTO T 209 or ASTM D6857	To determine air voids of compacted specimens	Report[1]
Air Voids of Compacted and Cured Specimens, AASHTO T 269	Information only: FDR should not be designed, nor the asphalt emulsion content altered, to meet a specific air void content	Report[1]
Type 1 FDR < 8% Passing No. 200 sieve		
Indirect Tensile Strength, Cured Specimen, AASHTO T 283, 77°F (25°C)	To evaluate cured strength	40 - 45 psi minimum
Conditioned Tensile Strength, AASHTO T 283, 77°F (25°C) Based on Moisture Conditioning on Cured Specimen, Saturation to 55% minimum	To evaluate resistance to moisture induced damage	25 psi minimum,
Resilient Modulus, ASTM D7369, 25°C	To evaluate mixture strength and stiffness	150,000 -175,000 psi, minimum

[1]These items are reported by convention and are necessary for mix design calculations and to assess the overall quality of the mix design.

Table 23 (Con't.). FDR Laboratory Mix Design Tests.

Design Parameters	Objective	Requirement
Type 2 FDR 8 - 20% Passing No. 200 sieve		
Indirect Tensile Strength, Cured Specimen, AASHTO T 283, 77°F (25°C)	To evaluate cured strength	35 - 40 psi minimum
Conditioned Tensile Strength, AASHTO T 283, 77°F (25°C) Based on Moisture Conditioning on Cured Specimen, 24 hour soak at 77°F (25°C)	To evaluate resistance to moisture induced damage	20 psi minimum
Resilient Modulus, ASTM D7369, 25°C	To evaluate mixture strength and stiffness	120,000 -150,000 psi, minimum

Bulk Specific Gravity of Compacted Samples

Bulk specific gravity of compacted samples can be determined using either AASHTO T 166 Method A or AASHTO T 331. Due to the high air voids, AASHTO T 331 will most likely be required. Bulk specific gravity is necessary to determine air void contents and to determine percent saturation of AASHTO T 283 samples. Calculations for AASHTO T 283 percent saturation use SSD weights and recommends saturation levels of 70-80 percent saturation. Most FDR mix design procedures that require vacuum saturation recommend saturation levels of a minimum of 55 percent or 55-75 percent. If AASHTO T 331 is used, void contents will be higher than with AASHTO T 166 and it might not be possible to saturate samples to 70 -80 percent levels without damaging samples. It is recommended that reduced saturation levels of 55-75 percent be used for AASHTO T 283 with AASHTO T 166, or AASHTO T 331 if required, used for bulk specific gravity measurement.

Maximum Theoretical Specific Gravity

Maximum theoretical specific gravity can be determined in accordance with AASHTO T 209. However, *Section 11 Supplemental Procedure for Mixtures Containing Porous Aggregate*, or the dry-back procedure, will be necessary due to partially coated aggregate base material and uncoated RAP particles. An alternative method that does not require the dry-back procedure is *ASTM D6857 Standard Test Method for Maximum Specific Gravity and Density of Bituminous Paving Mixtures Using Automatic Vacuum Sealing Method.* However, ASTM D6857 will be affected by uncoated particles.

With higher aggregate base percentages and higher minus No. 200 materials, the dry-back procedure of AASHTO T 209 may not be feasible (type 2 materials). Many mix design procedures eliminate vacuum saturation for these mixtures and require a 24 hour soak for conditioned strength testing. Use of ASTM D6857 should be considered for these mixtures.

Initial Strength

Procedures are available for evaluating initial strength of FDR mixtures. Published procedures use either the Hveem Cohesiometer or Marshall stability. Hveem equipment is not readily available, especially the Cohesiometer, and many agencies are moving away from and no longer have Marshall equipment. In addition, issues have been reported with these procedures. These procedures are available and can be found in the literature [22,23,24] but are not recommended for inclusion in the FDR mix design at this time.

AASHTO T 283

A modified AASHTO T 283 to account for cold mix can be used to determine cured mixture strength and resistance to moisture induced damage. Samples should be mixed, compacted and cured as described in the mix design procedure in Appendix C. The dry or unconditioned tensile strengths are used to evaluate cured strength and conditioned samples to evaluate resistance to moisture induced damage. Specifications for dry tensile strengths vary from 35-45 psi depending upon materials (Type 1 or 2) and climate. Specifications for wet or conditioned tensile strengths vary from 20-25 psi depending upon saturation levels, materials (Type 1 or 2) and climate. Tensile strength ratios are not normally used but specifications vary from 0.50 to 0.70, depending upon the same factors for wet strengths. Recommended levels are contained in the draft mix design in Appendix C.

Cleaner samples (< 8-10% passing No. 200 sieve) are typically conditioned by vacuum saturation to a minimum of 55 or 55-75 percent saturation as determined in AASHTO T 283. AASHTO T 166 Method A or AASHTO T 331 bulk specific gravity could be used to determine volume of air and volume of sample. Vacuum saturated samples are soaked in a 77°F (25°C) water bath for 24 hours and the indirect tensile strength determined.

Due to the difficulty of determining maximum specific gravity (AASHTO T 209) of GSB samples (10-20% passing No. 200 sieve), vacuum saturation is often not required, removing the need for Gmm determination. Samples are conditioned by a 24 hour soak in a 77°F (25°C) water bath. An alternative to this procedure would be to use ASTM D6857 for determination of Gmm and requiring vacuum saturation to a minimum of 55 or 55-75 percent saturation as determined in AASHTO T 283.

Unconditioned or dry samples should be placed in leak proof bags and placed in a 77°F (25°C) water bath for a minimum of 45 minutes and up to 2 hours and then tested for indirect tensile strength.

Resilient Modulus

A few FDR mix design procedures require resilient modulus testing. Testing can be performed on dry indirect tensile strength specimens prior to tensile strength testing. The test is performed in accordance with ASTM D7369 on two specimens at each emulsion content. Samples are tested after a minimum 2 hour conditioning at 25°C. A Poisson's ratio of 0.30 – 0.40 is used for analysis. Minimum resilient modulus values range from 120,000 – 175,000 psi depending upon materials (type 1 or 2). This test is not recommended for inclusion at this time due to the added cost and limited number of testing firms that could perform the test.

Mix Design Report

The findings of a mix design should be presented in a mix design report. A good report should include the following data: 1) washed gradation of materials, 2) gradation of millings, 2) gradation of blended material, 3) sand equivalent of blended material, 4) density and OMC from modified Proctor test, 5) moisture content used in mix design, 5) optimum emulsion content as a percentage of dry material, 6) amount of additive as a percentage of dry material, 7) corresponding density, 8) air void level, 9) tensile strength 10) conditioned tensile strength.

CHAPTER 8 – SPECIFICATION RECOMMENDATIONS

SCOPE

This chapter presents proposed modifications to FLH's CIR draft specification *Section 324. – COLD IN-PLACE RECYCLED ASPHALT BASE COURSE,* dated 3/26/12. The information used to develop the recommendations was obtained from the construction data and literature generated in this study and from the Asphalt Recycling and Reclaiming Association (ARRA) construction guidelines [26]. A complete review of FLH's specification book was outside the scope of this project. Therefore, some of the recommendations made could exist in other sections of the specifications book. One of the objectives of this study was to recommend compaction specifications; therefore, the review is broken down into two sections, one on general draft specifications and the other on compaction requirements.

DRAFT SPECIFICATION REVIEW

324.03 Proportioning

A comment "unless directed by the engineer" should be added to testing conditions for the Raveling Test (ASTM D7196) to account for conditions when the standard compaction at 50% relative humidity at 50^{o}F could be too severe, as explained in Chapter 7 under CIR mix designs.

The notes to Table 324-1 are not in agreement with the proposed CIR mix design in Appendix B. The proposed mix design does not recommend compacting samples to a design air void content but to a standard compactive effort of 35 gyrations in the SGC. The proposed mix design recommends saturating specimens to 55-75% saturation as determined in AASHTO T 283 and using AASHTO T 166 Method A, with AASHTO T 331 if required, to measure bulk specific gravity for air void and saturation calculations as stated in Chapter 7.

324.05 Equipment

For section **(d) Paver**, many agencies add a requirement that a minimum 170 horsepower track paver be used when a windrow pickup device is used to deliver CIR mix to the paver hopper.

The minimum weights and minimum number of rollers for the project should be specified. Due to the inherent nature of the materials, obtaining accurate, timely density measurements of the compacted CIR mixture is problematic. For optimal CIR mixture performance it is important that the mat be adequately compacted. The most common method of controlling density is percent of a test strip density and the density of a test strip is a function of the type, number and weights of the rollers. Therefore, the minimum weights and number of rollers must be specified.

With production rates of over 2 lane miles per day, a minimum of three rollers are usually required to keep up with the paver. Pneumatic rollers should have a weight of not less than 25 tons and double drum vibratory steel-wheeled rollers should have a weight of not less than 10 tons. Rollers should have a minimum width of 65 inches, be self propelled, and have operating scrapers and water spray systems to prevent mixture pickup.

324.07 Production Start-Up Procedures

In section **(b) Test strip** the procedure calls for obtaining three samples of RAP before the addition of asphalt emulsion and verifying that 100 percent pass the 1-inch sieve. The original FLH specification allowed 100 percent passing the 1.5-inch sieve. It is recommended that this be raised to 100 percent passing the 1.5-inch sieve as recommended in Chapter 6 or adjust the mix design and specification to 100 percent passing the 1.25-inch sieve.

Many agencies allow the CIR contractor to continue operations if the test strip is completed successfully. Section **(c) Mix design verification** requires mix design verification in accordance with Table 324-1 using materials from the test strip. There are two important issues with these requirements that need to be addressed before implementation. The first issue is the effect of time on mix properties and the second is the impact of construction delays and idle equipment due to testing times.

As shown in Table 7, initial curing time and temperature of the CIR mixture prior to compaction can have a significant effect on bulk specific gravity and mix properties. Samples will need to be compacted within the same expected timeframe they would in the roadway to provide meaningful results. Samples for AASHTO T 283 testing could be compacted in a field laboratory if located close to the project

The second issue is the time required to perform the required testing. AASHTO T 283 samples must cure for 16-48 hours, cool overnight, be saturated and soaked for 24 hours prior to testing. Samples compacted on day one would not have results until day five. Compaction temperatures would need to be addressed if compaction at other than in-situ temperatures (such as ambient) is required. Additional time could be required to bring samples to the proper temperature before compaction. The cost of having mobilized equipment idle for four days needs to be balanced against the benefits of mix verification. For the two projects evaluated in this study, meeting moisture sensitivity requirements was an issue for WR but not for QO.

324.08 Pavement Recycling and Mixing

The specification requires 100 percent passing the 1-inch sieve and it is recommended that this be raised to 100 percent passing either the 1.5 or 1.25-inch sieve as discussed above.

There was no requirement for water added to RAP at the milling head and/or in the mixing chamber. Many specifications require that “the water be clean and free of deleterious concentrations of acids, alkalis, salts, sugar and other organic, chemical or deleterious substances. The water shall not cause an adverse effect on either the recycling agent or the recycled pavement mixture.” However, without actual limits on the above quoted materials, the requirement could be unenforceable. Consideration should be made of including a statement that water shall have no adverse effect on either the recycling agent or the recycled pavement mixture.

COMPACTION SPECIFICATION

Table 324-2 lists the frequency of testing as one test per 2000 sy. A typical testing frequency is one test per 1000 feet and a minimum of 10 tests per day. Lot sizes vary from as small as 3000 sy to one day's production.

There are three methods of specifying percent compaction that have been used successfully, they are: 1) percent of maximum specific gravity (Gmm), 2) percent of laboratory molded density and 3) percent of control strip density. Percent of Gmm is the most common procedure used for control of compacted density for HMA construction. Percent of laboratory molded density is the most common method of controlling density of soil and aggregate base materials. Percent of control strip density is often used when laboratory measurements of materials are problematic.

Percent Compaction Based on Gmm

Air voids are one of the most important, if not the most important, mix property that relates to HMA performance. Therefore, control of air voids through the use of Gmm makes sense for HMA. CIR mixtures are typically open graded mixes and not as sensitive to air void content as HMA. For CIR the goal of compaction is not to reach a target air void content but to ensure the mixture has obtained a maximum obtainable density using reasonable compactive effort based on in situ gradation, asphalt content and other mix properties and environmental conditions.

There are several obstacles that would need to be overcome before percent of Gmm could be used for control of density of CIR mixtures. They are:

- CIR is not designed on an air void content so what target value would be used.
- Gmm was shown to vary considerably along WR and QO, requiring considerable testing to have enforceable results.
- Dry-back procedures of AASHTO T 209 would be required or ASTM D6857 could be used and the reliability of these results for CIR mixtures is not well established.

Mix design Gmm values are not used with HMA due to issues with how closes the mix design Gmm reproduces field conditions. This would be even more of an issue with CIR mixtures. CIR projects tend to be much more variable along their length, with patches and maintenance mixes, compared to the uniform nature of plant produced HMA. Figure 25 shows the variability between mix design Gmm and field Gmm samples for WR and QO. Error bars are ± 2 standard deviations. The differences between mix design and field Gmm values for WR were considerable. The number of Gmm tests required to accurately determine maximum theoretical density and the time required to perform the dry-back procedure of AASHTO T 209 could make this method unworkable. Finally, it would be difficult for contractors to accurately monitor percent compaction in a meaningful way as correlations between nuclear gauge readings and cores could not be established in a timely manner. The major advantage to this procedure would be that density could be checked using cores from the pavement cut at a later date. Sufficient core data was not available from this project to evaluate this method and percent of Gmm has not been used for CIR construction. It is not recommended for use at this time.

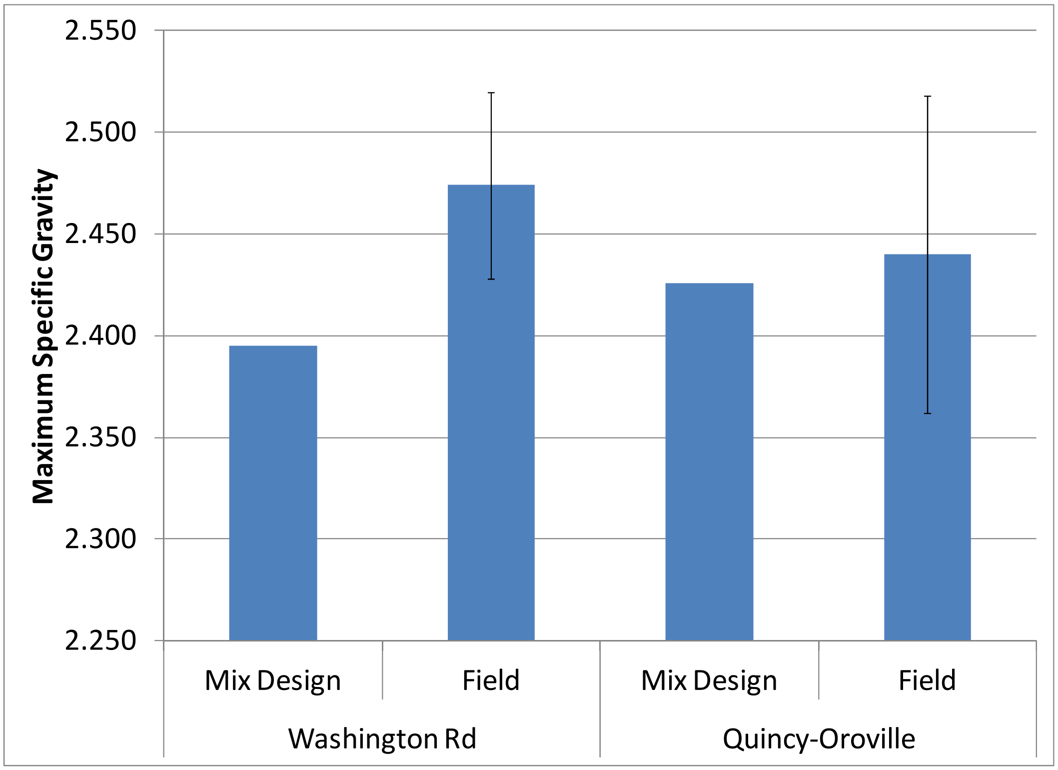

Figure 25. Bar Chart. Comparison of Mix Design Gmm with Variation (± 2 Standard Deviations) in Field Produced Gmm.

Percent of Laboratory Compaction 324.09 (1) Type A Compaction

FLH Type A compaction requires CIR mixtures be compacted to a minimum of 98 percent the mix design density. Table 24 shows compaction results from WR and QO using percent of both mix design density and lab molded density. Figure 25 shows the difficulty that could be experienced with non representative mix design values when used for compaction monitoring. Lab molded density for QO is not shown in Table 24 due to effects of sample age and temperature when compacted in the field lab, as explained in Chapter 6.

Regardless of the lab molded density used, only a small portion of either project exceeded the recommended 98 percent compaction. Either the percent compaction is unrealistically high, nuclear gauges underestimate density or the compactive effort of 35 gyrations is not representative of field produced mix. If Type A compaction is used then it is recommended that 30 gyrations in the SGC be used for mix design compactive effort.

There are several other issues with this method of compaction control. What actually caused low density results, a lack of field compactive effort or changed materials? In addition, with a change in materials resulting in higher percent compaction, a contractor could simply reduce compactive effort and stay within specification limits. Use of a mix design target density does not account for changed materials. If lab molded samples are used, then issues must be resolved related to when and how often to sample and what restrictions should be placed on compaction times and mixture temperatures, as discussed in Chapter 6. It is interesting to note that for WR, the standard deviation for percent compaction based on lab molded samples is higher than when using the mix design value.

Table 24. Type A Compaction Statistics.

		WR	QO
	Mix Design		
Percent	Avg.	90.7	94.5
Compaction	Std. Dev.	3.48	2.18
Percent	> 90%	58	98
Project	> 95%	11	42
Compacted	>98%	2	6
	Lab Molded Field Sample		
Percent	Avg.	90.4	N/A
Compaction	Std. Dev.	4.40	N/A
Percent	> 90%	58	N/A
Project	> 95%	18	N/A
Compacted	>98%	5	N/A

N/A Data not available.

Percent of Laboratory Compaction 324.09 (2) Type B Compaction

FLH Type B compaction requires CIR mixtures be compacted using the rolling pattern established in a test strip. The main issue with this procedure is no provision for density testing and no adjustments for changed materials. This method is not recommended for use.

PROPOSED COMPACTION METHOD

The goal of CIR compaction is not to reach a target air void content but to ensure the mixture has obtained a maximum obtainable density using reasonable compactive effort based on in situ gradation, asphalt content and other mix properties and environmental conditions. The procedure needs to allow the contractor to monitor compaction as well as provide the owner agency some assurance that adequate compaction is being achieved. The following procedure is recommended for adoption by FLH. It is a combination of some parts Type A compaction with Type B compaction.

For the following procedure to adequately monitor percent relative compaction, it is imperative that a minimum of three rollers meeting the requirements previously described be used. Either wet or dry densities could be used. Wet densities will not address wet areas of the pavement and dry densities will require moisture content sampling and testing. Moisture content can be determined in accordance with AASHTO T 255 or AASHTO T 329. Oven drying can result in higher moisture contents than micro-wave drying. Strict adherence to weather restrictions is required and inspection would be required to ensure the contractor follows the prescribed rolling pattern.

If the proposed procedure is followed with the above requirements, then reasonable compaction would be obtained that would allow for inherent material variability. If a higher level of reliability is required, lab molded samples could be compared to test strip target values to verify compactive effort. A correlation between nuclear gauge density and lab molded density would need to be established.

Proposed Compaction Specification

Initial Compaction

During the first day of production construct a minimum 1,000 foot long control strip to prove to the owner agency that the proposed construction procedures will meet the project requirements including:

1. Demonstrate that the equipment, materials, and processes proposed can produce a recycled pavement material layer that conforms to the requirements;
2. validate the optimal rates for recycling agent, water and any additives recommended for the reclaimed asphalt pavement; and
3. Determine the sequence and manner of rolling necessary to obtain the maximum obtainable (target) density.

Determine a rolling pattern during construction of the control strip necessary to achieve maximum obtainable (target) field density. The contractor will provide the sequence and manner of rolling which will define maximum compaction by establishing a rolling vs. density chart that shows the progress of densification from initial lay down through maximum obtainable density at the “break over point” using a properly calibrated nuclear density gauge in accordance with ASTM D2950. The peak density thus obtained is the target density. Strictly follow the rolling pattern determined to ensure compaction is met for the entire CIR surface area or until conditions indicate a new rolling pattern is required.

To determine relative compaction divide the project into lots consisting of a maximum of one day’s production. For each lot determine the density in accordance with ASTM D2950 and determine percent relative compaction by dividing the test density obtained by the target density from the control strip. Select density test locations at random at a frequency of one test per 1000 feet and a minimum of 10 tests per lot.

Establish a new rolling pattern and target density if:

1. Two or more consecutive individual locations have less than 95 percent or greater than 105 percent of the target density.
2. The average percent relative compaction for the lot is less than 97 percent or greater than 103 percent of the target density.

Any individual test result that is more than three standard deviations from the mean of the lot is an outlier and should not be used.

Secondary Compaction

A minimum of two days after initial compaction and after completion of moisture cure but before placing any final surfacing, perform secondary compaction with a pneumatic and steel drum roller. Perform secondary compaction after the morning sun has risen and when the pavement temperature is at least 80°F (27°C).

Establish a new rolling pattern using a minimum of four passes to establish a maximum obtainable (target) field density using the same equipment and procedures used to initially compact the CIR mix. Continuously verify secondary compaction is within 5% of the maximum obtainable (target) density by nuclear density gauge testing in accordance with ASTM D2950. Cease secondary compaction if roller checking or cracking occurs or if the pavement temperature drops below 80°F (27°C).

EVALUATION OF PROPOSED SPECIFICATION

Density test results from WR and QO are available to evaluate the above proposed specification. To make the evaluation it is assumed that the required rolling pattern was followed by the contractor and that consistent changes in relative compaction were caused by material variations and not changes in the prescribed compactive effort. Density test results from the initial days testing on the first 1,000 feet were used to establish the initial target density.

Washington Road

Figure 26 shows individual and average lot percent relative compactions for WR. Individual test results are the average of three measurements. Outliers are identified on the plots and were not used to calculate averages or standard deviations. As shown in Figure 26, two or more individual test results fall below 95% relative compaction in lot 2 and the lot average is below 97 percent. A new rolling pattern to establish the maximum obtainable density (new target value) would be required. A new target value was estimated from the data for lot 2 and applied to lots 2-4. The results are shown in Figure 27. As shown in Figure 27, all test results now fall within the specification requirements.

Quincy-Oroville Road

Density testing on QO was performed at a much higher frequency than for WR and production days were not indicated on the QA data sheets. Therefore, lots were set at approximately 10,000 foot intervals, resulting in approximately twice the number of individual test results per lot compared to WR. Individual density results are not averages of multiple measurements.

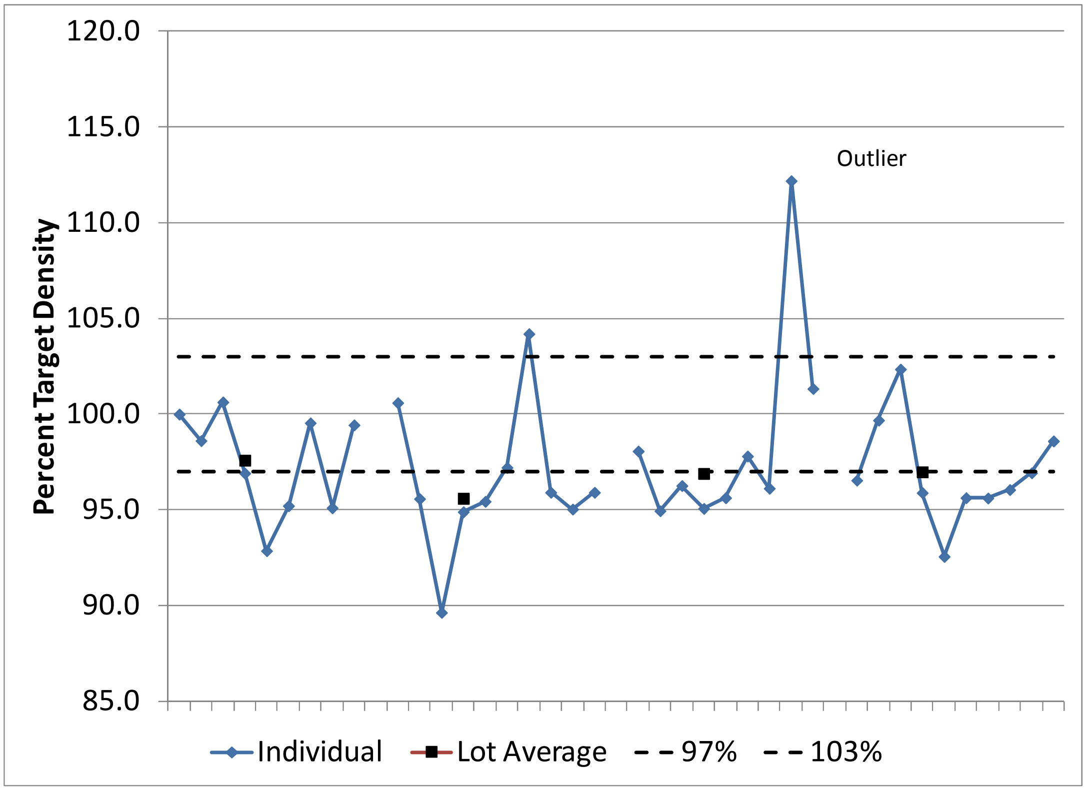

Figure 26. Scatter Plot. Percent Relative Compaction, WR.

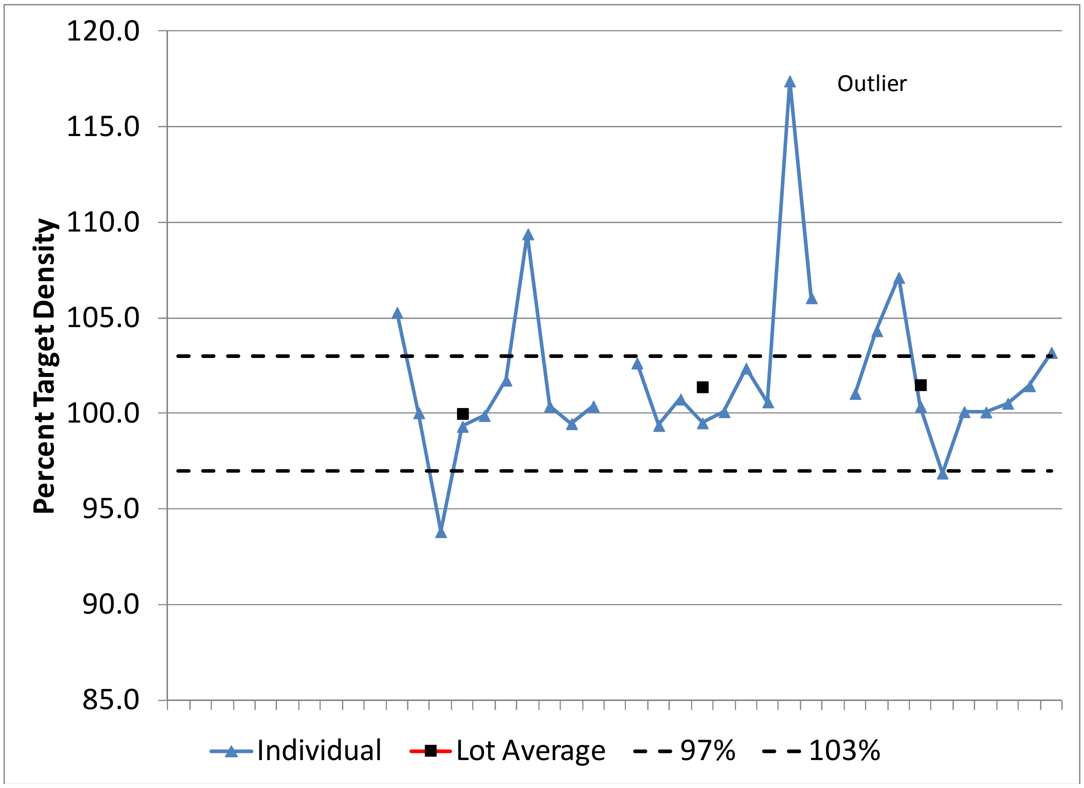

Figure 27. Scatter Plot. Revised Percent Relative Compaction, WR.

QO Right Lane

Figure 28 shows individual and average lot percent relative compactions for the right lane of QO. All lot averages are within 97-103 percent relative compaction. However, beginning in lot 2 numerous individual values were below 95 percent. Additional roller passes would be required to determine if additional density could be achieved. This would establish a new rolling pattern and possibly a new maximum obtainable density (new target value). A new target value was estimated from the data for lot 2 and applied the remaining lots. The results are shown in Figure 29. There are several individual test results in lot 2 that exceed 105 percent relative compaction and a new target density would be required for the first half of lot 2. This is possibly an area of high material variability which is what would be expected in an area of extensive maintenance patches. A new target density (third) would again be required for the last half of lot 2 because compaction results would be below 95 percent. The third target density would be similar to the previously established one. A revised plot is not shown. Without the adjustment for the first half of lot 2, all lot average percent relative compactions fall within the specification requirements.

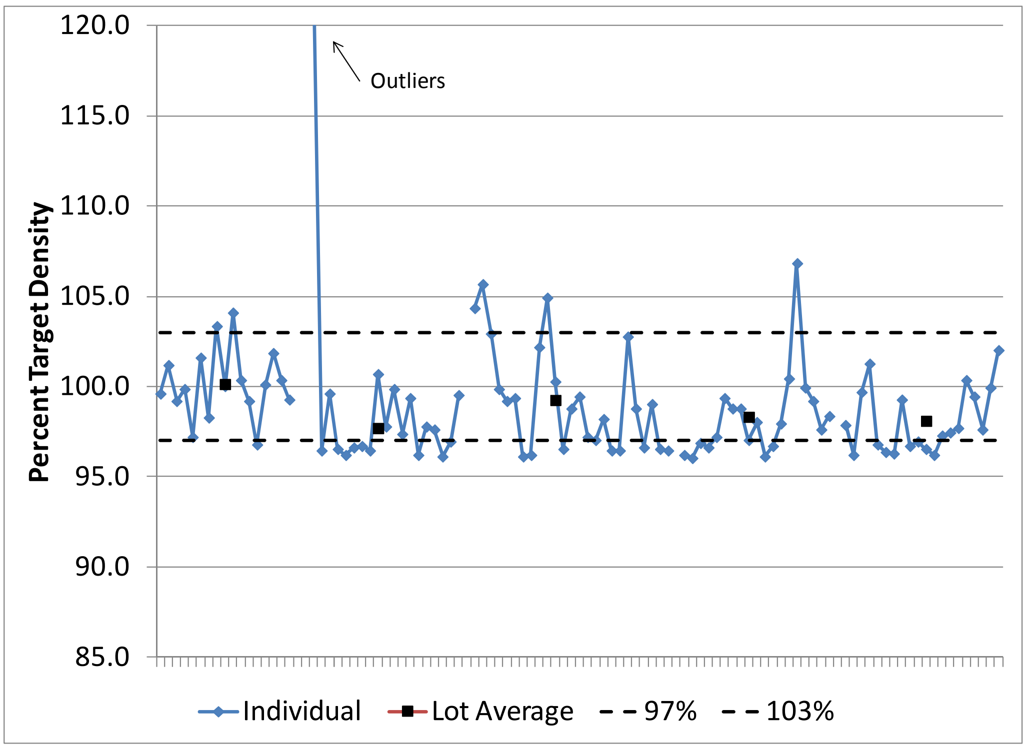

Figure 28. Scatter Plot. Percent Relative Compaction, Right Lane, QO.

QO Left Lane

Figure 30 shows individual and average lot percent relative compactions for the left lane of QO. All results are outside acceptable limits indicating that the first section of the left lane was not representative of the remainder of the project and a new rolling pattern and target density would need to be established. A new target value was estimated from the data after the first 3 tests of lot

1 and applied to the remainder of the data. The results are shown in Figure 31. As shown in Figure 31, all test results now fall within the specification requirements.

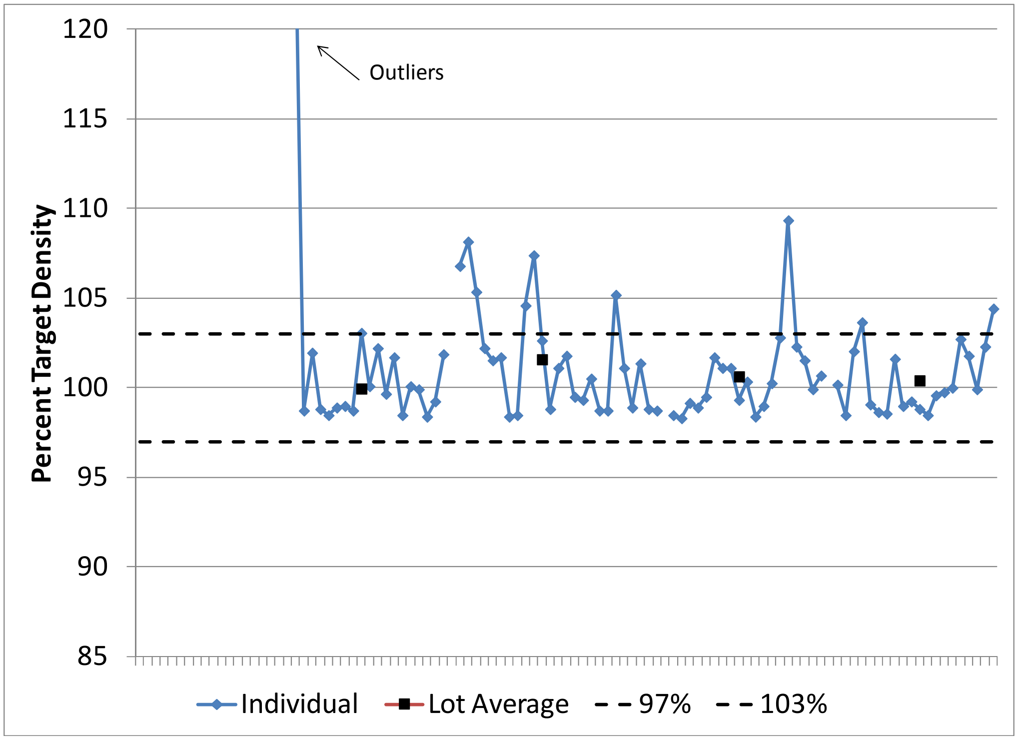

Figure 29. Scatter Plot. Revised Percent Relative Compaction, Right Lane, QO.

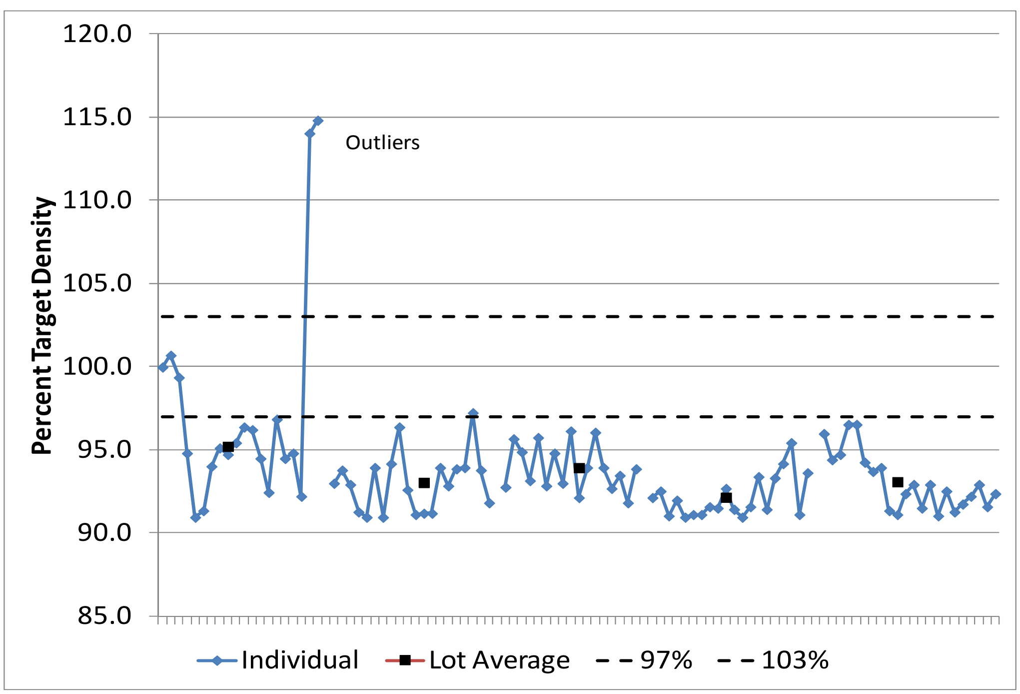

Figure 30. Scatter Plot. Percent Relative Compaction, Left Lane, QO.

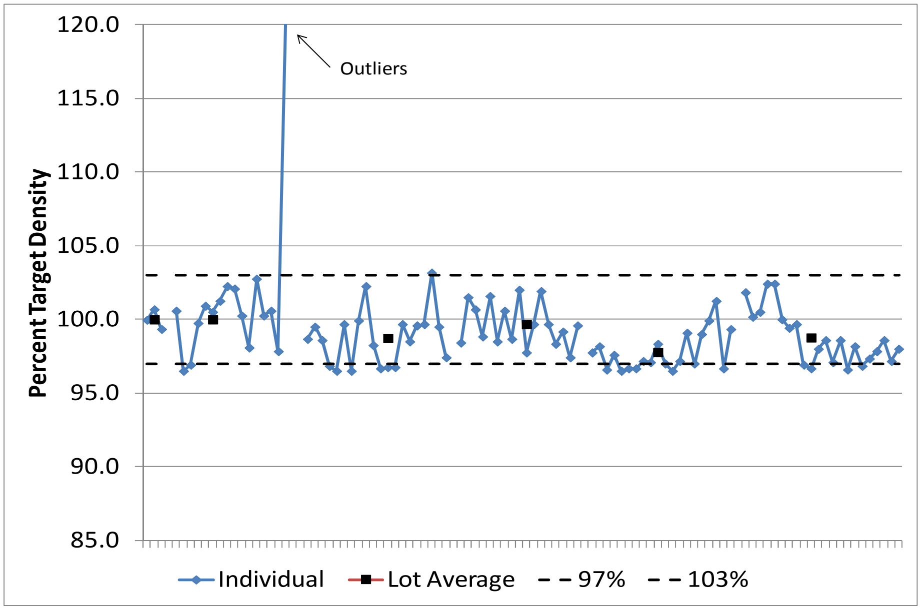

Figure 31. Scatter Plot. Revised Percent Relative Compaction, Left Lane, QO.

Pay Factors

Pay factors have been utilized in HMA construction to provide extra pay for high quality work and reduce pay for lower quality work. Use of pay factors generally requires a high standard of precision for test methods, certified technicians and a good understanding of inherent material variability. In addition, quality characteristics used for pay should be directly, or at least indirectly, related to performance. Even though one of the objectives of this study was to evaluate CIR variability, the precision of the required test methods, the inherent material variability, and their effects on quality characteristics such as compaction have not been sufficiently established to recommend pay factors at this time. However, the proposed compaction specification should result in a high majority of the project falling within the prescribed relative compaction specification limits.

Table 25 shows individual and lot averages and standard deviations for WR and QO using a single target value established at the beginning of the project and adjusting the target value for material variability as recommended. As shown in Table 25, using a single target value does not adequately account for inherent material variability. This would be true for Type A compaction using mix design target values as well. For WR, adjusting the target value resulted in 86 percent of the individual test results falling within the prescribed 95-105 percent relative compaction and 90 percent of the lot averages falling within the prescribed 97-103 percent relative compaction. For QO, the percentages are 90 and 99 percent for individual and lot averages, respectively. Most HMA PWL specifications require 90 percent within limits for full pay. For the two pavements evaluated, the proposed specification appears to adequately control relative compaction.

Table 25. Proposed Compaction Procedure Statistics.

		WR		QO	
		Avg	Individual	Avg	Individual
Single Target Density					
Percent Compaction	Avg.	96.8	97.0	96.1	96.3
	Std. Dev.	0.84	2.84	2.94	3.90
% Project Compacted	97-103	39	N/A	37	N/A
	95-105	N/A	75	N/A	62
Adjusted Target Density					
Percent Compaction	Avg.	100.1	100.4	99.7	100.0
	Std. Dev.	1.82	3.36	1.09	3.11
% Project Compacted	97-103	90	N/A	99	N/A
	95-105	N/A	86	N/A	90

N/A = Not applicable.

CHAPTER 9 – CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

Based on the data obtained and the limits of the materials and test methods evaluated in this study, the following conclusions are warranted.

- Difficulty was encountered performing AASHTO T 209 on WR samples due to excessive fines in the mix making the dry-back procedure impractical.
- Although still affected by water absorption, ASTM D6857 provided more reasonable maximum specific gravity results than AASHTO T 209 without the dry-back procedure.
- The existing pavements prior to in-place recycling were not uniform as shown by extracted aggregate gradations, back-calculated effective specific gravity values and extracted asphalt contents.
- The lack of uniformity of the existing pavements, contractor operations and construction temperatures contributed to the higher standard deviation of CIR mix properties compared to HMA values found in the literature. Standard deviations were 2-3 times higher for CIR compared to reported HMA values.
- The majority of 150 mm diameter molded CIR samples exceeded the threshold value of 2 percent water absorption of AASHTO T 166 and would require AASHTO T 331 or equivalent testing. A statistically significant difference was found between AASHTO T 166 and AASHTO T 331 bulk specific gravity values.
- The higher air voids calculated using AASHTO T 331 bulk specific gravities resulted in difficultly vacuum saturating samples to the 70-80 percent saturation level required in AASHTO T 283 as many of the AASHTO T 331 measured void spaces could not hold water for the SSD mass required for percent saturation calculations.
- Age and temperature of field produced mix had a significant effect on lab molded bulk specific gravity and many mix properties, excluding TSR. If field produced mix properties are used for control of CIR mixtures then compaction delay and mix temperature must be specified.
- The FLH AASHTO structural layer coefficient for CIR of 0.28 appears conservative. A value of 0.32 – 0.34 appears reasonable based on dynamic modulus values at 68^{o}F and 0.1 – 1.0 Hz.
- Based on the testing performed and data from the literature, the preliminary dynamic modulus values shown in Table 26 can be used for design until more data is available.
- The proposed mix design compactive effort of 35 gyrations in the SGC matched the few available core densities but was higher than reported nuclear gauge densities.
- Using percent relative compaction based on a target value of the maximum obtainable density from a control strip, if revised when conditions indicate, appears to adequately ensure average compaction to within 97-103 percent of the maximum obtainable density of the material.
- Additional test data, including cores form existing projects, would be required before recommendations could be made on the use of pay factors and statistical based specifications for control of CIR mixtures.

Table 26. Preliminary Average CIR Dynamic Modulus Values.

Temperature ^{o}C	Frequency		
	10 Hz.	1 Hz.	0.1 Hz.
4	1,200,000 psi	875,000 psi	630,000 psi
20	560,000 psi	375,000 psi	210,000 psi
35	280,000 psi	160,000 psi	80,000 psi

RECOMMENDATIONS

Based on the data obtained and the limits of the materials and test methods evaluated in this study, the following recommendations are warranted.

- Use AASHTO T 209 with the dry-back procedure or ASTM D6857 for determination of maximum specific gravity of CIR mixtures.
- Use of AASHTO T 166 will require the use of AASHTO T 331 in the majority of cases. When determining percent vacuum saturation for modified AASHTO T 283 testing use 55-75 percent saturation and AASHTO T 166 Method A, or AASHTO T 331 when required, for determination of bulk specific gravity
- For control of mix properties, compact field samples at in-situ temperatures within ± 30 minutes of the compaction delay between mixing and compaction used in the field. Keep field samples sealed and protected from excessive heat or cool prior to compaction.
- Use an AASHTO structural layer coefficient of 0.32-0.34 for structural design of CIR mixtures.
- Use the proposed CIR mix design method in Appendix B for CIR mixtures.
- Use the proposed FDR mix design method in Appendix C for FDR mixtures.
- Implement the specification changes and proposed compaction procedure outlined in Chapter 8.

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Appendix A

Field Test Data

Table A-1. RAP Gradation Analysis and Flat and Elongated Results, Washington Road.

Sta.	2+60	4+00	59+50	69+00	100+50	111+50	131+50	165+00	175+21	220+00	224+25	255+00
Lane	Left	Right	Right	Left	Right	Left	Left	Right	Left	Left	Right	Left
Sieve	Percent Passing											
1.5 inch	100	100	100	100	100	100	100	100	100	100	100	100
1 inch	98	100	100	94	100	97	99	97	95	96	99	99
3/4 inch	96	96	98	88	97	96	95	94	88	81	97	95
1/2 inch	85	86	90	75	88	85	83	84	70	52	88	85
3/8 inch	74	76	81	66	80	76	73	76	59	36	81	76
No. 4	43	28	54	42	57	47	46	56	35	14	58	52
No. 8	23	26	33	26	38	29	29	33	19	6.2	39	33
No. 16	12	16	17	14	26	18	18	21	10	3.4	23	19
No. 30	5.2	6.6	7.3	6.0	14	9.6	10	12	4.0	2.0	11	7.6
No. 50	2.3	2.4	2.8	2.3	5.5	4.1	4.5	5.2	1.4	1.1	4.5	2.5
No. 100	1.0	1.1	1.1	0.9	1.9	1.8	1.8	2.0	0.5	0.6	1.8	1.0
No. 200	0.5	0.5	0.5	0.4	0.8	0.8	0.8	0.8	0.2	0.3	0.8	0.4
5:1 F&E	4.8	0.7	0.9	0.4	3.6	2.5	0.9	0.9	1.3	1.7	N/A	0.4
3:1 F&E	45.4	42	37	28	28	35	40	32	40	34	N/A	32

Table A-2. Maximum Specific Gravity Results, Washington Road.

Sta.	Lane	Sample	Gmm	Avg Gmm
2+60	Left	A	2.471	
		B	2.473	2.472
4+00	Right	A	2.463	
		B	2.452	2.458
59+50	Right	A	2.482	
		B	2.471	2.476
69+00	Left	A	2.463	
		B	2.465	2.464
100+50	Right	A	2.481	
		B	2.474	2.478
111+50	Left	A	2.462	
		B	2.463	2.463
131+50	Left	A	2.494	
		B	2.496	2.495
165+00	Right	A	2.486	
		B	2.493	2.489
175+21	Left	A	2.442	
		B	2.446	2.444
220+00	Left	A	2.444	
		B	2.453	2.449
22425	Right	A	2.529	
(225+00)		B	2.526	2.528
255+00	Left	A	2.469	
		B	2.467	2.468

Table A-3. Asphalt Content and Recovered Gradation, Washington Road.

Sta.	2+60	4+00	59+50		69+00		100+50		111+50	
Lane	Left	Right	Right	Right	Left	Left	Right	Right	Left	Left
Sample	A	A	A	B	A	B	A	B	A	B
Sieve	Percent Passing									
2"										
1.5"						100	100			
1"	100			100		98	98	100		
3/4"	100	100	100	99	100	97	98	99	100	100
1/2"	98	99	98	97	95	91	95	97	99	99
3/8"	93	95	93	92	89	84	87	91	95	95
No. 4	68	74	73	73	67	63	67	72	74	74
No. 8	49	53	54	55	48	46	49	54	56	55
No. 16	37	38	39	40	36	34	37	41	43	42
No. 30	27	27	28	29	26	24	27	30	32	31
No. 50	17	17	18	19	16	15	16	18	20	19
No. 100	11	11	13	13	11	10	10	14	13	12
No. 200	7.5	7.3	8.5	8.5	6.9	6.5	6.8	7.9	8.5	8.5
AC (%)	6.4	6.8	6.4	6.2	6.3	6.5	5.6	5.7	6.2	6.3

Table A-3 (Con't.). Asphalt Content and Recovered Gradation, Washington Road.

								(225+00)	
Sta.	131+50		165+00	175+21		220+00		224+25	
Lane	Left	Left	Right	Left	Left	Left	Left	Right	Right
Sample	A	B	A	A	B	A	B	A	B
Sieve	Percent Passing								
2"						100			
1.5"		100	100			99			
1"		99	99			99		100	100
3/4"	100	98	98	100	100	99	100	99	99
1/2"	93	94	94	99	99	96	98	94	92
3/8"	87	88	89	93	92	88	89	88	88
No. 4	66	65	68	74	77	63	65	66	70
No. 8	47	47	50	54	57	46	47	46	52
No. 16	35	35	39	41	43	34	35	33	37
No. 30	26	26	29	30	32	25	26	24	26
No. 50	16	17	18	17	18	16	16	16	17
No. 100	10	11	12	11	12	11	11	11	11
No. 200	6.7	7.5	8.1	7.5	8.5	7.4	7.6	7.2	7.3
AC (%)	5.7	5.0	5.8	6.2	6.2	6.0	6.2	5.3	5.2

Table A-4. Bulk Specific Gravity, AASHTO T 283 Samples, Washington Road.

Sta.	Lane	Gmm	Sample	AASHTO T 166 Gmb	AASHTO T 166 Abs (%)	AASHTO T 166 VTM (%)	AASHTO T 331 Gmb	AASHTO T 331 VTM (%)
2+60	Left	2.472	2	2.067	2.3	16.4	2.050	17.1
2+60	Left	2.472	3	2.056	1.9	16.8	1.995	19.3
2+60	Left	2.472	4	2.066	2.0	16.4	2.052	17.0
4+00	Right	2.458	1	2.096	2.4	14.7	2.059	16.2
4+00	Right	2.458	2	2.090	1.5	15.0	2.073	15.7
4+00	Right	2.458	3	2.088	1.9	15.1	2.047	16.7
4+00	Right	2.458	4	2.091	1.9	14.9	2.066	16.0
4+00	Right	2.458	5	2.100	2.0	14.6	2.086	15.2
4+00	Right	2.458	6	2.099	2.7	14.6	2.077	15.5
59+50	Right	2.476	1	2.098	2.3	15.3	2.086	15.7
59+50	Right	2.476	2	2.100	2.1	15.2	2.087	15.7
59+50	Right	2.476	3	2.108	2.0	14.9	2.097	15.3
59+50	Right	2.476	4	2.100	3.0	15.2	2.070	16.4
59+50	Right	2.476	5	2.099	3.1	15.2	2.067	16.5
59+50	Right	2.476	6	*	*	*	2.072	16.3
69+00	Left	2.464	1	2.081	1.7	15.5	*	*
69+00	Left	2.464	2	2.088	2.4	15.3	2.056	16.5
69+00	Left	2.464	3	2.085	2.4	15.4	2.093	15.1
69+00	Left	2.464	4	*	*	*	2.070	16.0
69+00	Left	2.464	5	2.069	2.1	16.0	2.060	16.4
69+00	Left	2.464	6	2.087	2.0	15.3	2.065	16.2
100+50	Right	2.478	1	2.083	1.8	15.9	2.067	16.6
100+50	Right	2.478	2	2.085	2.3	15.9	2.072	16.4
100+50	Right	2.478	3	2.076	1.9	16.2	2.062	16.8
100+50	Right	2.478	4	2.099	1.7	15.3	2.090	15.6
100+50	Right	2.478	5	2.092	1.6	15.6	2.084	15.9
100+50	Right	2.478	6	2.076	2.2	16.2	2.067	16.6
111+50	Left	2.463	1	2.044	2.5	17.0	2.007	18.5
111+50	Left	2.463	2	2.062	1.8	16.3	2.029	17.6
111+50	Left	2.463	3	2.079	2.2	15.6	2.054	16.6
111+50	Left	2.463	4	2.072	3.0	15.9	2.076	15.7
111+50	Left	2.463	5	2.093	2.7	15.0	2.071	15.9
111+50	Left	2.463	6	2.097	2.5	14.8	2.080	15.5

* Not tested.

Table A-4 (Con't.). Bulk Specific Gravity, AASHTO T 283 Samples, Washington Road.

				AASHTO T 166			AASHTO T 331	
Sta.	Lane	Gmm	Sample	Gmb	Abs (%)	VTM (%)	Gmb	VTM (%)
131+50	Left	2.495	1	2.121	3.2	15.0	2.104	15.7
131+50	Left	2.495	2	2.102	1.8	15.8	2.095	16.0
131+50	Left	2.495	3	2.111	1.4	15.4	2.086	16.4
131+50	Left	2.495	4	2.112	1.6	15.3	2.106	15.6
131+50	Left	2.495	5	2.105	1.9	15.6	2.095	16.1
131+50	Left	2.495	6	2.116	1.9	15.2	2.097	15.9
175+21	Left	2.444	1	2.115	2.8	13.5	2.095	14.3
175+21	Left	2.444	2	2.123	2.6	13.1	2.113	13.6
175+21	Left	2.444	3	2.127	1.8	13.0	2.117	13.4
175+21	Left	2.444	4	2.121	2.2	13.2	2.072	15.2
175+21	Left	2.444	5	2.134	2.7	12.7	2.114	13.5
175+21	Left	2.444	6	2.121	2.1	13.2	2.099	14.1
220+00	Left	2.449	1	2.129	3.1	13.1	2.120	13.4
220+00	Left	2.449	2	2.123	3.5	13.3	2.102	14.2
220+00	Left	2.449	3	2.118	3.1	13.5	2.107	14.0
220+00	Left	2.449	4	2.118	1.6	13.5	2.111	13.8
220+00	Left	2.449	5	2.119	2.6	13.5	2.105	14.1
220+00	Left	2.449	6	2.130	1.9	13.0	2.108	13.9
225+00	Right	2.528	1	2.051	3.2	18.9	2.014	20.3
225+00	Right	2.528	2	2.047	3.8	19.0	1.997	21.0
225+00	Right	2.528	3	2.049	4.1	19.0	2.010	20.5
225+00	Right	2.528	4	2.054	2.5	18.7	2.025	19.9
225+00	Right	2.528	5	2.054	2.3	18.7	2.028	19.8
225+00	Right	2.528	6	2.059	2.7	18.6	2.038	19.4
255+00	Left	2.468	1	2.116	1.8	14.3	2.097	15.0
255+00	Left	2.468	2	2.102	3.0	14.8	2.081	15.7
255+00	Left	2.468	3	2.105	3.1	14.7	2.074	15.9
255+00	Left	2.468	4	2.122	2.0	14.0	2.099	14.9
255+00	Left	2.468	5	2.110	2.9	14.5	2.086	15.5
255+00	Left	2.468	6	2.112	2.2	14.4	2.091	15.3

Table A-5. AASHTO T 283 Results, Washington Road.

Sta.	Lane	Sample	Cond	T 166 Gmb	T 331 Gmb	Gmm	VTM (%)	% Sat (%)	Avg Sat. (%)	Ht (mm)	Tensile Load (lbs)	Tensile Strength Sat. (psi)	Tensile Strength Dry (psi)	TSR	Avg Density (pcf)	Avg VTM (%)
2+60	Left	2	S	2.067	2.050	2.472	17.1	70.3		95.0	800	23.1				
2+60	Left	3	D	2.056		2.472	16.8			85.9	2100		66.9			
2+60	Left	4	S	2.066		2.472	16.4	70.3	70.3	96.8	930	26.3		0.37	128.4	16.8
4+00	Right	1	D	2.096	2.059	2.458	16.2			97.0	1830		51.7			
4+00	Right	2	D	2.090		2.458	15.0			94.8	1870		54.0			
4+00	Right	3	S	2.088		2.458	15.1	70.1		94.5	520	15.1				
4+00	Right	4	D	2.091		2.458	14.9			94.0	1900		55.3			
4+00	Right	5	S	2.100		2.458	14.6	74.4		96.5	990	28.1				
4+00	Right	6	S	2.099	2.077	2.458	15.5	71.3	72.0	96.5	990	28.1		0.44	130.1	15.2
59+50	Right	1	S	2.098	2.086	2.476	15.8	70.3		96.2	1070	30.5				
59+50	Right	2	S	2.100	2.087	2.476	15.7	71.9		96.5	1090	30.9				
59+50	Right	3	D	2.108	2.097	2.476	15.3			96.3	2050		58.3			
59+50	Right	4	D	2.100	2.070	2.476	16.4			96.2	1710		48.7			
59+50	Right	5	S	2.099	2.067	2.476	16.5	74.9		97.3	1010	28.4				
59+50	Right	6	D		2.072	2.476	16.3		72.3	95.7	1820		52.1	0.56	129.8	16.0
69+00	Left	1	D	2.081		2.464	15.5			98.2	1860		51.9			
69+00	Left	2	S	2.088	2.056	2.464	16.6	70.3		96.9	690	19.5				
69+00	Left	3	S	2.085	2.093	2.464	15.1	71.2		98.3	910	25.3				
69+00	Left	4	S	2.094	2.070	2.464	16.0	74.0		97.1	600	16.9				
69+00	Left	5	D	2.069	2.060	2.464	16.4			94.8	1790		51.7			
69+00	Left	6	D	2.087	2.065	2.464	16.2		71.8	97.6	1820		51.1	0.40	129.2	16.0

Table A-5 (Con't.). AASHTO T 283 Results, Washington Road.

Sta.	Lane	Sample	Cond	T 166 Gmb	T 331 Gmb	Gmm	VTM (%)	% Sat (%)	Avg Sat. (%)	Ht (mm)	Tensile Load (lbs)	Tensile Strength Sat. (psi)	Tensile Strength Dry (psi)	TSR	Avg Density (pcf)	Avg VTM (%)
100+50	Right	1	S	2.083		2.478	15.9	78.5		96.4	730	20.7				
100+50	Right	2	D	2.085	2.072	2.478	16.4			96.0	1660		47.3			
100+50	Right	3	S	2.076		2.478	16.2	76.2		96.4	750	21.3				
100+50	Right	4	D	2.099		2.478	15.3			95.1	1860		53.6			
100+50	Right	5	S	2.092		2.478	15.6	71.4		95.4	810	23.2				
100+50	Right	6	D	2.076	2.067	2.478	16.6		75.4	96.8	1830		51.8	0.43	129.9	16.0
111+50	Left	1	D	2.046	2.007	2.463	18.5			98.5	1830		50.9			
111+50	Left	2	D	2.062		2.463	16.3			86.8	1970		62.1			
111+50	Left	3	S	2.079	2.054	2.463	16.6	73.6		98.4	1190	33.1				
111+50	Left	4	S	2.072	2.076	2.463	15.7	75.3		97.8	580	16.2				
111+50	Left	5	S	2.093	2.071	2.463	15.9	71.7		98.2	1090	30.4				
111+50	Left	6	D	2.097	2.080	2.463	15.6		73.5	97.5	2260		63.5	0.45	128.4	16.4
131+50	Left	1	D	2.121	2.104	2.495	15.7			98.5	1370		38.1			
131+50	Left	2	D	2.102		2.495	15.8			86.8	1450		45.7			
131+50	Left	3	D	2.111		2.495	15.4			98.4	1310		36.5			
131+50	Left	4	S	2.112		2.495	15.3	70.1		97.8	610	17.1				
131+50	Left	5	S	2.105		2.495	15.6	73.1		98.2	480	13.4				
131+50	Left	6	S	2.116		2.495	15.2	71.4	71.6	97.5	490	13.8		0.37	131.6	15.5
175+21	Left	1	S	2.115	2.095	2.444	14.3	79.0		98.2	780	21.7				
175+21	Left	2	S	2.123	2.113	2.444	13.5	77.5		96.9	840	23.7				
175+21	Left	3	D	2.127		2.444	13.0			98.3	1640		45.7			
175+21	Left	4	S	2.121	2.072	2.444	15.2	70.5		97.1	740	20.9				
175+21	Left	5	D	2.128	2.114	2.444	13.5			94.8	1630		47.1			
175+21	Left	6	D	2.121	2.009	2.444	17.8		75.7	97.6	1490		41.8	0.49	130.3	14.5

Table A-5 (Con't.). AASHTO T 283 Results, Washington Road.

Sta.	Lane	Sample	Cond	T 166 Gmb	T 331 Gmb	Gmm	VTM (%)	% Sat (%)	Avg Sat. (%)	Ht (mm)	Tensile Load (lbs)	Tensile Strength Sat. (psi)	Tensile Strength Dry (psi)	TSR	Avg Density (pcf)	Avg VTM (%)
220+00	Left	1	S	2.129	2.120	2.449	13.4	81.0		95.5	980	28.1				
220+00	Left	2	D	2.123	2.102	2.449	14.2			96.2	1320		37.6			
220+00	Left	3	S	2.118	2.107	2.449	14.0	74.2		96.1	900	25.6				
220+00	Left	4	S	2.118		2.449	13.5	75.5		94.6	960	27.8				
220+00	Left	5	D	2.119	2.105	2.449	14.0			95.9	1510		43.1			
220+00	Left	6	D	2.130		2.449	13.0		76.9	95.8	1510		43.2	0.66	131.9	13.7
225+00	Right	1	S	2.051	2.014	2.528	18.9	72.9		99.0	710	19.6				
225+00	Right	2	D	2.047	1.997	2.528	21.0			99.6	960		26.4			
225+00	Right	3	S	2.049	2.010	2.528	19.0	71.5		98.8	500	13.9				
225+00	Right	4	S	2.054	2.025	2.528	18.7	70.1		98.4	590	16.4				
225+00	Right	5	D	2.054	2.028	2.528	18.7			98.9	1080		29.9			
225+00	Right	6	D	2.059	2.038	2.528	19.4		71.5	98.1	1070		29.9	0.58	127.3	19.3
255+00	Left	1	S	2.116		2.468	14.3	70.2		95.8	1450	41.4				
255+00	Left	2	D	2.102	2.081	2.468	15.7			96.9	2360		66.7			
255+00	Left	3	S	2.105	2.074	2.468	16.0	70.3		97.1	1280	36.1				
255+00	Left	4	D	2.122	2.099	2.468	15.0			96.3	2300		65.4			
255+00	Left	5	S	2.110	2.086	2.468	15.5	71.8		96.4	1070	30.4				
255+00	Left	6	D	2.112	2.091	2.468	15.3		70.7	95.8	2430		69.5	0.54	130.5	15.3

Table A-6. Retained Marshall Stability Results, Washington Road.

Sta.	Lane	Sample	Cond	Gmb	Gmm	VTM (%)	% Sat	Avg Sat (%)	Ht (mm)	Marshall Load (lbs)	CF	Marshall Stability Sat (lbs)	Marshall Stability Dry (lbs)	Ret. Stab.	Avg Density (pcf)	Avg VTM (%)
2+60	Left	1	D	2.018	2.472	18.4			61.3	2050	1.00		2050			
2+60	Left	2	S	2.036	2.472	17.6	62.6		48.8	1090	1.56	1700				
2+60	Left	3	D	2.064	2.472	16.5			66.3	3760	0.93		3497			
2+60	Left	4	S	2.061	2.472	16.6	62.7	62.7	62.3	2020	1.04	2101		0.69	127.6	17.3
4+00	Right	1	D	2.085	2.458	15.2			52.4	3340	1.39		4643			
4+00	Right	2	S	2.124	2.458	13.6	62.7		48.7	1530	1.56	2387				
4+00	Right	3	S	2.064	2.458	16.0	63.4		57.2	1330	1.19	1583				
4+00	Right	4	S	2.126	2.458	13.5	61.3		52.6	1570	1.39	2182				
4+00	Right	5	D	2.119	2.458	13.8			59.9	4650	1.09		5069			
4+00	Right	6	D	2.115	2.458	13.9		62.5	53.4	3920	1.32		5174	0.41	131.4	14.3
59+50	Right	1	S	2.056	2.476	17.0	68.2		55.0	1250	1.25	1563				
59+50	Right	2	S	2.112	2.476	14.7	58.2		60.4	1850	1.09	2017				
59+50	Right	3	S	2.101	2.476	15.1	58.1		54.8	1450	1.25	1813				
59+50	Right	4	D	2.098	2.476	15.3			59.0	4070	1.14		4640			
59+50	Right	5	D	2.124	2.476	14.2			53.0	3820	1.39		5310			
59+50	Right	6	D	2.054	2.476	17.1		61.5	56.5	2260	1.19		2689	0.43	130.5	15.6
69+00	Left	1	D	2.039	2.464	17.2			53.2	1850	1.39		2572			
69+00	Left	2	S	2.030	2.464	17.6	68.1		56.1	850	1.25	1063				
69+00	Left	3	S	2.089	2.464	15.2	65.7		47.5	1290	1.67	2154				
69+00	Left	4	D	2.046	2.464	16.9			48.7	2360	1.56		3682			
69+00	Left	5	S	2.081	2.464	15.5	66.4	66.8	53.0	1280	1.39	1779				
69+00	Left	6	D	2.088	2.464	15.2			55.2	3140	1.25		3925	0.49	128.7	16.3
100+50	Right	1	S	2.046	2.478	17.4	59.3		57.7	1220	1.19	1452				
100+50	Right	2	S	2.081	2.478	16.0	55.0		54.7	1210	1.32	1597				
100+50	Right	3	S	2.069	2.478	16.5	55.6		51.1	1010	1.47	1485				
100+50	Right	4	D	2.056	2.478	17.0			53.9	1810	1.32		2389			
100+50	Right	5	D	2.052	2.478	17.2			58.8	1950	1.14		2223			
100+50	Right	6	D	2.025	2.478	18.3		56.6	58.8	1450	1.14		1653	0.72	128.2	17.1

Table A-6 (Con't.). Retained Marshall Stability Results, Washington Road.

Sta.	Lane	Sample	Cond	Gmb	Gmm	VTM (%)	% Sat	Avg Sat (%)	Ht (mm)	Marshall Load (lbs)	CF	Marshall Stability Sat (lbs)	Marshall Stability Dry (lbs)	Ret. Stab.	Avg Density (pcf)	Avg VTM (%)
111+50	Left	1	D	2.046	2.463	16.9			58.6	2650	1.14		3021			
111+50	Left	2	D	2.090	2.463	15.1			56.3	3310	1.25		4138			
111+50	Left	3	S	2.088	2.463	15.2	57.9		55.4	1560	1.25	1950				
111+50	Left	4	D	2.057	2.463	16.5			56.2	2680	1.25		3350			
111+50	Left	5	S	2.087	2.463	15.3	61.6		56.5	1570	1.19	1868				
111+50	Left	6	S	2.045	2.463	17.0	62.7	60.7	53.8	1100	1.32	1452		0.50	129.1	16.0
131+50	Left	1	S	2.042	2.495	18.1	59.0		57.2	770	1.19	916				
131+50	Left	2	S	2.047	2.495	18.0	60.4		54.3	740	1.32	977				
131+50	Left	3	D	2.110	2.495	15.4			58.5	2100	1.14		2394			
131+50	Left	4	D	2.029	2.495	18.7		60.4	51.6	3340	1.25		4175			
131+50	Left	5	S	2.062	2.495	17.3	60.3	60.3	57.2	780	1.19	928				
131+50	Left	6	D	2.064	2.495	17.3		60.3	53.6	1220	1.32		1610	0.34	128.5	17.5
165+00	Right	1	D	2.101	2.489	15.6			59.0	2890	1.14		3295			
165+00	Right	2	S	2.092	2.489	16.0	59.4		58.8	1370	1.14	1562				
165+00	Right	3	D	2.117	2.489	14.9			49.8	2530	1.56		3947			
165+00	Right	4	S	2.090	2.489	16.0	55.3		51.3	920	1.47	1352				
165+00	Right	5	D	2.072	2.489	16.7			50.5	1540	1.47		2264			
165+00	Right	6	S	2.123	2.489	14.7	59.6	58.1	55.9	1070	1.25	1338		0.45	131.0	15.7
175+21	Left	1	D	2.055	2.444	15.9			56.8	1120	1.19		1333			
175+21	Left	2	D	2.049	2.444	16.2			57.2	1030	1.19		1226			
175+21	Left	3	S	2.069	2.444	15.3	60.4		53.8	810	1.32	1069				
175+21	Left	4	S	2.039	2.444	16.6	61.8		60.2	920	1.09	1003				
175+21	Left	5	S	2.056	2.444	15.9	59.0		53.8	740	1.32	977				
175+21	Left	6	D	2.053	2.444	16.0		60.4	53.4	1230	1.32		1624	0.73	128.1	16.0

Table A-6 (Con't.). Retained Marshall Stability Results, Washington Road.

Sta.	Lane	Sample	Cond	Gmb	Gmm	VTM (%)	% Sat	Avg Sat (%)	Ht (mm)	Marshall Load (lbs)	CF	Marshall Stability Sat (lbs)	Marshall Stability Dry (lbs)	Ret. Stab.	Avg Density (pcf)	Avg VTM (%)
220+00	Left	1	D	2.039	2.449	16.7			51.5	920	1.47		1352			
220+00	Left	2	S	2.037	2.449	16.8	57.6		53.4	570	1.32	752				
220+00	Left	3	S	2.075	2.449	15.3	58.7		54.3	700	1.32	924				
220+00	Left	4	D	2.056	2.449	16.0			58.6	1250	1.14		1425			
220+00	Left	5	D	2.080	2.449	15.1			58.0	1500	1.14		1710			
220+00	Left	6	S	2.082	2.449	15.0	61.3	59.2	53.0	760	1.32	1003		0.60	128.6	15.8
225+00	Right	1	D	2.102	2.528	16.9			52.2	1670	1.39		2321			
225+00	Right	2	S	2.055	2.528	18.7	56.0		54.2	830	1.32	1096				
225+00	Right	3	D	2.114	2.528	16.4			49.2	2210	1.56		3448			
225+00	Right	4	S	2.137	2.528	15.5	55.8		54.9	1220	1.25	1525				
225+00	Right	5	S	2.079	2.528	17.8	59.5		51.0	1010	1.47	1485				
225+00	Right	6	D	2.069	2.528	18.2		57.1	51.0	1320	1.47		1940	0.53	130.6	17.2
255+00	Left	1	D	2.193	2.468	11.2			58.0	2250	1.14		2565			
255+00	Left	2	S	2.167	2.468	12.2	57.5		56.0	1070	1.25	1338				
255+00	Left	3	S	2.183	2.468	11.5	65.0		50.2	780	1.47	1147				
255+00	Left	4	D	2.050	2.468	16.9			52.6	820	1.39		1140			
255+00	Left	5	S	2.169	2.468	12.1	57.0		55.2	1060	1.25	1325				
255+00	Left	6	D	2.166	2.468	12.2		59.8	55.9	2140	1.25		2675	0.60	134.4	12.7

Table A-7. Bulk Specific Gravity Results, E* Samples, Washington Road.

			Before Sawing & Coring					After Sawing & Coring				
			AASHTO T 166			AASHTO T 331		AASHTO T 166			AASHTO T 331	
Lane	Sample	Gmm	Gmb	Abs (%)	VTM (%)	Gmb	VTM	Gmb	Abs (%)	VTM (%)	Gmb	VTM (%)
Left	1	2.472	2.054	2.5	16.9	2.043	17.4	2.131	0.9	13.8	2.092	15.4
Right	1	2.458	*	*	*	1.999	18.7	2.056	2.4	16.3	2.070	15.8
Right	2	2.458	*	*	*	2.067	15.9	*	*	*	2.067	15.9
Right	3	2.458	2.056	2.4	16.3	2.070	15.8	*	*	*	1.999	18.7
Right	1	2.476	*	*	*	2.046	17.4	2.129	1.2	14.0	2.185	11.8
Right	2	2.476	2.062	2.7	16.7	2.043	17.5	2.118	1.8	14.5	2.102	15.1
Right	3	2.476	*	*	*	2.038	17.7	2.132	2.1	13.9	2.185	11.8
Left	1	2.464	*	*	*	1.963	20.3	*	*	*	1.963	20.3
Left	2	2.464	2.064	3.6	16.2	2.032	17.5	2.064	3.6	16.2	2.032	17.5
Left	3	2.464	*	*	*	1.981	19.6	*	*	*	1.981	19.6
Right	1	2.478	2.091	2.0	15.6	2.066	16.6	2.129	0.7	14.1	2.185	11.8
Right	2	2.478	2.096	1.1	15.4	2.185	11.8	2.124	0.7	14.3	2.185	11.8
Right	3	2.478	2.091	1.6	15.6	2.185	11.8	2.123	0.9	14.3	2.185	11.8
Left	1	2.463	2.064	2.4	16.2	1.924	21.9	2.110	0.8	14.3	2.080	15.5
Left	2	2.463	*	*	*	2.048	16.8	2.141	2.0	13.1	2.185	11.3
Left	3	2.463	*	*	*	2.061	16.3	2.031	0.7	17.5	2.185	11.3
Left	1	2.495	2.102	1.7	15.8	2.185	12.4	2.132	1.0	14.5	2.185	12.4
Left	2	2.495	2.091	1.8	16.2	2.057	17.6	2.140	0.8	14.2	2.185	12.4
Left	3	2.495	2.092	2.2	16.1	2.185	12.4	2.149	0.9	13.9	2.185	12.4
Left	1	2.444	*	*	*	2.088	14.5	2.161	1.3	11.6	2.140	12.5
Left	2	2.444	2.126	2.8	13.0	2.105	13.9	2.166	0.8	11.4	2.185	10.6
Left	3	2.444	*	*	*	2.100	14.1	2.163	0.9	11.5	2.185	10.6
Left	1	2.449	2.104	3.4	14.1	2.072	15.4	*	*	*	2.140	12.6
Left	2	2.449	*	*	*	1.891	22.8	1.932	4.4	21.1	1.928	21.3
Left	3	2.449	*	*	*	2.107	13.9	*	*	*	2.155	12.0
Right	3	2.528	*	*	*	1.902	24.8	2.106	1.3	16.7	2.083	17.6
Left	1	2.468	2.116	2.6	14.3	2.115	14.3	2.150	1.2	12.9	2.138	13.4
Left	2	2.468	2.203	1.1	10.7	2.164	12.3	2.241	0.5	9.2	2.185	11.5
Left	3	2.468	2.196	1.1	11.0	2.162	12.4	2.229	0.6	9.7	2.185	11.5

* Not tested.

Table A-8. Dynamic Modulus Results, Washington Road.

Station	Lane	Sample	VTM (%)	Method	Test Temp. C	Dynamic Modulus (psi) Test Frequency 10 Hz	5 Hz	1 Hz	0.5 Hz	0.1 Hz
2+60	Left	1	15.4	T 331	4	1,049,457	970,153	764,749	685,920	532,494
					20	616,535	562,820	383,174	319,701	218,484
					35	271,394	236,682	151,802	127,361	90,599
4+00	Right	1	15.8	T 331	4	1,200,982	1,075,741	823,146	748,932	613,160
					20	Sample failed during testing				
		2	15.9	T 331	4	1,107,520	956,282	670,699	602,661	460,078
					20	Sample failed during testing				
		3	18.7	T 331	4	Sample failed during testing				
59+50	Right	1	14.0	T 166	4	1,595,344	1,157,384	985,705	838,726	579,476
					20	407,205	372,607	260,470	223,746	167,192
					35	303,712	264,481	166,319	138,836	95,695
		2	14.5	T 166	4	1,523,073	1,391,852	1,029,650	962,561	628,017
					20	758,889	576,960	392,848	318,669	179,409
					35	184,206	152,968	88,228	70,609	54,590
		3	11.8	T 331	4	1,412,085	1,322,957	1,048,687	935,777	700,161
					20	507,792	338,136	307,817	255,090	173,689
					35	235,780	203,593	115,060	90,842	63,892
69+00	Left	1	20.3	T 331	4	1,437,407	1,108,805	888,544	820,030	689,186
					20	503,889	401,831	239,343	208,639	134,201
					35	Sample failed during testing				
		2	17.5	T 331	4	1,239,052	1,090,756	811,573	688,642	465,636
					20	435,066	422,672	301,102	266,679	208,620
					35	321,470	279,663	171,263	150,431	113,920
		3	19.6	T 331	4	1,598,430	1,285,510	983,861	836,056	789,556
					20	783,699	717,133	656,522	608,136	589,794
					35	392,115	363,871	226,846	192,503	142,587

Table A-8 (Con't.). Dynamic Modulus Results, Washington Road.

Station	Lane	Sample	VTM (%)	Method	Test Temp. C	Dynamic Modulus (psi) Test Frequency 10 Hz	5 Hz	1 Hz	0.5 Hz	0.1 Hz
100+50	Right	1	14.1	T 166	4	843,800	768,209	567,310	488,479	377,053
					20	436,626	395,508	295,566	242,265	191,703
					35	305,004	280,406	179,599	163,863	128,062
		2	14.3	T 166	4	1,234,527	1,187,033	930,447	841,667	646,337
					20	568,658	527,227	351,793	301,136	210,748
					35	343,107	326,004	194,141	108,251	79,023
		3	14.3	T 166	4	1,114,628	1,034,055	782,327	670,866	544,275
					20	594,339	534,954	358,960	311,756	229,869
					35	308,191	272,444	173,628	147,293	111,533
111+50	Left	1	14.3	T 166	4	Readings not recorded				
					20	405,453	372,346	270,491	239,308	181,149
					35	290,405	261,870	167,053	139,896	106,268
		2	13.1	T 166	4	811,728	762,826	654,635	570,620	493,299
					20	566,948	499,598	357,080	298,141	202,194
					35	337,358	288,985	191,862	163,127	124,383
		3	17.5	T 166	4	1,424,426	1,287,521	1,023,636	954,973	774,735
					20	532,253	480,841	341,736	300,828	219,587
					35	413,382	368,284	249,923	206,506	138,132
131+50	Left	1	14.5	T 166	4	790,910	716,513	551,058	496,400	397,949
					20	449,740	410,676	262,880	226,278	163,927
					35	222,024	195,152	99,623	84,033	61,120
		2	14.2	T 166	4	1,308,024	1,047,889	750,725	679,673	580,692
					20	420,468	375,514	234,008	196,411	144,079
					35	325,989	303,127	159,602	136,046	88,089
		3	13.9	T 166	4	867,235	774,262	573,044	503,150	394,372
					20	441,404	400,374	281,192	253,437	185,837
					35	291,588	279,225	156,082	143,799	117,195

Table A-8 (Con't.). Dynamic Modulus Results, Washington Road.

Station	Lane	Sample	VTM (%)	Method	Test Temp. C	Dynamic Modulus (psi) Test Frequency 10 Hz	5 Hz	1 Hz	0.5 Hz	0.1 Hz
					4	1,011,871	896,638	719,016	606,653	493,264
		1	11.6	T 166	20	507,555	450,440	301,633	256,932	187,542
					35	262,152	208,105	124,713	110,434	94,935
					4	1,400,843	1,303,723	1,022,906	915,695	708,671
175+21	Left	2	11.4	T 166	20	555,718	480,575	305,648	254,025	170,879
					35	339,558	279,274	164,849	143,272	117,642
					4	1,467,422	1,346,479	1,015,698	846,850	770,204
		3	11.5	T 166	20	422,102	379,249	252,421	217,180	157,783
					35	253,839	222,941	124,719	106,826	80,406
					4	1,648,297	1,481,151	1,113,535	988,045	723,029
		1	12.6	T 331	20	615,226	533,302	340,680	280,333	181,300
					35	311,350	253,842	130,393	101,517	62,979
					4	1,986,378	1,459,703	1,253,323	1,079,193	712,306
220+00	Left	2	21.3	T 331	20	374,485	328,984	176,445	136,244	83,162
					35	176,132	134,723	97,159	64,691	50,875
					4	1,555,996	1,375,608	1,054,014	944,449	692,225
		3	12.0	T 331	20	581,786	514,799	343,187	275,987	164,212
					35	314,537	286,120	192,693	163,026	117,114
225+00	Right				4	1,216,978	895,125	766,896	618,973	427,996
		3	16.7	T 166	20	551,297	499,968	380,067	314,888	191,699
					35	265,300	206,952	112,137	90,939	64,559
					4	1,298,685	1,015,660	838,256	723,954	611,603
		1	12.9	T 166	20	532,569	354,395	286,648	177,069	61,265
					35	275,230	224,040	118,271	92,050	58,699
					4	1,234,442	1,175,857	706,590	620,552	463,235
255+50	Left	2	9.2	T 166	20	545,207	397,054	292,334	213,173	137,726
					35	216,142	178,875	92,322	73,986	51,207
					4	980,053	873,250	629,734	538,034	358,462
		3	9.7	T 166	20	303,548	263,177	157,987	135,172	104,342
					35	187,346	150,793	88,273	73,600	57,543

Table A-9. Bulk Specific Gravity, Lab Molded Samples, Washington Road.

Sta.	Lane	Gmm	Sample	AASHTO T 166 Gmb	AASHTO T 166 Abs (%)	AASHTO T 166 VTM (%)	AASHTO T 331 Gmb	AASHTO T 331 VTM (%)
2+60	Left	2.472	1	2.054	2.5	16.9	2.043	17.4
4+00	Right	2.458	1	2.056	2.4	16.3	2.070	15.8
4+00	Right	2.458	2	*	*	*	1.999	18.7
4+00	Right	2.458	3	*	*	*	2.067	15.9
59+50	Right	2.476	1	2.062	2.7	16.7	2.043	17.5
59+50	Right	2.476	2	*	*	*	2.046	17.4
59+50	Right	2.476	3	*	*	*	2.038	17.7
69+00	Left	2.464	1	2.064	3.6	16.2	2.032	17.5
69+00	Left	2.464	2	*	*	*	1.963	20.3
69+00	Left	2.464	3	*	*	*	1.981	19.6
100+50	Right	2.478	1	2.091	2.0	15.6	2.066	16.6
100+50	Right	2.478	2	2.096	1.1	15.4	2.185	11.8
100+50	Right	2.478	3	2.091	1.6	15.6	2.185	11.8
111+50	Left	2.463	1	2.064	2.4	16.2	1.924	21.9
111+50	Left	2.463	2	*	*	*	2.048	16.8
111+50	Left	2.463	3	*	*	*	2.061	16.3
131+50	Left	2.495	1	2.102	1.7	15.8	2.185	12.4
131+50	Left	2.495	2	2.091	1.8	16.2	2.057	17.6
131+50	Left	2.495	3	2.092	2.2	16.1	2.185	12.4
175+21	Left	2.444	1	2.126	2.8	13.0	2.105	13.9
175+21	Left	2.444	2	*	*	*	2.088	14.5
175+21	Left	2.444	3	*	*	*	2.100	14.1
220+00	Left	2.449	1	2.104	3.4	14.1	2.072	15.4
220+00	Left	2.449	2	*	*	*	1.891	22.8
220+00	Left	2.449	3	*	*	*	2.107	13.9
225+00	Right	2.528	1	2.007	4.6	20.6	1.970	22.1
225+00	Right	2.528	2	*	*	*	1.950	22.9
225+00	Right	2.528	3	*	*	*	1.902	24.8
255+00	Left	2.468	1	2.116	2.6	14.3	2.115	14.3
255+00	Left	2.468	2	2.203	1.1	10.7	2.164	12.3
255+00	Left	2.468	3	2.196	1.1	11.0	2.162	12.4

* Not tested.

Table A-10. QA Data, Washington Road.

	Station	Offset	Density Tests (Gmb)			%Comp.	Microwave BC	Microwave AC	Oven BC
Recycled 9/25/09-Up Hill									
A	4+00	L7	1.888	1.888	1.925	93.2%	4.10%	3.16%	3.70%
B	16+00	L8	1.887	1.860	1.875	91.9%	4.80%	3.57%	4.92%
C	33+50	L7	1.928	1.898	1.911	93.7%	3.80%	2.15%	3.01%
D	51+50	L5	1.836	1.846	1.843	90.3%	3.22%	2.92%	2.10%
E	59+75		1.700	1.810	1.784	86.5%			
F	59+75R		1.830	1.842	1.789	89.2%	3.48%	2.44%	2.55%
G	71+50		1.809	1.839	1.780	88.7%	3.46%	4.48%	2.29%
H	98+50		1.900	1.909	1.866	92.7%	3.95%	3.62%	1.59%
I	108+00		1.816	1.798	1.808	88.6%	3.66%	2.94%	3.36%
J	121+00		1.927	1.900	1.842	92.6%	3.61%	2.27%	1.57%
Recycled 9/26/09-Up Hill									
A	141+50	L4	1.906	1.916	1.913	93.7%	4.78%	3.92%	2.88%
B	156+30	L4	1.810	1.814	1.825	89.0%	2.64%	3.32%	2.40%
C	172+00	L5	1.669	1.710	1.731	83.5%	2.74%	2.64%	2.57%
D	187+25	L5	1.775	1.819	1.816	88.4%	3.51%	2.83%	2.74%
E	172+00R	L5	1.799	1.818	1.853	89.4%		See Above	
F	215+75	L5	1.830	1.851	1.861	90.6%	3.94%	2.54%	3.22%
G	231+00	L5	1.992	1.958	1.991	97.1%	3.96%	4.25%	2.33%
H	246+00	L4	1.809	1.829	1.830	89.3%	3.23%	2.53%	2.69%
I	261+00	L5	1.805	1.818	1.795	88.5%	3.49%	1.93%	3.43%
J	201+50	L6	1.820	1.813	1.808	88.9%	3.69%	1.33%	2.23%
k	276+00	L4	1.819	1.832	1.817	89.3%	3.25%	1.24%	2.97%

BC= before cure, AC = after cure.

Table A-10 (Con't.). QA Data, Washington Road.

	Station	Offset	Density Tests (Gmb)			%Comp.	Microwave BC	Microwave AC	Oven BC
Recycled 9/28/09-Down Hill									
A	1+83	R7	1.853	1.865	1.873	91.4%	2.06%	2.39%	1.93%
B	15+25	R5	1.811	1.801	1.801	88.4%	4.05%	2.25%	1.78%
C	31+50	R4	1.855	1.831	1.802	89.7%	3.80%	2.27%	2.15%
D	48+00	R5	1.814	1.789	1.817	88.6%	4.47%	1.43%	3.24%
E	61+40	R6	1.837	1.820	1.795	89.1%	1.43%	1.85%	3.31%
F	78+00	R7	1.861	1.862	1.853	91.1%	2.27%	1.95%	2.57%
G	106+00	NA	1.865	1.813	1.802	89.5%	2.53%	3.07%	1.46%
Recycled 9/29/09-Down Hill									
A	130+53		2.133	2.121	2.142	104.5%	3.69%	1.75%	4.75%
B	136+50		1.927	1.924	1.926	94.4%	3.94%	3.89%	4.44%
Recycled 9/30/09-Down Hill									
A	150+00	R5	1.820	1.846	1.838	89.9%	3.60%	3.79%	2.36%
B	166+00	R5	1.882	1.898	1.903	92.9%	2.59%	3.89%	2.53%
C	180+00	R5	1.932	1.973	1.930	95.3%	3.40%	4.56%	2.43%
D	185+50	R5	1.851	1.834	1.782	89.3%	3.15%	4.65%	1.62%
E	196+50	R5	1.792	1.728	1.757	86.2%			
F	196+50R	R5	1.813	1.809	1.797	88.5%	2.94%	4.28%	3.38%
G	211+00	R5	1.807	1.810	1.835	89.1%	1.44%	4.13%	1.54%
H	226+00	R5	1.837	1.803	1.811	89.1%	1.61%	4.67%	1.20%
I	241+00	R5	1.831	1.817	1.828	89.5%	1.07%	2.13%	2.07%
J	256+00	R5	1.844	1.826	1.856	90.3%	1.03%	1.52%	2.00%
K	271+00	R5	1.914	1.889	1.818	91.8%	1.63%	2.30%	2.84%

BC= before cure, AC = after cure.

Table A-11. RAP Gradation Analysis, Quincy-Oroville Road.

Sta.	222+45	260+00	299+00	201+00	194+00	477+00	644+00	655+00	598+00	602+50	660+26
Lane	Right	Right	Left	Left	Left	Right	Right	Right	Left	Left	Left
Sieve	Percent Passing										
1.5 inch	100	100	100	100	100	100	100	100	100	100	100
1 inch	95	97	96	94	97	96	96	97	99	97	97
3/4 inch	91	93	91	88	93	87	91	90	87	90	89
1/2 inch	76	82	76	72	81	68	76	78	68	78	73
3/8 inch	63	72	61	58	67	55	63	64	53	68	59
N0. 4	33	40	27	29	33	24	29	33	24	37	29
N0. 8	13	18	8.2	11	12	7.0	8.8	11	8.4	14	11
No. 16	3.5	5.2	1.7	2.7	2.7	1.3	1.6	2.3	2.0	3.1	2.7
No. 30	0.9	1.2	0.4	0.8	0.6	0.3	0.3	0.4	0.4	0.6	0.6
No. 50	0.3	0.3	0.2	0.3	0.2	0.1	0.1	0.1	0.1	0.1	0.2
No. 100	0.1	0.1	0.1	0.1	0.1	0.0	0.0	0.0	0.1	0.1	0.1
No. 200	0.1	0.1	0.1	0.1	0.1	0.0	0.0	0.0	0.1	0.0	0.0

Table A-12. Maximum Specific Gravity Results, Quincy-Oroville Road.

			AASHTO T 209		ASTM D 6857	
Sta.	Lane	Sample	Gmm	Avg. Gmm	Gmm	Avg. Gmm
225+45	Right	A	2.484		2.465	
		B	*	2.484	*	2.465
260+00	Right	C	2.499		2.497	
		D	2.516	2.508	2.514	2.506
299+00	Left	E	2.448		2.433	
		F	2.445	2.447	2.370	2.402
201+00	Left	G	2.493		2.505	
		H	2.507	2.500	2.504	2.505
194+00	Left	I	2.470		*	
		J	2.475	2.473	*	*
477+00	Right	K	2.440		2.435	
		L	2.450	2.445	2.449	2.442
644+00	Right	M	2.411		2.417	
		N	2.412	2.412	2.407	2.412
655+00	Right	O	2.407		2.417	
		P	2.419	2.413	2.390	2.404
477+35	Left	Q	2.446		2.373	
		R	2.463	2.455	2.460	2.417
598+00	Left	S	2.458		2.460	
		T	2.477	2.468	2.472	2.466
602+50	Left	U	2.420		2.419	
		V	2.411	2.416	2.413	2.416
660+26	Left	W	2.408		2.410	
		X	2.414	2.411	2.410	2.410

* Sample not tested.

Table A-13. Asphalt Content and Recovered Gradation, Quincy-Oroville Road.

Sta.	222+45	260+00	299+00	201+00	194+00	477+00	644+00	655+00	477+35	598+00	602+50	660+26
Lane	Right	Right	Left	Left	Left	Right	Right	Right	Left	Left	Left	Left
Sieve	Percent Passing											
1.5 inch												
1 inch					100					100		
3/4 inch	100	100	100	100	99	100	100	100	100	98	100	100
1/2 inch	96	96	97	97	94	97	99	99	95	86	99	99
3/8 inch	89	90	89	88	87	90	94	94	84	75	92	91
N0. 4	69	71	62	69	61	64	70	72	59	52	70	67
N0. 8	51	51	43	52	42	43	49	53	40	37	50	48
No. 16	39	38	31	40	30	31	35	39	29	27	36	36
No. 30	29	29	24	30	23	24	26	29	22	20	27	27
No. 50	21	21	20	21	19	19	19	21	18	15	20	19
No. 100	15	15	17	15	15	16	15	15	15	12	15	14
No. 200	10.9	11.1	12.8	11.1	11.6	11.9	11.2	11.6	11.6	8.7	11.5	10.9
% AC	7.9	7.4	10.3	8.2	9.2	9.9	9.6	8.6	10.2	6.7	9.1	8.4

Table A-14. Bulk Specific Gravity - Lab Molded Samples, Quincy-Oroville Road.

				AASHTO T 166			AASHTO T 331	
Sta.	Lane	Gmm	Sample	Gmb	Abs. (%)	VTM (%)	Gmb	VTM (%)
222+45	Right	2.465	13	2.151	5.59	12.72	2.120	14.01
		2.465	14	2.152	5.19	12.68	2.112	14.31
260+00	Right	2.506	21	2.169	4.31	13.46	2.133	14.90
		2.506	22	2.153	4.47	14.11	2.117	15.53
299+00	Left	2.402	29	2.089	9.07	13.04	1.962	18.33
		2.402	30	2.061	8.24	14.19	1.983	17.45
201+00	Left	2.505	37	2.277	2.38	9.12	2.249	10.24
		2.505	38	2.255	2.78	9.98	2.227	11.08
194+00	Left	2.473	45	2.014	10.93	18.56	1.863	24.68
		2.473	46	2.041	10.81	17.45	1.894	23.41
477+00	Right	2.442	53	2.067	8.74	15.36	1.966	19.47
		2.442	54	2.077	8.46	14.93	1.991	18.49
644+00	Right	2.412	61	2.021	4.59	16.22	1.971	18.28
		2.412	62	2.002	5.29	16.99	1.888	21.74
655+00	Right	2.404	69	2.013	8.18	16.27	1.941	19.26
		2.404	70	2.005	8.39	16.59	1.947	19.03
477+35	Left	2.431	77	2.038	9.82	15.67	1.928	20.24
		2.431	78	2.057	9.00	14.90	1.981	18.03
598+00	Left	2.466	85	2.181	3.92	11.57	2.155	12.59
		2.466	86	2.194	3.39	11.02	2.179	11.65
602+50	Left	2.416	93	2.010	9.94	16.79	1.933	20.00
		2.416	94	2.004	9.90	17.06	1.952	19.22
660+26	Left	2.410	101	2.113	4.84	12.32	2.065	14.31
		2.410	102	2.093	5.27	13.16	2.059	14.56

Table A-15. AASHTO T 283 Results, Quincy-Oroville Road.

Sta.	Lane	Cond.	Sample	Gmb	Gmm	VTM Dry (%)	VTM Sat. (%)	% Vacuum Saturation	Ht (mm)	Tensile Load (lbs)	Tensile Strength Dry (psi)	Tensile Strength Sat. (psi)	TSR
222+45	Right	Dry	8	2.100	2.465	14.81			90.9	1725	51.96		
222+45	Right	Dry	10	2.104	2.465	14.65			94.1	1530	44.52		
222+45	Right	Dry	12	2.112	2.465	14.32			94.1	1765	51.36		
222+45	Right	Sat.	7	2.057	2.465		16.55	63.4	90.8	1375		41.46	
222+45	Right	Sat.	9	2.109	2.465		14.44	62.7	94.9	1330		38.37	
222+45	Right	Sat.	11	2.089	2.465		15.25	62.3	94.3	1220		35.42	
			Average	2.095		14.59	15.42	62.8			49.28	38.42	0.78
260+00	Right	Dry	15	2.077	2.506	17.12			94.7	1600	69.39		
260+00	Right	Dry	19	2.093	2.506	16.48			93.9	1525	66.70		
260+00	Right	Dry	20	2.086	2.506	16.76			93.4	1540	67.72		
260+00	Right	Sat.	16	2.076	2.506		17.16	71.1	95.6	1190		51.13	
260+00	Right	Sat.	17	2.106	2.506		15.96	61.8	94.1	1150		50.19	
260+00	Right	Sat.	18	2.083	2.506		16.88	62.2	94.8	970		42.03	
			Average	2.087		16.79	16.67	65.0			67.94	47.78	0.70
299+00	Left	Dry	23	1.928	2.402	19.73			104.5	1635	64.26		
299+00	Left	Dry	25	1.961	2.402	18.36			93.2	1410	62.14		
299+00	Left	Dry	27	1.935	2.402	19.44			94.6	1375	59.70		
299+00	Left	Sat.	24	1.946	2.402		18.98	65.2	103.4	1550		61.57	
299+00	Left	Sat.	26	1.972	2.402		17.90	59.4	93.7	1450		63.56	
299+00	Left	Sat.	28	1.926	2.402		19.82	65.1	95.7	1325		56.87	
			Average	1.945		19.18	18.90	63.2			62.03	60.66	0.98
201+00	Left	Dry	31	2.208	2.505	11.86			89.8	1560	71.35		
201+00	Left	Dry	32	2.213	2.505	11.66			93.2	1500	66.10		
201+00	Left	Dry	36	2.202	2.505	12.10			95.1	1465	63.27		
201+00	Left	Sat.	33	2.209	2.505		11.82	66.1	94	1525		66.63	
201+00	Left	Sat.	34	2.221	2.505		11.34	63.9	94.1	1385		60.45	
201+00	Left	Sat.	35	2.171	2.505		13.33	63.1	95.9	1275		54.61	
			Average	2.204		11.87	12.16	64.4			66.91	60.56	0.91

Table A-15 (Con't.). AASHTO T 283 Results, Quincy-Oroville Road.

Sta.	Lane	Cond.	Sample	Gmb	Gmm	VTM Dry (%)	VTM Sat. (%)	% Vacuum Saturation	Ht (mm)	Tensile Load (lbs)	Tensile Strength Dry (psi)	Tensile Strength Sat. (psi)	TSR
194+00	Left	Dry	41	1.885	2.473	23.78			94.2	1610	46.80		
194+00	Left	Dry	43	1.881	2.473	23.94			94.4	1470	42.64		
194+00	Left	Dry	44	1.887	2.473	23.70			94.4	1630	47.28		
194+00	Left	Sat.	39	1.877	2.473		24.10	58.9	94.6	1500		43.42	
194+00	Left	Sat.	40	1.885	2.473		23.78	64.8	94.3	1500		43.55	
194+00	Left	Sat.	42	1.887	2.473		23.70	73.9	94.7	1410		40.77	
			Average	1.884		23.80	23.86	65.9			45.57	42.58	0.93
477+00	Right	Dry	47	2.005	2.442	17.90			90.5	1710	77.61		
477+00	Right	Dry	51	1.99	2.442	18.51			94.9	1635	70.76		
477+00	Right	Dry	52	1.973	2.442	19.21			96	1600	68.45		
477+00	Right	Sat.	48	1.992	2.442		18.43	74.4	90.2	1825		83.10	
477+00	Right	Sat.	49	1.998	2.442		18.18	63.9	95.3	1575		67.88	
477+00	Right	Sat.	50	1.983	2.442		18.80	63.8	95.8	1460		62.59	
			Average	1.990		18.54	18.47	67.4			72.27	71.19	0.99
644+00	Right	Dry	57	1.953	2.412	19.03			94.4	1580	68.74		
644+00	Right	Dry	58	1.93	2.412	19.98			96.1	1475	63.04		
644+00	Right	Dry	60	1.945	2.412	19.36			95.8	2225	95.39		
644+00	Right	Sat.	55	1.932	2.412		19.90	71.8	96.6	1630		69.30	
644+00	Right	Sat.	56	1.952	2.412		19.07	69.0	94	1525		66.63	
644+00	Right	Sat.	59	1.932	2.412		19.90	63.3	95.6	1320		56.71	
			Average	1.941		19.46	19.62	68.0			75.72	64.22	0.85
655+00	Right	Dry	64	1.914	2.404	20.38			94.7	2150	93.25		
655+00	Right	Dry	66	1.927	2.404	19.84			94.2	2000	87.20		
655+00	Right	Dry	68	1.906	2.404	20.72			94.7	1900	82.40		
655+00	Right	Sat.	63	1.904	2.404		20.80	68.1	94.9	1490		64.49	
655+00	Right	Sat.	65	1.922	2.404		20.05	66.4	93.5	1670		73.36	
655+00	Right	Sat.	67	1.911	2.404		20.51	65.8	94.8	1560		67.59	
			Average	1.914		20.31	20.45	66.8			87.62	68.48	0.78

Table A-15 (Con't.). AASHTO T 283 Results, Quincy-Oroville Road.

Sta.	Lane	Cond.	Sample	Gmb	Gmm	VTM Dry (%)	VTM Sat. (%)	% Vacuum Saturation	Ht (mm)	Tensile Load (lbs)	Tensile Strength Dry (psi)	Tensile Strength Sat. (psi)	TSR
477+35	Left	Dry	72	1.959	2.431	19.42			93.9	2050	89.67		
477+35	Left	Dry	73	1.925	2.431	20.81			93.6	1660	72.84		
477+35	Left	Dry	75	1.945	2.431	19.99			95	1800	77.82		
477+35	Left	Sat.	71	1.94	2.431		20.20	64.5	94.1	1600		69.84	
477+35	Left	Sat.	74	1.944	2.431		20.03	64.8	93.7	1625		71.23	
477+35	Left	Sat.	76	1.956	2.431		19.54	66.6	93.5	1525		66.99	
			Average	1.945		20.07	19.92	65.3			80.11	69.35	0.87
598+00	Left	Dry	79	2.159	2.466	12.45			88.8	1675	77.47		
598+00	Left	Dry	82	2.168	2.466	12.08			93.6	1675	73.50		
598+00	Left	Dry	83	2.157	2.466	12.53			94.3	1700	74.04		
598+00	Left	Sat.	80	2.17	2.466		12.00	73.3	91.1	1075		48.47	
598+00	Left	Sat.	81	2.174	2.466		11.84	74.8	94.1	1050		45.83	
598+00	Left	Sat.	84	2.124	2.466		13.87	71.7	94.6	875		37.99	
			Average	2.159		12.35	12.57	73.3			75.00	44.09	0.59
602+50	Left	Dry	89	1.946	2.416	19.45			93.6	1900	83.37	73.26	
602+50	Left	Dry	90	1.942	2.416	19.62			94.5	2060	89.53	81.68	
602+50	Left	Dry	92	1.916	2.416	20.70			96.8	1700	72.13		
602+50	Left	Sat.	87	1.921	2.416		20.49	64.6	97.3	1710		72.18	
602+50	Left	Sat.	88	1.932	2.416		20.03	62.4	94.8	1725		74.74	
602+50	Left	Sat.	91	1.942	2.416		19.62	72.4	94.7	1680		72.86	
			Average	1.933		19.92	20.05	66.5			81.68	74.94	0.92
660+26	Left	Dry	95	1.997	2.41	17.14			95.2	2750	118.64		
660+26	Left	Dry	97	2.017	2.41	16.31			94.4	2760	120.08		
660+26	Left	Dry	99	1.962	2.41	18.59			95.6	2360	101.39		
660+26	Left	Sat.	96	2.011	2.41		16.56	72.6	94	2240		97.87	
660+26	Left	Sat.	98	1.988	2.41		17.51	65.6	95.8	1870		80.17	
660+26	Left	Sat.	100	1.993	2.41		17.30	73.3	97	1950		82.57	
			Average	1.995		17.34	17.12	70.5			113.37	86.87	0.77

Table A-16. Bulk Specific Gravity, Dynamic Modulus Samples, Quincy-Oroville Road.

Sta	Sample	150 mm Bulk Sample VTM (%)	100 mm Cored Sample VTM (%)
194+00L	1	24.3	21.8
194+00L	2	24.1	22.4
201+00L	1	14.3	12.1
260+00R	1	15.0	12.6
299+00L	1	18.9	16.0
477+00R	1	19.2	18.0
477+00R	2	19.0	16.7
477+35L	1	19.7	17.4
598+00L	1	14.3	12.1
598+00L	2	12.2	10.6
602+50L	1	20.5	18.0
602+50L	2	21.3	18.8
644+00R	1	19.6	17.3
655+00R	1	19.0	16.9
660+26L	1	18.2	18.0
660+26L	2	17.6	15.7

Table A-17. Dynamic Modulus Results, Quincy-Oroville Road.

Station	Lane	Sample	VTM (%)	Test Temp. C	Dynamic Modulus (MPa) Test Frequency 10 Hz	1 Hz	0.1 Hz	0.01 Hz
602+20	Left	1	18.0	4	1,040,355	849,776	656,876	
				20	585,662	395,518	238,152	
				35	270,785	135,610	59,320	26,397
		2	18.8	4	974,943	790,020	599,731	
				20	542,296	357,228	208,274	
				35	234,671	115,015	48,878	21,611
660+26	Left	1	18.0	4	1,161,462	932,157	700,677	
				20	653,105	429,892	247,869	
				35	279,923	136,045	58,740	26,832
		2	15.7	4	1,120,271	903,875	688,929	
				20	587,113	395,228	239,312	
				35	268,610	139,671	65,992	31,908
225+45	Right	1	15.9	4	1,390,476	1,127,813	861,234	
				20	753,616	492,258	283,259	
				35	332,862	162,152	68,748	28,282
194+00	Left	1	21.8	4	740,562	616,990	488,632	
				20	441,930	303,854	185,213	
				35	210,595	108,778	49,023	22,191
299+00	Left	1	16.0	4	855,577	699,372	539,830	
				20	495,739	330,976	192,900	
				35	225,389	108,488	45,542	20,160
201+00	Left	1	12.1	4	1,461,400	1,194,240	924,180	
				20	771,601	525,617	325,320	
				35	363,319	194,060	91,519	40,756
644+00	Right	1	17.3	4	970,157	786,975	598,281	
				20	487,472	347,655	200,732	
				35	233,076	115,885	47,862	20,885
655+00	Right	1	16.9	4	1,145,363	946,806	738,097	
				20	645,998	432,067	256,717	
				35	293,846	145,038	62,076	25,962
260+00	Right	1	12.6	4	1,357,698	1,083,722	817,432	
				20	736,066	483,121	288,045	
				35	327,640	170,419	81,656	39,885
477+35	Left	1	17.4	4	825,990	675,296	519,960	
				20	503,861	340,113	200,442	
				35	249,755	127,778	58,305	27,557
598+00	Left	1	12.1	4	1,088,363	848,761	620,616	
				20	576,380	368,106	210,450	
				35	249,175	125,893	59,756	31,038

Table A-18. QA Data, Quincy-Oroville Road.

		Unit Weight (pcf)			% Compaction	
Sta.	Offset	Wet	Dry	Max	Wet	Dry
13+89	R	120.0	106.7	125.8	95.4	84.8
17+50	R	121.9	106.8	125.8	96.9	84.9
21+20	R	119.5	106.7	125.8	95.0	84.8
27+00	R	120.3	108.0	125.8	95.6	85.9
31+50	R	117.1	104.6	125.8	93.1	83.1
39+00	R	122.4	108.0	125.8	97.3	85.9
43+50	R	118.4	105.4	125.8	94.1	83.8
49+00	R	124.5	111.6	125.8	99.0	88.7
51+00	R	120.5	107.2	125.8	95.8	85.2
58+00	R	125.4	110.3	125.8	99.7	87.7
62+50	R	120.9	106.4	125.8	96.1	84.6
67+40	R	119.5	105.3	125.8	95.0	83.7
71+50	R	116.6	104.3	125.8	92.7	82.9
78+00	R	120.6	106.4	125.8	95.9	84.6
82+50	R	122.7	109.0	125.8	97.5	86.6
88+00	R	120.9	106.9	125.8	96.1	85.0
90+25	R	119.6	106.4	125.8	95.1	84.6
12+00	L	127.3	112.4	125.8	101.2	89.3
17+00	L	128.2	114.1	125.8	101.9	90.7
22+00	L	126.5	108.7	125.8	100.6	86.4
27+00	L	120.7	107.5	125.8	95.9	85.5
32+00	L	115.8	102.7	125.8	92.1	81.6
37+00	L	116.3	103.8	125.8	92.4	82.5
42+00	L	119.7	107.2	125.8	95.2	85.2
47+00	L	120.6	107.8	125.8	95.9	85.7
42+00	L	121.1	108.7	125.8	96.3	86.4
47+00	L	121.5	109.0	125.8	96.6	86.6
52+00	L	122.7	109.2	125.8	97.5	86.8
57+00	L	122.5	107.9	125.8	97.4	85.8
62+00	L	120.3	109.9	125.8	95.6	87.4
67+00	L	117.7	106.0	125.8	93.6	84.3
72+00	L	123.3	111.6	125.8	98.0	88.7
77+00	L	120.3	109.0	125.8	95.6	86.6
82+00	L	120.7	109.9	125.8	95.9	87.4
87+00	L	117.4	106.1	125.8	93.3	84.3
107+00	R	116.2	102.2	125.8	92.4	81.2
121+00	R	120.0	106.6	125.8	95.4	84.7

Table A-18 (Con't.). QA Data, Quincy-Oroville Road.

		Unit Weight (pcf)			% Compaction	
Sta.	Offset	Wet	Dry	Max	Wet	Dry
123+50	R	116.3	103.3	125.8	92.4	82.1
129+00	R	115.9	104.4	125.8	92.1	83.0
134+00	R	116.4	104.5	125.8	92.5	83.1
139+00	R	116.5	104.1	125.8	92.6	82.8
102+00	R	146.7	133.9	134.7	108.9	99.4
100+00	R	146.3	133.4	134.7	108.6	99.0
96+00	L	146.2	131.0	134.7	108.5	97.2
95+50	L	145.2	132.0	134.7	107.8	98.0
174+00	R	115.9	102.8	125.8	92.1	81.7
179+00	R	117.8	106.1	125.8	93.6	84.3
184+00	R	117.6	106.3	125.8	93.5	84.5
189+00	R	115.8	102.9	125.8	92.1	81.8
195+00	R	116.8	103.4	125.8	92.8	82.2
144+00	R	116.2	109.7	125.8	92.4	87.2
149+00	R	121.3	109.1	125.8	96.4	86.7
154+00	R	117.8	106.0	125.8	93.6	84.3
159+00	R	120.3	108.9	125.8	95.6	86.6
164+00	R	117.3	106.1	125.8	93.2	84.3
169+00	R	119.7	107.2	125.8	95.2	85.2
198+00	R	119.9	107.9	125.8	95.3	85.8
203+00	R	125.7	111.2	125.8	99.9	88.4
208+00	R	124.0	110.4	125.8	98.6	87.8
213+00	R	120.3	107.3	125.8	95.6	85.3
218+00	R	119.5	106.3	125.8	95.0	84.5
223+00	R	119.7	106.8	125.8	95.2	84.9
228+00	R	115.8	101.0	125.8	92.1	80.3
233+00	R	115.9	103.3	125.8	92.1	82.1
238+00	R	123.1	108.8	125.8	97.9	86.5
243+00	R	120.8	107.9	125.8	96.0	85.8
248+00	R	116.3	103.2	125.8	92.4	82.0
253+00	R	119.0	104.6	125.8	94.6	83.1
258+00	R	119.8	107.0	125.8	95.2	85.1
263+00	R	117.1	103.9	125.8	93.1	82.6
268+00	R	118.3	106.6	125.8	94.0	84.7
273+00	R	116.2	102.7	125.8	92.4	81.6
278+00	R	116.2	108.1	125.8	92.4	85.9
283+00	R	119.0	106.6	125.8	94.6	84.7

Table A-18 (Con't.). QA Data, Quincy-Oroville Road.

		Unit Weight (pcf)			% Compaction	
Sta.	Offset	Wet	Dry	Max	Wet	Dry
288+00	R	116.4	103.1	125.8	92.5	82.0
293+00	R	119.3	106.4	125.8	94.8	84.6
303+60	L	119.5	108.4	125.8	95.0	86.2
298+00	L	116.9	105.2	125.8	92.9	83.6
293+00	L	119.0	106.2	125.8	94.6	84.4
288+00	L	118.0	106.2	125.8	93.8	84.4
283+00	L	119.6	107.6	125.8	95.1	85.5
278+00	L	122.3	109.9	125.8	97.2	87.4
273+00	L	119.6	106.4	125.8	95.1	84.6
268+00	L	117.3	104.5	125.8	93.2	83.1
263+00	L	122.4	109.2	125.8	97.3	86.8
258+00	L	118.4	104.8	125.8	94.1	83.3
253+00	L	120.7	107.2	125.8	95.9	85.2
248+00	L	118.2	106.8	125.8	94.0	84.9
233+00	L	121.9	110.1	125.8	96.9	87.5
228+00	L	118.6	104.9	125.8	94.3	83.4
223+00	L	120.8	107.1	125.8	96.0	85.1
218+00	L	121.8	108.6	125.8	96.8	86.3
213+00	L	118.1	105.9	125.8	93.9	84.2
198+00	L	116.9	105.2	125.8	92.9	83.6
299+50	R	116.2	107.5	125.8	92.4	85.5
294+74	R	116.3	105.2	125.8	92.4	83.6
282+00	R	123.8	113.1	125.8	98.4	89.9
263+50	R	116.9	107.0	125.8	92.9	85.1
242+00	R	126.4	112.6	125.8	100.5	89.5
205+50	R	127.3	114.2	125.8	101.2	90.8
190+00	L	119.4	105.0	125.8	94.9	83.5
185+00	L	123.8	113.1	125.8	98.4	89.9
180+00	L	119.6	107.6	125.8	95.1	85.5
175+00	L	119.5	108.4	125.8	95.0	86.2
170+00	L	118.2	106.8	125.8	94.0	84.9
165+00	L	119.6	106.4	125.8	95.1	84.6
160+00	L	116.1	104.5	125.8	92.3	83.1
155+00	L	116.1	105.5	125.8	92.3	83.9
150+00	L	116.0	102.3	125.8	92.2	81.3
145+00	L	117.9	104.4	125.8	93.7	83.0
140+00	L	122.7	109.3	125.8	97.5	86.9
135+00	L	119.9	106.8	125.8	95.3	84.9

Table A-18 (Con't.). QA Data, Quincy-Oroville Road.

		Unit Weight (pcf)			% Compaction	
Sta.	Offset	Wet	Dry	Max	Wet	Dry
130+00	L	115.8	101.4	125.8	92.1	80.6
125+00	L	119.6	105.2	125.8	95.1	83.6
120+00	L	115.8	102.2	125.8	92.1	81.2
115+00	L	116.2	102.4	125.8	92.4	81.4
110+00	L	118.3	105.8	125.8	94.0	84.1
105+00	L	119.4	106.5	125.8	94.9	84.7
100+00	L	118.4	104.8	125.8	94.1	83.3
480+00	R	115.9	105.2	125.8	92.1	83.6
485+00	R	115.7	104.8	125.8	92.0	83.3
490+00	R	116.7	104.1	125.8	92.8	82.8
495+00	R	116.4	103.8	125.8	92.5	82.5
500+00	R	117.1	104.0	125.8	93.1	82.7
505+00	R	119.7	106.9	125.8	95.2	85.0
510+00	R	119.0	108.4	125.8	94.6	86.2
515+00	R	119.0	106.4	125.8	94.6	84.6
520+00	R	116.9	103.5	125.8	92.9	82.3
525+00	R	118.1	104.7	125.8	93.9	83.2
530+00	R	115.8	105.6	125.8	92.1	83.9
535+00	R	116.5	104.8	125.8	92.6	83.3
545+00	R	118.0	104.5	125.8	93.8	83.1
550+00	R	121.0	108.3	125.8	96.2	86.1
555+00	R	128.7	115.5	125.8	102.3	91.8
560+00	R	120.4	109.4	125.8	95.7	87.0
565+00	R	119.5	109.5	125.8	95.0	87.0
570+00	R	117.6	105.2	125.8	93.5	83.6
575+00	R	118.5	106.2	125.8	94.2	84.4
580+00	R	117.9	107.2	125.8	93.7	85.2
585+00	R	115.9	104.7	125.8	92.1	83.2
590+00	R	120.1	107.9	125.8	95.5	85.8
595+00	R	122.0	108.5	125.8	97.0	86.2
600+00	R	116.6	104.5	125.8	92.7	83.1
605+00	R	116.1	106.9	125.8	92.3	85.0
610+00	R	116.0	103.0	125.8	92.2	81.9
615+00	R	119.6	108.6	125.8	95.1	86.3
620+00	R	116.5	104.7	125.8	92.6	83.2
625+00	R	116.8	104.0	125.8	92.8	82.7
630+00	R	116.3	103.1	125.8	92.4	82.0

Table A-18 (Con't.). QA Data, Quincy-Oroville Road.

		Unit Weight (pcf)			% Compaction	
Sta.	Offset	Wet	Dry	Max	Wet	Dry
635+00	R	115.9	103.8	125.8	92.1	82.5
640+00	R	117.2	105.3	125.8	93.2	83.7
645+00	R	117.4	105.3	125.8	93.3	83.7
650+00	R	117.7	105.1	125.8	93.6	83.5
655+00	R	120.9	107.5	125.8	96.1	85.5
660+00	R	119.8	109.1	125.8	95.2	86.7
665+00	R	117.6	105.7	125.8	93.5	84.0
675+00	R	120.4	108.8	125.8	95.7	86.5
680+00	R	122.9	110.3	125.8	97.7	87.7
480+00	L	117.3	105.3	125.8	93.2	83.7
485+00	L	117.8	105.4	125.8	93.6	83.8
490+00	L	115.9	105.1	125.8	92.1	83.5
495+00	L	117.1	102.6	125.8	93.1	81.6
500+00	L	115.8	104.1	125.8	92.1	82.8
505+00	L	116.0	104.3	125.8	92.2	82.9
510+00	L	116.0	105.2	125.8	92.2	83.6
515+00	L	116.6	104.1	125.8	92.7	82.8
520+00	L	116.5	104.5	125.8	92.6	83.1
525+00	L	118.0	105.0	125.8	93.8	83.5
530+00	L	116.4	105.2	125.8	92.5	83.6
535+00	L	115.8	105.9	125.8	92.1	84.2
540+00	L	116.6	106.2	125.8	92.7	84.4
545+00	L	118.9	106.2	125.8	94.5	84.4
550+00	L	116.4	104.8	125.8	92.5	83.3
555+00	L	118.8	107.4	125.8	94.4	85.4
560+00	L	119.9	109.0	125.8	95.3	86.6
565+00	L	121.5	109.3	125.8	96.6	86.9
570+00	L	116.0	104.5	125.8	92.2	83.1
575+00	L	119.2	106.4	125.8	94.8	84.6
580+00	L	122.2	111.8	125.8	97.1	88.9
585+00	L	120.2	106.5	125.8	95.5	84.7
590+00	L	120.6	109.6	125.8	95.9	87.1
595+00	L	122.9	108.0	125.8	97.7	85.9
600+00	L	122.9	110.3	125.8	97.7	87.7
600+00	L	120.0	106.2	125.8	95.4	84.4
605+00	L	119.3	108.9	125.8	94.8	86.6
610+00	L	119.6	108.6	125.8	95.1	86.3

Table A-18 (Con't.). QA Data, Quincy-Oroville Road.

		Unit Weight (pcf)			% Compaction	
Sta.	Offset	Wet	Dry	Max	Wet	Dry
615+00	L	116.3	103.1	125.8	92.4	82.0
620+00	L	116.0	104.1	125.8	92.2	82.8
625+00	L	117.6	104.3	125.8	93.5	82.9
630+00	L	118.3	106.6	125.8	94.0	84.7
635+00	L	116.5	104.7	125.8	92.6	83.2
640+00	L	118.3	106.1	125.8	94.0	84.3
645+00	L	115.9	105.1	125.8	92.1	83.5
650+00	L	117.8	106.7	125.8	93.6	84.8
655+00	L	116.2	102.9	125.8	92.4	81.8
660+00	L	116.8	104.0	125.8	92.8	82.7
665+00	L	117.4	106.7	125.8	93.3	84.8
670+00	L	118.3	106.0	125.8	94.0	84.3
675+00	L	116.6	105.1	125.8	92.7	83.5
680+00	L	117.6	107.2	125.8	93.5	85.2

Table A-19. Core Test Results, Quincy-Oroville Road.

		Bulk Specific Gravity			Unit Weight		
		AASHTO T 331		Moisture	Nuclear	AASHTO T 331	
Sta.	Sample	Moist	Dry	Content (%)	Guage* (pcf)	Moist (pcf)	Dry (pcf)
213+00	1B	1.980	1.959	1.02	117.1	123.5	122.3
106+00	2B	2.009	1.982	1.38	116.6	125.4	123.7
111+30	3B	1.949	1.932	0.87	116.7	121.6	120.6
118+75	4B	1.966	1.944	1.11	116.1	122.7	121.3
202+10	5B	2.105	2.086	0.92	122.6	131.4	130.2
207+00	6B	2.058	2.043	0.77	119.0	128.4	127.5
251+10	8B	1.967	1.948	0.98	119.0	122.7	121.5
246+00	9B	2.065	2.049	0.81	123.0	128.9	127.8
241+00	10B	2.004	1.990	0.68	117.8	125.0	124.2

*Nuclear gauge unit weight is a moist unit weight.

Standard Method for

Determination of Optimum Asphalt Emulsion Content

of Cold In-Place Recycled Mixtures

FLH Designation: T 524

1. SCOPE

1.1 This procedure is used to determine the percent and grade of recycling agent to use for recycling asphalt concrete when using Cold In-Place Recycling (CIR) of bituminous pavements.

2. COLD MIX REQUIREMENTS

2.1 The recycled pavement mixture shall conform to the following quality requirements shown in Table B-1.

Table B-1. Cold Mix Requirements.

Design Parameters	Requirement
Gradation of Design Reclaimed Asphalt Pavement (RAP), AASHTO T 27	Table 2
Asphalt Content of RAP, AASHTO T 308	Report
Bulk Specific Gravity of Compacted Samples (1) (2), AASHTO T 166 Method A, AASHTO T 331 when required	Report
Maximum Theoretical Specific Gravity (2), AASHTO T 209 or ASTM D6857	Report
Air Voids of Compacted and Cured Specimens (2), AASHTO T 269	Report(3)
Indirect Tensile Strength, Cured Specimen (2), AASHTO T 283, 77 ± 2°F (25 ± 1°C)	70 psi Minimum
Tensile Strength Ratio, AASHTO T 283, 77 ± 2°F (25 ± 1°C) Based on Moisture Conditioning on Cured Specimen (2) (4)	0.70 minimum
Raveling Test, ASTM D7196, cured for 4 hours at 50°F and 50% relative humidity, unless directed by engineer(5).	Maximum 5% loss
Ratio of Residual Asphalt Content to Cement	Minimum 3.0:1.0
RAP Coating Test, AASHTO T 59, using RAP from mix design and emulsion, water and additive rates at optimum from mix design	Minimum good

Notes:

1. 150 mm diameter mold, compaction based on gyratory compaction in accordance with AASHTO T 312, 35 gyrations. Do not heat molds or materials.
2. Measurement on specimens after 140°F (60°C) curing to constant weight for no less than 16 hours and no more than 48 hours.
3. Typical values 8-16% or more. Do not design CIR or adjust emulsion content to meet specific air void content.

4. Vacuum saturation of 55 to 75 percent, no freeze cycle, 24 hour soak in water bath at 77°F (25°C).
5. 150 mm diameter mold, compaction based on gyratory compaction in accordance with AASHTO T 312, 25 gyrations. Do not heat molds or materials.

3. PREPARATION OF SAMPLES

3.1 *Sampling & Processing of Existing Asphalt Pavement Materials*

3.1.1 In coordination with the project engineer, obtain cores from the areas to be recycled. Depending upon project length and existing pavement thickness, develop a coring plan that will provide at least 350 lbs. of RAP for each mix design and additional cores for asphalt content and gradation analysis of the existing pavement (to the specified milling depth). Extract three or more cores for each lane mile to check for pavement consistency with additional cores where visual differences in the pavement are noticed. If cores show significant differences in various areas, such as different type or thickness of layers between cores, then perform separate mix designs for each of these pavement segments.

3.1.2 Cores are to be cut to the depth specified for the cold recycling project.

3.1.3 Determine the average asphalt content (using AASHTO T 308) and recovered aggregate gradation (using AASHTO T 30) from the depth specified for milling on two representative cores for each mix design.

3.1.4 Milled RAP from the areas and depth to be recycled or other approved means of obtaining RAP samples can be used as an alternative to cores.

3.1.5 Crush cores to obtain materials that meet the gradation shown in Table B-2.

3.1.6 Perform a mix design by recombining the RAP material in the laboratory in order to meet the gradation criteria shown in Table B-2.

Table B-2. RAP Gradation Requirements.

Sieve Size	Percent Passing
1.5 inch	100
1 inch	90-100
3/4 inch	85-95
1/2 inch	75-85
No. 4	35-50
No. 16	5-16
No. 200	0-7

3.1.7 Determine gradation of RAP after milling or crushing in accordance with AASHTO T 27 with the exception that drying of RAP samples to constant mass shall be performed at 104 ± 4°F (40 ± 2°C). The washed sieve analysis of AASHTO T 11 is not required.

3.1.8 Determine the asphalt content in accordance with AASHTO T 308 on a sample of RAP batched to the proposed mix design gradation, and meeting requirements of Table B-2.

3.2 *Sampling of Emulsified Asphalt Binder Agent*

3.2.1 Obtain 3 gallons of the emulsified asphalt that will be used to produce the cold recycled mix. Include the name and location of the supplier in the mix design report. Include the grade and properties of the emulsified asphalt in the mix design report.

3.3 *Sampling of Other Additives*

3.3.1 Obtain 5 lbs of quicklime if quicklime will be used as a part of the mix design.

3.3.2 Obtain a sufficient amount of other additives that will be used to complete the mix design. List the name and source of all additives in the mix design report.

4. TEST SAMPLE PREPARATION

4.1 *Specimen Size*

4.1.1 Determine the amount of RAP required to produce a 95 ± 5 mm tall specimen when compacting 150 mm diameter specimens with the gyratory compactor at 35 gyrations for testing.

4.2 *Number of Specimens*

4.2.1 Use a minimum of three emulsion contents that bracket the estimated recommended emulsion content for all tensile strength testing outlined in Table B-1. Select three emulsion contents in either 0.5% or 1.0% increments covering a range typically between 1% and 4.0% by dry weight of RAP. Compact 6 samples at each emulsion content for tensile strength testing, 3 for unconditioned (dry) tensile strength on cured samples and 3 for conditioned tensile strength on cured samples for moisture conditioning.

4.2.2 Two specimens are required for determination of theoretical maximum specific gravity according to AASHTO T 209 or ASTM D6857 with the exception that loose RAP mixtures are cured in a forced draft oven at 140 ± 2°F (60 ± 1°C) to constant weight but no more than 48 hours and no less than 16 hours. Constant weight is defined as 0.05% change in weight in 2 hours. Do not break any agglomerates that will not easily reduce with a flexible spatula. Test both specimens at the highest emulsion content in the design and back calculate for the lower emulsion contents. Use the dry-back procedure of AASHTO T 209 to account for uncoated particles. ASTM D6857 may be used as an alternative to AASHTO T 209.

4.3 *Mechanical Mixing*

4.3.1 Mix samples for testing using a mechanical bucket mixer or laboratory sized pugmill or a combination of the two. Add moisture that is expected to be added at the milling head, typically 1.5 to 2.5 percent, and mix thoroughly. If any additives (such as lime) are in the mixture, introduce the additives in a similar manner that they will be added during field production.

4.3.2 Mix RAP thoroughly with water first or water and additives as appropriate, then mix with emulsion at room temperature, 77 ± 4°F (25 ± 2°C). One specimen will be mixed at a time. Mixing time with emulsion should not exceed 60 seconds.

5. COMPACTION

5.1 After mixing, compact specimens immediately. Compact specimens at room temperature, 77 ± 4°F (25 ± 2°C).

5.2 Specimens for AASHTO T 283 testing are compacted using 150 mm molds to 35 gyrations in accordance with AASHTO T 312, with the exception that materials and molds are not heated.

5.3 If paper disks are used, place paper disks on the top and bottom of the specimen before compaction and remove paper disks from specimens immediately after compaction.

6. CURING AFTER COMPACTION

6.1 Extrude specimens from molds after compaction without damaging samples. Carefully remove paper disks if used.

6.2 Place specimens in 140 ± 2°F (60 ± 1°C) forced draft oven with ventilation on sides and top. Place each specimen in a small container to account for material loss from the specimens. Cure compacted specimens at 140 ± 2°F (60 ± 1°C) to constant weight but do not heat for more than 48 hours and not less than 16 hours. Constant weight is defined as 0.05% change in weight in 2 hours. After curing, cool specimens at ambient temperature a minimum of 12 hours and a maximum of 24 hours.

6.3 Cure maximum specific gravity specimens at the same conditions as the compacted specimens.

7. SAMPLE CONDITIONING

7.1 Perform same oven conditioning and volumetric measurements on moisture-conditioned specimens as on other specimens.

7.2 Perform moisture conditioning on 3 compacted samples at each emulsion content by applying a vacuum of 13 to 67 kPa absolute pressures (10 to 26 in. of Hg partial pressure) for a time duration required to vacuum saturate samples to 55 to 75 percent. Saturation calculation shall be in accordance with AASHTO T 283. Soak moisture conditioned samples in a 77 ± 2°F (25 ± 1°C) water bath for 24 ± 1 hours.

8. MEASUREMENTS ON COMPACTED SAMPLES

8.1 Determine bulk specific gravity of each compacted, cured and cooled specimen according to AASHTO T 116 Method A or AASHTO T 331, if required.

8.2 Determine specimen heights according to AASHTO T 245. Alternatively, the height can be obtained from the SGC readout.

8.3 Determine maximum theoretical specific gravity in accordance with AASHTO T 209 or ASTM D6857 as detailed in Section 4.2.2.

8.4 Determine air void contents of the compacted and oven-cured samples at each emulsion content according to AASHTO T 269.

8.5 Determine tensile strength ratio by AASHTO T 283. Dry or unconditioned samples are tested after a minimum of 45 minutes temperature conditioning by immersing in a 77 ± 2°F (25 ± 1°C) water bath. Place dry specimens in a leak proof bag to prevent samples from coming in contact with water. This testing is performed at the same time that moisture-conditioned specimens are tested.

8.6 Determine results of the Raveling Test by ASTM D7196 on samples mixed with the optimum emulsion content. Report the test temperature and relative humidity used. Test temperature and relative humidity will be 50ºF (10ºC) and 50% relative humidity unless directed by the engineer.

9. EMULSION CONTENT SELECTION

9.1 Choose the design emulsion content such that the cold mix requirements listed in Table B-1 are met. If the requirements of Table B-1 are met at more than one emulsion content, select the emulsion content that optimizes density.

10. REPORT

10.1 At a minimum, report the following information:

1) Gradation of RAP,
2) RAP asphalt content,
3) Recommended water content range as a percentage of dry RAP,
4) Amount of additive as a percentage of dry RAP,

5) Amount of additional aggregates if any as a percentage of dry RAP,
6) Range of emulsion contents.
7) Density, G_{mm}, and air voids at each emulsion content (average values) of AASHTO T 283 samples.
8) Indirect tensile strength at each emulsion content (average values).
9) Level of saturation and conditioned indirect tensile strength at each emulsion content (average values).
10) Tensile strength ratio.
11) Density, air void level, tensile strength, tensile strength ratio, and raveling at recommended moisture and emulsion contents.
12) Optimum emulsion content as a percentage of dry RAP,
13) Emulsion and additive designation, supplier company name and location,
14) Emulsion residue content;
15) Additive designation, company name and location;
16) Certificates of compliance for emulsion and additives.

Standard Method for

Determination of Optimum Asphalt Emulsion Content

of Full Depth Reclamation Mixtures

FLH Designation: T 522

1. SCOPE

1.1 This procedure is used to determine the percent and grade of recycling agent to use for recycling asphalt concrete when using Full Depth reclamation (FDR) of bituminous pavements.

2. FDR MIX REQUIREMENTS

2.1 The recycled pavement mixture shall conform to the following quality requirements shown in Table C-1.

Table C-1. FDR Laboratory Mix Design Tests.

Design Parameters	Requirement
Gradation of Design Reclaimed Asphalt Pavement (RAP), AASHTO T 27	Table 2
Asphalt Content of RAP, AASHTO T 308	Report
Gradation of Aggregate Base Material, AASHTO T 11 and AASHTO T 27	Report
Calculated Gradation of Combined RAP and Aggregate Base Material	Report
Sand Equivalent Combined RAP and Aggregate Base Material, AASHTO T 176	Report
Maximum Dry Density and Optimum Moisture Content, Combined RAP and Aggregate Base, AASHTO T 180, Method D	Report
Bulk Specific Gravity of Compacted Samples,(1)(2) AASHTO T 166 Method A, ASHTO T 331 when required	Report
Maximum Theoretical Specific Gravity,(2) AASHTO T 209 or ASTM D6857	Report
Air Voids of Compacted and Cured Specimens,(2) AASHTO T 269	Report(3)
Indirect Tensile Strength, Cured Specimen,(2) AASHTO T 283, 77 ± 2°F (25 ± 1°C)	40 psi minimum
Conditioned Indirect Tensile Strength, AASHTO T 283, 77 ± 2°F (25 ± 1°C) Based on Moisture Conditioning on Cured Specimen(2)(4).	25 psi minimum, TSR minimum 0.60
Ratio of Residual Asphalt Content to Cement	Minimum 3.0:1.0

Notes:

1. 150 mm diameter mold, compaction based on gyratory compaction in accordance with AASHTO T 312, 35 gyrations. Do not heat molds or materials.
2. Measurement on specimens after 140°F (60°C) curing to constant weight for no less than 16 hours and no more than 48 hours.
3. Typical values 8-16% or more. Do not design FDR or adjust emulsion content to meet specific air void content.
4. Vacuum saturation of 55-75 percent, no freeze cycle, 24 hour soak in water bath at 77 ± 2°F.

3. SAMPLING MATERIALS

3.1 *Sampling Existing Pavement Materials*

3.1.1 In coordination with the project engineer, obtain cores, auger borings, or test pit samples from the areas to be recycled. Depending upon project length and existing pavement thickness, develop a sampling plan that will provide at least 350 lbs. of RAP and aggregate base for each mix design. Extract three or more cores for each lane mile to check for pavement consistency with additional cores where visual differences in the pavement are noticed. If cores/ auger borings/test pits show significant differences in various areas, such as different type or thickness of layers between cores, then separate mix designs will be performed for each of these pavement segments. Separate mix designs are required for FDR projects with more than a 2-inch (50 mm) difference in bituminous surface between sections.

3.1.2 Cores are to be cut for the full depth of the asphalt pavement. Sample aggregate base to the depth specified for the full depth reclamation project.

3.1.3 Milled RAP from the areas and depth to be recycled or other approved means of obtaining RAP samples can be used as an alternative to cores.

3.2 *Sampling of Emulsified Asphalt Binder Agent*

3.2.1 Obtain 3 gallons of the emulsified asphalt that will be used to produce the cold recycled mix. Include the name and location of the supplier in the mix design report. Include the grade and properties of the emulsified asphalt in the mix design report.

3.3 *Sampling of Other Additives*

3.3.1 Obtain 10 lbs of cement if cement will be used as a part of the mix design.

3.3.2 Obtain a sufficient amount of other additives that will be used to complete the mix design. List the name and source of all additives in the mix design report.

4. PROCESSING PAVEMENT MATERIALS

4.1 *Processing Pavement Cores*

4.1.1 Determine the average asphalt content (using AASHTO T 308) and recovered aggregate gradation (using AASHTO T 30) on two representative cores for each mix design.

4.1.2 Crush cores or pavement chunks to the gradation shown in Table C-2 before blending with the aggregate base materials.

TABLE C-2. RAP Gradation for FDR Mixtures.

Sieve Size	Percent Passing
1.5 in. (38 mm)	100
1 in. (25 mm)	85-95
¾ in. (19 mm)	75-85
No. 4 (4.75 mm)	30-40
No. 30 (0.600 mm)	1-5

4.2 *Processing Aggregate Base Materials*

4.2.1 Perform a washed sieve analysis in accordance with AASHTO T 11 and AASHTO T 27 on the aggregate base material.

4.3 *Combined Materials*

4.3.1 Combine RAP prepared to the gradation in Table C-2 with aggregate base of the gradation determined in 4.2.1 to the planned percentages. Calculate the combined gradation of RAP and aggregate base.

4.3.2 Maximum size of the combined materials shall have 100 percent passing the 2 inch sieve unless directed by the engineer.

4.3.3 Perform a mix design by recombining RAP prepared to the gradation in Table C-2 and aggregate base of the gradation determined in 4.2.1 to the planned percentages and gradation determined in 4.3.2.

4.3.4 Specimens prepared for mix design shall have 100 percent passing the 1.5 inch sieve by removing plus 1.5 inch materials.

4.3.5 Determine the sand equivalent (AASHTO T 156) of the combined material.

4.3.6 Perform a Modified Proctor test on the combined material in accordance with AASHTO T 180, Method D, to determine optimum moisture content (OMC) at peak dry density of the combined RAP and base material. The OMC shall be defined by a best-fit curve. Materials shall be mixed with target moisture, sealed and set aside a minimum of 3 hours. If a material contains a significant amount of RAP or coarse material or less than 4 percent passing the No. 200 sieve and does not produce a well-defined OMC curve, then the moisture content shall be fixed at 3 percent.

5. TEST SAMPLE PREPARATION

5.1 *Specimen Size*

5.1.1 Determine the amount of RAP and aggregate base, before the addition of water, required to produce a 95 ± 5 mm tall specimen when compacting 150 mm diameter specimens with the gyratory compactor at 35 gyrations for testing.

5.2 *Number of Specimens*

5.2.1 Use a minimum of three emulsion contents that bracket the estimated design emulsion content for all tensile strength testing outlined in Table C-1. Select emulsion contents in either 0.5% or 1.0% increments covering a range typically between 2% and 6% by dry weight of mix. Compact 6 samples at each emulsion content for tensile strength testing, 3 for unconditioned (dry) tensile strength on cured samples and 3 for conditioned tensile strength on cured samples for moisture conditioning.

5.2.2 Two specimens are required for determination of theoretical maximum specific gravity according to AASHTO T 209 or ASTM D6857 with the exception that loose mix samples are cured in a forced draft oven at 140 ± 2°F (60 ± 1°C) to constant weight but no more than 48 hours and no less than 16 hours. Constant weight is defined as 0.05% change in weight in 2 hours. Do not break any agglomerates that will not easily reduce with a flexible spatula. Test both specimens at the highest emulsion content in the design and back calculate for the lower emulsion contents. Use the dry-back procedure of AASHTO T 209 to account for uncoated particles. ASTM D6857 may be used as an alternative to AASHTO T 209.

5.3 *Selection of Water Content for Design*

5.3.1 Select a water content of specimens, not including water in the emulsion, of 60-65 percent of the OMC determined in 4.3.6.

5.4 *Mechanical Mixing*

5.4.1 Mix samples for testing using a mechanical bucket mixer or laboratory sized pugmill or a combination of the two. Mix specimens with the required amount of water before addition of emulsion. If any additives (such as lime) are in the mixture, introduce the additives in a similar manner that they will be added during field production.

5.4.2 Mix specimens thoroughly with water first, or water and additives as appropriate, then mix with emulsion at room temperature, 77 ± 4°F (25 ± 2°C). One specimen will be mixed at a time. Mixing time with emulsion should not exceed 60 seconds.

5.5 Curing Before Compaction

5.5.1 Loose specimens shall be cured individually in plastic containers of 4-7 inches (100-180 mm) in height and 6 inches (150 mm) in diameter. Specimens shall be cured at 104 ± 2°F (40 ± 1°C) for 30 ± 3 minutes.

6. COMPACTION

6.1 Compact samples immediately after curing in 5.5.1.

6.2 Specimens for AASHTO T 283 testing are compacted using 150 mm molds to 35 gyrations in accordance with AASHTO T 312, with the exception that molds are not heated.

6.4 If paper disks are used, place paper disks on the top and bottom of the specimen before compaction and remove paper disks from specimens immediately after compaction.

7. CURING AFTER COMPACTION

7.1 Extrude specimens from molds after compaction without damaging samples. Carefully remove paper disks if used.

7.2 Place specimens in 140 ± 2°F (60 ± 1°C) forced draft oven with ventilation on sides and top. Place each specimen in a small container to account for material loss from the specimens. Cure compacted specimens at 140 ± 2°F (60 ± 1°C) to constant weight but do not heat for more than 48 hours and not less than 16 hours. Constant weight is defined as 0.05% change in weight in 2 hours. After curing, cool specimens at ambient temperature a minimum of 12 hours and a maximum of 24 hours.

7.3 Cure maximum specific gravity specimens at the same conditions as compacted specimens.

8. SAMPLE CONDITIONING

8.1 Perform same oven conditioning and volumetric measurements on moisture-conditioned specimens as on other specimens.

8.2 Perform moisture conditioning on 3 compacted samples at each emulsion content by applying a vacuum of 13 to 67 kPa absolute pressures (10 to 26 in. of Hg partial pressure) for a time duration required to vacuum saturate samples to 55 to 75 percent. Saturation calculation shall be in accordance with AASHTO T 283. Soak moisture conditioned samples in a 77 ± 2°F (25 ± 1°C) water bath for 24 ± 1 hours.

9. MEASUREMENTS ON COMPACTED SAMPLES

9.1 Determine bulk specific gravity of each compacted, cured and cooled specimen according to AASHTO T 166 Method A or AASHTO T 331 if required. Specimens shall be kept in bags until testing or vacuum saturation is performed.

9.2 Determine specimen heights according to AASHTO T 245. Alternatively, the height can be obtained from the SGC readout.

9.3 Determine maximum specific gravity in accordance with AASHTO T 209 or ASTM D6857 as detailed in section 5.2.2.

9.4 Determine air void contents of the compacted and oven-cured samples at each emulsion content according to AASHTO T 269.

9.5 Determine indirect tensile strengths and tensile strength ratio in accordance with AASHTO T 283. Dry or unconditioned samples are tested after a minimum of 45 minutes temperature conditioning by immersing in a 77 ± 2°F (25 ± 1°C) water bath. Place dry specimens in a leak proof bag to prevent samples from coming in contact with water. This testing is performed at the same time that moisture-conditioned specimens are tested.

10. EMULSION CONTENT SELECTION

10.1 Choose the design emulsion content such that the cold mix requirements listed in Table C-1 are met. If the requirements of Table C-1 are met at more than one emulsion content, select the emulsion content that optimizes density.

11. REPORT

11.1 At a minimum, report the following information:

1. Gradation of RAP.
2. Gradation of aggregate base.
3. Planned percentages of RAP and aggregate base.
4. Combined gradation of the blended RAP and aggregate base material.
5. Sand equivalent value of the blended material.
6. Density and OMC from Proctor compaction of the blended material.
7. Moisture content used in mix design.
8. Range of emulsion contents.
9. Density, G_{mm}, and air voids at each emulsion content (average values) of AASHTO T 283 samples.
10. Indirect tensile strength at each emulsion content (average values).
11. Level of saturation and conditioned indirect tensile strength at each emulsion content (average values).
12. Tensile strength ratio.
13. Design emulsion content.

14. Design water content.
15. Emulsion and additive designation, supplier company name and location,
16. Emulsion residue content;
17. Additive designation, company name and location;
18. Certificates of compliance for emulsion and additives.

APPENDIX D
CIR LITERATURE REVIEW

August 8, 2011

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Cold In-Place Recycling (CIR) Study
Literature Review

DEFINITIONS

Cold in-place recycling (CIR) is a sub discipline of the general field of cold recycling. Cold recycling is generally broken down into two disciplines, cold central plant recycling and cold in-place recycling. Cold in-place recycling is further divided into two sub disciplines, partial depth recycling and full depth recycling. Full depth recycling is commonly referred to as full depth reclamation (FDR) (1). The "partial depth" portion of partial depth cold in-place recycling appears to have been dropped in the majority of the literature and the term cold in-place recycling (CIR) is used; hence the confusion. This publication is limited to partial depth cold in-place recycling or simply, CIR.

CIR is a process where the bituminous portion of a pavement is recycled in-place, without the addition of heat (2). Most CIR is performed using recycling trains but two-unit and single unit equipment is available. Multiple unit trains can mill the pavement to the proper depth and grade, screen and crush oversized material, precisely meter in additives, mix the millings and recycling additives to a uniform mixture and place the material in a windrow or directly into a paver. The mixture is placed and compacted using conventional asphalt paving equipment. Two-unit and single unit trains are preferred in some instances due to their shorter length. Advances in equipment have narrowed the differences in overall quality between multi-unit trains and modern two-unit and single-unit trains (3).

BACKGROUND

CIR is not a new process or procedure; it has been around for over 50 years (1). Today, through innovations of equipment manufacturers, contracting agencies and contractors, remarkable advancements have been made in the cold in-place recycling process. Modern cold in-place recycling equipment can process up to 2 lane miles of material a day (3). The result is a stable, rehabilitated roadway at a total expenditure of 40 to 50 percent less than that required by conventional construction methods (2).

There are two methods of cold recycling asphalt pavements, cold in-place and central plant recycling. Cold in-place recycling is faster, more economical, and less disruptive and environmentally preferable because trucking is greatly reduced. However, in many locations high quality millings are available and central plant recycling can produce a high quality, economical paving material. Central plant methods are appropriate when an existing pavement cannot be in-place recycled and must be removed to allow treatment of underlying materials (1).

There are four very good publications, published in the 1990's, that summarize CIR research and state-of-the-practice up to that time. One of the first publications to summarize CIR was NCHRP Synthesis of Highway Practice 160 *Cold-Recycled Bituminous Concrete Using Bituminous Materials* (4). The synthesis, published in 1990, contains a good description of equipment, processes and early mix design practices. Kearney (3) authored a journal article titled *Cold Mix*

Recycling: State-of-the-Practice in 1997. The article focuses on best construction procedures and practices. FHWA produced a publication on recycling in 1997 titled *Pavement Recycling Guidelines for State and Local Governments* [(5)]. The publication deals with all types of recycling including CIR. In 2001, ARRA published the *Basic Asphalt Recycling Manual* [(1)] or BARM which focused on hot and cold in-place recycling, cold planing and full depth reclamation. A second edition of the BARM is due out in 2012. An NCHRP Synthesis of practice is due out in late 2011. All of these publications provide a good overview of CIR and have extensive bibliographies of early work in the field. Any of these publications provide a good basic description of the construction procedures, equipment, mixture design and quality control associated with CIR with the BARM being the most up-to-date due to its later publication date.

COLD IN-PLACE RECYCLING METHODS

Recycling Trains

The use of cold in-place recycling trains has become the rehabilitation method of choice since the greatest savings typically occur when trucking costs are eliminated. Both single unit and multiple unit trains exist with high productivity and excellent quality control and reliability [(1)].

Single Unit Train

There are several variations of single unit trains available. With the single unit train, the milling machine cutting head removes the pavement to the required depth and cross slope, sizes the RAP and blends the additive with RAP. Single unit trains do not contain crushing units, making control of the maximum size of RAP more difficult. Most single unit trains are capable of producing uniform RAP with a maximum size of 50 mm by operating the cutting head in the down cutting mode and controlling the forward speed [(3)].

A spray bar in the cutting chamber adds liquid additives. The amount is based on volumetrics, determined by the cutting depth and width and forward speed. Roadways that are badly distorted due to rutting, edge drop-off, etc. are not good candidates for CIR with the single unit train because proper additive application rate is difficult to assure [(3)]. Liquid additive is either self-contained in the unit or provided by a tanker truck, which is often towed or pushed by the train. The recycled mix is either windrowed and picked up with a windrow-elevator or placed directly into the paver hopper [(1)].

Advantages of the single unit train are simplicity of operation and high production capacity. The single unit train may be preferred over the multi-unit train in urban areas and on roads with short turning radius due to its shorter length. The main disadvantage of this method is the limitation on controlling maximum size of RAP, oversize material [(5)]. A single unit train with tanker truck is shown in Figure D-1.

Figure D-1. Photo. Single-Unit CIR Recycling Train.

Multi-Unit Trains

The multi-unit train typically consists of a milling machine, a trailer mounted screening and crushing unit and a trailer mounted pugmill mixer as shown in Figure D-2. The milling machine mills the pavement to the desired depth or cross-slope. Millings are deposited into the screening and crushing unit. Maximum size of the RAP is controlled by the opening size of the top screen. All material is passed over the screening unit and oversize material sent to a crushing unit, typically a jaw crusher. Crushed material is returned to the screening unit for resizing (1).

Figure D-2 Photo. Multi-Unit CIR Recycling Train.

RAP proceeds from the screening and crushing unit to the pugmill mixer. The weight of the RAP entering the pugmill is determined by a belt scale on the belt carrying the RAP to the pugmill. The amount of liquid additive is controlled by a computerized metering system, using the mass of material on the belt. Liquid additive is added to the pugmill by a pump equipped with a positive interlock system, which will shut off when material is not in the mixing chamber or the equipment stops. A meter connected to the pump registers the rate of flow and total delivery of liquid additive introduced into the mixture. A twin shaft pugmill blends the liquid additive and RAP into a homogenous mixture (1).

The material leaving the pugmill is either deposited in a windrow or deposited directly into the paver hopper, the same as with single unit trains. Material from the windrow is picked up with a paver with a windrow attachment and is placed and compacted using conventional paving equipment (1).

Construction

Selection of Construction Equipment

When evaluating a project for CIR, several factors can influence the type of equipment best suited for the project. Milling machines currently used with CIR have 10 to 12.5 foot wide cutting heads with extensions up to 4 feet available. Pavements less than 20 feet wide could prove difficult to CIR and maintaining traffic could be a problem [(1)].

Shoulders should be incorporated into the cold recycled mixture. This prevents existing distress in the shoulder, typically cracks, from propagating into the recycled mix. Pavements with shoulders 4 feet wide or less can be recycled in one pass by using an appropriate sized extension to the milling machine. A second approach is to use a smaller milling machine to mill the shoulder and deposit the RAP in a windrow in front of the train. The windrow and full lane is then recycled in one pass. Shoulders wider than 10 feet can be recycled in one pass [(3)].

CIR has been successfully completed on all types of roads, ranging from low volume rural county roads, to city streets, to Interstate highways with heavy truck traffic. However, maintaining traffic through and around the construction zone needs to be considered. This is especially true on roads with limited pavement and or shoulder widths, and or few alternate or bypass routes. The single unit train is sometimes preferred in urban areas with numerous cross streets and business or residential access [(3)].

Field Adjustments to CIR Mix

Changes in gradation of RAP result in changes in the workability of the mix. Adjustments in mix water content or recycling agent content, made by experience personnel, will be necessary to promote good coating and workability. Optimum moisture and recycling agent contents are starting points for construction. Changes to these values should be made judiciously and only by experienced personnel. Rigid adherence to these original recommendations will result in less than optimum performance [(6)].

One of the first things to evaluate on the job is the coating of the mix. Complete coating is desired but may not be possible in all instances. All particles should have some emulsion coating [(7)]. If the mix is not sufficiently coated, the mix water content is increased first. Excessive mix water may cause the asphalt to flush to the surface and will retard curing. Too little mix water results in mix segregation, raveling under traffic or poor density [(8)].

If the mix is adequately coated but lacks cohesion, the emulsion content is increased. Too much emulsion will result in an unstable mix and too little emulsion may cause the mixture to ravel, although minor raveling is generally acceptable [(8)]. Balling of fines in the windrow is usually the result of either excessive emulsion or excessive fines in the RAP [(7)].

The following field test has been used to evaluate cohesion [(5)]. A ball of the material is made by squeezing the material in the fist. If the ball is friable after the pressure is released (falls apart), the mix lacks cohesion. The palm of the hand should contain specks of asphalt, indicating the proper emulsion content. If the hand is stained, the emulsion content could be too high.

The appearance of the mat after initial compaction can give an indication if adjustments to the initial emulsion content are necessary. The mat should be brown and cohesive. A shinny black mat is an indication of too much emulsion [(8)] and excess raveling is an indication of too little emulsion. Adjustments in emulsion are typically made in 0.2% increments and should only be made by experienced personnel. An increase or decrease in the emulsion content is usually followed by an equivalent change in the mix water content to keep the total liquids content the same.

Many agencies have reported letting traffic evaluate the construction of CIR mixtures. Traffic is allowed back on the CIR mixture after a minimum cure period. Mixtures with too much emulsion will rut and mixtures with too little emulsion will ravel. Raveled areas are generally repaired with a fog seal. Rutted areas are removed and replaced or reworked by the contractor [(9)].

Laydown and Compaction

Conventional asphalt pavers, with automatic screed controls for grade and cross slope, are used to place the mix. The paver should be operated as close to the milling machine as possible as this will reduce fluids necessary for placement and reduce aeration time required before compaction [(6)]. The screed should be operated cold as a heated screed causes RAP to stick, tearing the mat. A heated screed will not promote extra density or reduced breaking time [(10)].

Compaction is accomplished with heavy pneumatic and double drum vibratory steel wheel rollers. Cold mix is more viscous than conventional hot mix and requires heavier rollers. It is not possible to compact cold mix to the same density range as hot mix. Well-compacted cold mix could have voids in the 9-14% range or higher [(11)].

Compaction commences after the mixture begins to break. If emulsions are used this could take from 1 to 2 hours depending on environmental conditions. The mix will turn from a brown to a black color when the emulsion breaks. Some agencies heat the emulsion and mix water to the 50 to 60^{o}C range to reduce curing or breaking problems in cool or damp conditions [(8)].

Breakdown rolling is usually accomplished with heavy pneumatic-tired rollers, 25 tons or more, followed by final rolling with 12 or more ton double drum vibratory steel wheeled rollers. Breakdown rolling with pneumatic rollers is continued until the roller “walks out” of the mix. Finish rolling is with the vibratory steel wheel roller to remove roller marks [(1)].

Rolling patterns should be established to determine procedures that result in optimum compaction. Passes with various combinations of rollers should be evaluated. A nuclear density meter or equivalent can be used to evaluate relative increase in density with roller passes. The number of passes that results in no further increase in density should be selected as the rolling pattern. The relative density of the mat can be recorded to assist in compaction monitoring. Rolling procedures should be followed and the mix compacted to a minimum of 95% of the relative density of the control strip. Difficulty in meeting this density could indicate a change in uniformity of the mixture and a new roller pattern and or new control density is needed [11].

The following rolling procedure was recommended by Task Force 38 [11].

> The longitudinal joint should be rolled first followed by the regular rolling procedure established in the control section. Initial rolling passes should begin on the low side and progress to the high side by overlapping longitudinal passes parallel to the pavement centerline. For static rollers the drive drum should be in the forward position or nearest to the paver except on steep grades where the position may need to be reversed to prevent shoving and tearing of the mat. Drums and tires should be uniformly wetted with a small quantity of water or water mixed with very small amounts of detergent or other approved material to prevent mixture pickup. Care should be exercised in rolling the edges of the mix so the line and grade are maintained.

Curing

Compacted CIR mixtures must cure before a wearing surface is placed. Sealing the surface prior to adequate moisture loss can result in premature failure of the CIR mix and or the surface mix [7]. The rate of curing depends on several factors, including temperature and humidity levels. Adequate curing is generally specified as a moisture content of less than 1-1.5% above the moisture content of the pavement prior to recycling. Other agencies require moisture content of the CIR mixture below 2.0% prior to overlay [11]. Typical curing times are generally 10 days to 2 weeks. The addition of lime has been reported to greatly accelerate the curing process [12].

A light application of fog seal may be necessary to prevent raveling of the mix or tire pick-up prior to overlay. The fog seal should consist of a light application of either a slow set emulsion or the emulsion used for recycling. The emulsion should be diluted 50% with water prior to application. Typical application rates are between 0.05 and 0.10 gal/sy [7]. Rolling with a steel wheel roller immediately prior to placement of the wearing surface may be required to remove minor surface rutting and seal the surface.

Wearing Surface

Due to the high in-place air void content of CIR mixtures, a wearing surface is necessary to protect the mixture from intrusion of surface moisture. For low traffic volumes, single and double chip seals have been successfully employed. For higher traffic volumes conventional hot mix wearing surfaces have been employed. A tack coat should be applied at a rate similar to the fog seal to promote good bond between the CIR and the asphalt overlay [1].

The minimum recommended overlay thickness is 1 inch with 1.5 inches preferred. Thin lifts are hard to adequately compact and a poorly compacted surface mix will not protect CIR mixtures from moisture intrusion [13]. The thickness of the overlay should be based on the traffic level and existing support. Some agencies report using the falling weight deflectometer (FWD) to evaluate CIR pavement sections prior to designing overlay thickness. Others [7,8,11] assign an “a” coefficient of 0.25 to 0.35 to the CIR layer for use with the 1986 AASHTO Thickness Design Guide. These numbers are based on resilient modulus testing.

Charmot and Romero (14) evaluated the fracture energy of CIR pavements with known cracking performance using ASTM D7313. Nine projects in Colorado, Nevada and Montana were evaluated. The fracture energy of both the CIR and wearing surface were evaluated. The authors (14) reported that the surface mixture fracture energy had the best match with the rankings of the sites in terms of transverse crack count performance, emphasizing how critical the quality of the roadway surface mixture is to performance.

SUSTAINABILITY

Several recent articles found in the literature to compare the sustainability benefits of CIR used the computer program Pavement Life-Cycle Assessment Tool for Environmental and Economic Effects (PaLATE) (15) to compare the environmental burden Life Cycle Environmental Analysis (LCEA) of employing CIR with the environmental burden of the conventional maintenance options. LCEA provides a more comprehensive assessment of the environmental burden resulting from a specific industrial activity. The LCEA approach differs from traditional environmental analysis in that the environmental impacts are not limited to the immediate geographic vicinity where the activity is occurring (16).

Comparative environmental analysis of pre-selected maintenance treatment options is complicated by the fact that the respective options are rarely equivalent. Each option will have specific structural ramifications, will preferentially relieve specific functional distresses (e.g., reflective cracking), and will respectively extend the life of the pavement for a given period. It is unlikely that such periods will be equal. Comparative environmental analysis tends to imply equivalency, but this may not necessarily be accurate (16).

Data available from many industrial sectors to project an accurate environmental burden is currently limiting. In some instances available data may be outdated, incomplete or nonspecific. Robinette and Epps (17) presented a detailed evaluation of the environmental burden factors in PaLATE in a review of the benefits of flexible pavement recycling. The authors found that environmental burden factors for asphalt cement were different and considerably larger than those reported in two previous studies (18,19).

Alkins et al. (20) reported on the benefits of CIR as a method for the Ontario Ministry of Transportation to meet their goals of reducing greenhouse gas emissions (GHE), as a part of the Kyoto Protocol, and producing sustainable pavements. The study used PaLATE to compare the reduction in consumption of aggregate resources, a non renewable resource, and reduction in greenhouse gasses for CIR pavements versus their conventional rehabilitation procedure. The CIR consisted of milling 100 mm and placing a 50 mm HMA wearing surface compared to milling 100 mm and placing 150 mm HMA wearing surface for the traditional rehabilitation treatment. The reported percent reductions for CIR compared to a traditional rehabilitation are shown in Table D-1. CIR was shown to significantly reduce greenhouse gas emissions and conserve aggregate resources.

Table D-1. Calculated Reduction in GHE for CIR.

Parameter	Reduction
Depletion of Aggregate Resources	62 %
Carbon Dioxide (CO_2) Emissions	52 %
Nitric oxide/nitrogen dioxide (NO_X)	54 %
Sulfur dioxide (SO_2)	61 %

Cross et al. [16] reported on the results of a study that utilized PaLATE to compare the environmental burden of employing CIR with the environmental burden of the conventional maintenance options. Four different treatment options were analyzed, they include: 1) CIR with four inch mill depth and 1.5 inch HMA overlay (CIPR-4), 2) CIR with four inch mill depth incorporating 20% add-stone with 1.5 inch HMA overlay (CIPR-4-AS), 3) mill and fill with 3 inch mill depth and 3 inch HMA overlay placed in two equal lifts (MF-3), and 4) two course overlay consisting of a 3 inch HMA overlay placed in two equal lifts (TCO). Asphalt contents, RAP usage and haul distances of materials are a few of the many input parameters required by PaLATE, shoulders were excluded from the analysis.

The authors [16] noted that PaLATE attributes well over 90% of the environmental burden of HMA and emulsion mixes to the presence of asphalt cement. To examine the effect of asphalt cement-related environmental burden estimates on the LCEA, a sensitivity analysis was undertaken to examine asphalt cement assumptions on the comparative analysis of the four rehabilitation options modeled in the study. The following three scenarios were developed for the analysis, 1) 100% asphalt cement environmental burden as projected by PaLATE, 2) 50% asphalt cement environmental burden, and 3) 0% asphalt cement environmental burden, which assumes no life cycle asphalt cement burden. The results for greenhouse gas emissions for a one mile 24-foot wide section of pavement, excluding shoulders, are shown in Figure D-3.

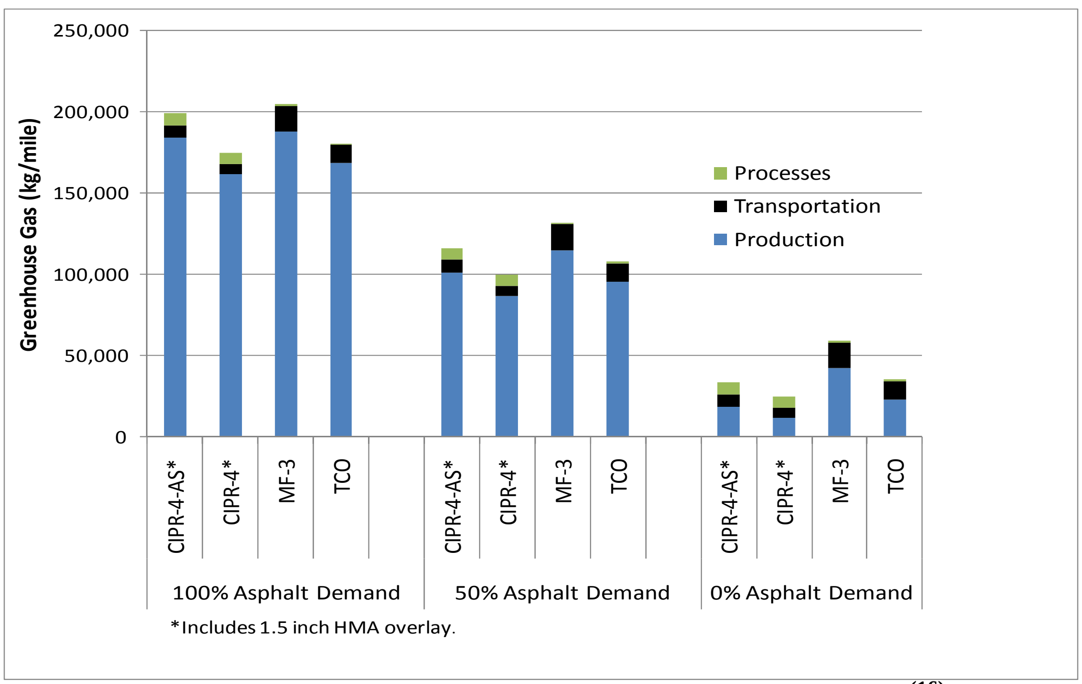

Figure D-3. Bar Chart. Total Greenhouse Gas Emissions [16].

For all environmental outputs evaluated by PaLATE, the authors reported that CIPR-4 generated the least environmental burden regardless of the asphalt cement-related burden and that the addition of add-stone to CIPR-4 (CIPR-4-AS) removes the distinct environmental advantage of CIPR as a maintenance option over TCO [(16)].

PERFORMANCE STUDIES

Several agencies have evaluated performance of their CIR pavement although most have not published formal reports. The majority of these reports are evaluations of a single project. A few agencies have performed system wide evaluations of their CIR pavements and they are summarized below.

New York

In 2010, Cross et al. [(21)] completed a comprehensive review of CIR in New York State DOT (NYSDOT). The report included sections on best practices, a survey of state practice, comparative performance analysis, life cycle modeling and service life predictions.

Cross [(21)] compared pavement condition evaluations on a select group of CIR, mill and fill (MF) and two course HMA overlay (TCO) pavements in New York State to evaluate the relative performance of these rehabilitation-maintenance options. The authors used ASTM D6433-07 Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys to determine the condition (PCI) of 37.7 miles of CIR pavement, 17.6 miles of MF pavement and 31.57 miles of TCO pavement. The pavements were selected to have statistically similar age traffic and came from similar regions of the state, having similar environmental conditions.

The authors [(21)] reported no statistical difference in performance between CIR, MF and TCO pavements, as measured by ASTM D6433 PCI, as shown in Figure D-4. CIR pavement sections exhibited lower total deduct values for Non-Load associated distress than MF or TCO pavement sections and higher total deduct values for Load associated distress than TCO or MF. The differences between treatments however were not statistically significant.

Chesner et al. [(22)] presented the results of a study that examined the effect of daily traffic, truck traffic, base thickness, base plus subbase thickness (total pavement thickness), geographical pavement location (environment and climate) and the condition of the pavement prior to CIR rehabilitation on service life of CIR pavements in New York State. Data used in the analysis were compiled from the 2008 New York State DOT Pavement Management Group Highway Sufficiency Ratings Database, which represented 163 CIPR projects covering a pavement distance of 756 miles. Figure D-5 shows the expected service life of CIR pavements in NY averaged approximately 11 years with the upper and lower confidence limits being 4 and 30 years, respectively. Other significant findings included [(22)]:

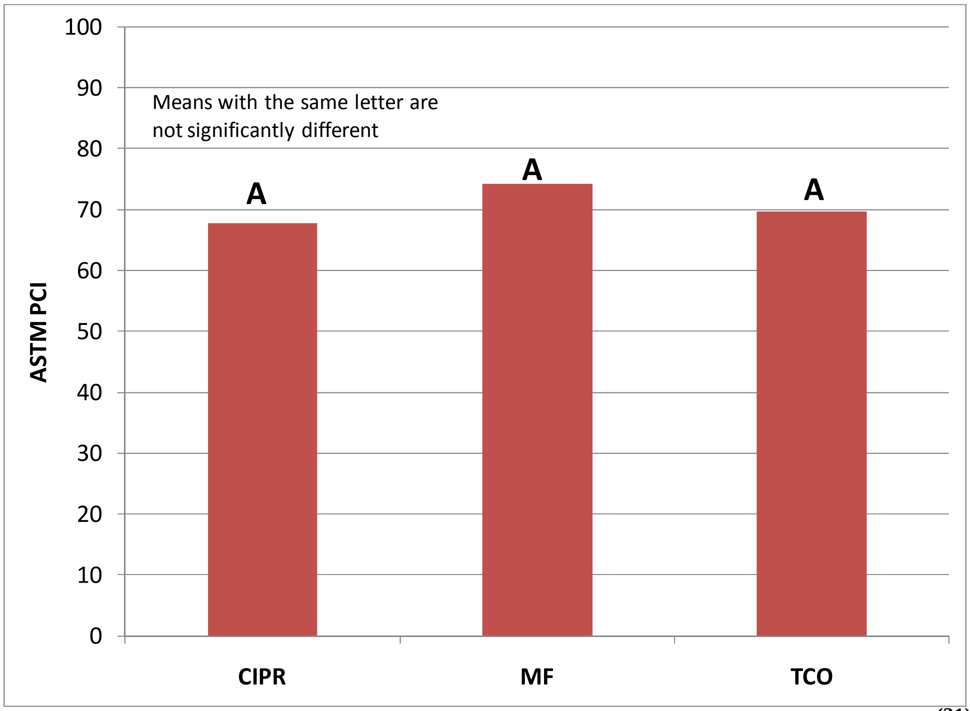

Figure D-4. Bar Chart. Pavement condition index vs. treatment type [(21)].

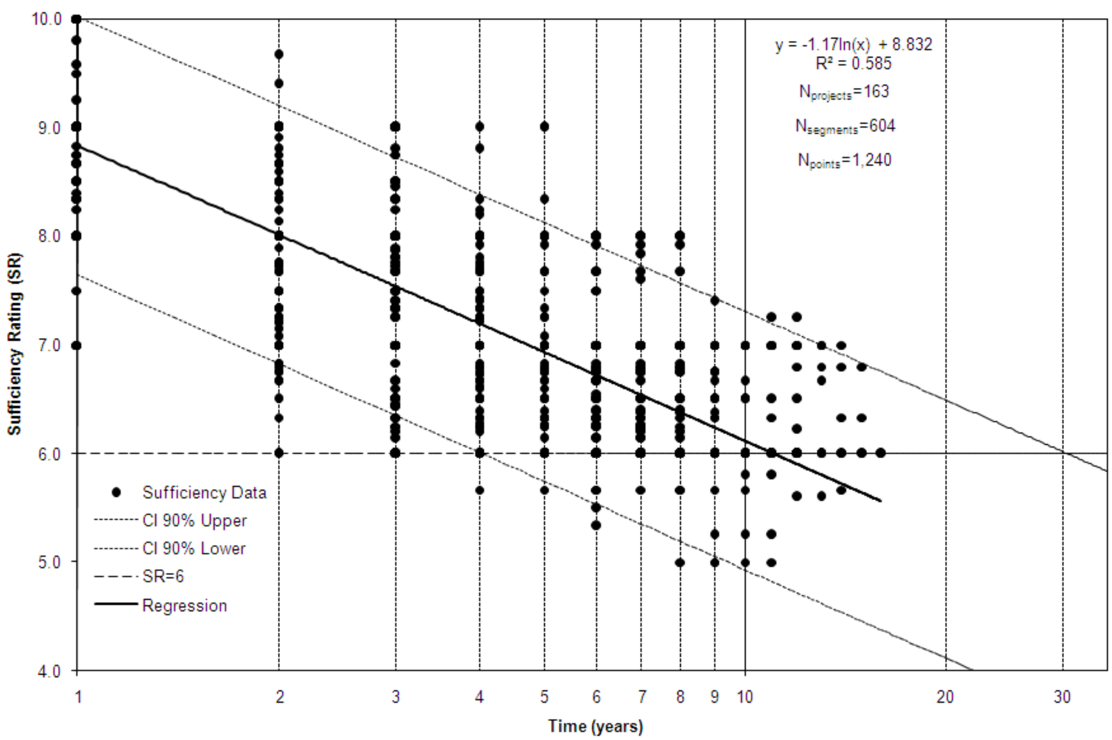

Figure D-5 Scatter Plot. Sufficiency Rating vs. Time [(22)].

1. Pavements constructed with thicker pavement base, base plus subbase and total pavement thickness exhibit longer CIR service lives.
2. Pavements subjected to higher AADT and higher truck traffic (due in great part to the thicker pavement base associated with higher trafficked pavements) exhibit longer service lives than pavements with lower AADT and lower truck traffic.
3. The environment and climate for CIR rehabilitated pavements examined in this study did not significantly affect the expected service life of the pavement.
4. The service life of pavements that were rehabilitated with CIR prior to severe pavement deterioration (Rehab SR ≥ 6.5) were approximately 50 percent longer than those pavements rehabilitated with CIR after severe pavement deterioration.

The authors [22] concluded that when CIR is used on better-designed pavements that have thicker supporting bases and subbases, CIR performance will benefit and the service life of the pavement will be extended. It was also concluded that a general policy of employing CIR as a rehabilitation strategy on low AADT and lightly traveled pavements with low truck traffic may be misleading. CIR pavements lasted longer when applied on pavements with higher AADT and higher levels of truck traffic. However, the primary factor was not traffic but the pavement support structure. Higher trafficked pavements tend to be designed with greater base and subbase thickness, thereby providing enhanced support to the CIR section, which increases the service life of the pavement.

New Mexico

The New Mexico State Highway and Transportation Department (NMSHTD) was one of the first agencies to evaluate their CIR projects system wide. CIR was used to address the problem of reflection cracking of conventional HMA overlays. McKeen et al. [23] reported on the performance of 45 CIR projects constructed from 1984 through 1991. The purpose of the review was to determine if CIR was providing cost savings to the NMSHTD construction program and if CIR pavements provided an acceptable level of service.

The CIR pavements reviewed varied in age from 5 to 11 years. Recycling agents used were high float emulsions, HFE 150, 150P, 300 or 300P. Two of the 45 projects were overlaid with chip seals, the remaining with 1.5 to 3 inches of HMA. The pavements were evaluated for pavement condition index (PCI) with a PCI in the 55 to 70 range considered as critical, needing maintenance. Forty of the 45 CIR projects had a PCI of excellent (> 85), two each were very good (85-70) and good (70-55), and one was listed as fair (40-55). Overall, the performance of CIR was listed as excellent [23].

Cost comparisons were made by comparing CIR to a mill and overlay option. Both initial cost and maintenance costs were combined for a total cost savings. CIR was reported to result in a 22 percent cost savings, on a dollar per square yard basis, compared to a conventional mill and HMA overlay. The authors also reported that the majority of the CIR projects would easily exceed their estimated design life of 10 years [23].

Nevada

The Nevada DOT (NDOT) has one of the most comprehensive CIR programs of any state DOT. NDOT has reported significant cost savings and improved road conditions, system wide, that they directly contribute to their CIR and FDR programs [24].

In 1997 NDOT began a proactive pavement management system with aggressive use of CIR and FDR for pavement rehabilitation. NDOT reported the results of this effort reduced their agency backlog of projects from approximately $600 million in 1986 to less than $290 million per year for 2003-2005. In addition, the condition of the system has improved where currently 93% of NDOT's NHS Interstate System roadways are in the good ride quality category compared to a national average of 73% and 97% of NDOT's NHS "other" category roadways are in the good ride quality catalog compared to a national average of 61%. NDOT reported using CIR on 770 centerline miles or 11% of its system and FDR on 900 centerline miles or 14% of its system. There are no total traffic or truck traffic restrictions on CIR in Nevada, all roadways are eligible for CIR [9].

NDOT [9] claims much of its success to careful project selection, as do many other agencies. CIR is limited to projects with functional failures such as non wheel path longitudinal cracking, block cracking, transverse cracking, poor ride quality, flushing and raveling. Typical milling depth is three inches. FDR is used to correct structural deficiencies such as rutting, fatigue cracking and excessive patching. Regardless of the procedure, inadequate drainage is always addressed.

NDOT [9] recognizes the importance of proper construction procedures; therefore, a knowledgeable contractor's representative is required on site at all times. The contractor is required to correct any areas of non uniform mixture, rutting or raveling of the surface that occurs prior to placement of the overlay unless the defect was caused by a weak structural section. NDOT requires all CIR contractors attend a 2-hour workshop where a CIR project worksheet is reviewed. An abbreviated version of the checklist is shown in Table D-2 [9].

Table D-2. NDOT CIR Checklist [9].

Item	Comments
Premix bituminous paving material	Submit samples of the aggregate and liquid asphalt prior to construction. Do not begin cold recycle operation until mix design is approved.
Plant-mix bituminous pavement	Do not be bin cold recycle operation until dense graded surfacing mix design is approved.
Samples and testing	Submit recycling agent for testing 14 days prior to beginning cold recycle operation Submit quicklime certificates. Submit a 1-quart sample of the water that will be used in the lime slurry. Perform Saybolt Furol viscosity testing in the field to

	determine if binder meets specifications and continue to sample and test binder over the course of project.
Limits of operation	Deliver and place shoulder material a minimum of 3 mi ahead of operation. Stockpile 500 tons of premix material (varies by project).
Core reports	Review the road history and core reports to identify areas where insufficient plantmix depths might be encountered.
Train Calibration	Contact NDOT personnel to calibrate the recycling train 2 days prior to beginning the cold recycle operation.
Rollers	Provide at least two pneumatic rollers and one double drum vibratory steel wheeled roller. Weigh all rollers prior to the cold recycling operation.
Weather conditions	Do not begin cold recycling of existing asphalt concrete pavement: • Until pavement surface is 60ºF and rising. • If it is anticipated that atmospheric temperature will drop below 35ºF within 48 h of mixing. • During stormy weather (The rate of precipitation exceeds the rate of evaporation.)
Milling depth	Measure depth of the milled material across the mat to assure specified depth and uniformity (milling depth may be reduced when insufficient plantmix depths are encountered to minimize intrusion of base or subgrade). Remember to review road history and core reports. Review specification when encountering areas that have unacceptable subgrade.
Asphalt content adjustment	The asphalt content should be adjusted according to the variability of the existing pavement. The estimated application rate of 1.5% by mass of the milled material is for quantity purposes. An experienced person on the contractor's staff will make the adjustment recommendations. Monitor and document any adjustments on the Inspector's Daily Construction Report. This data will be used for performance evaluations. Should raveling or rutting of the cold recycling material occur before the placement of the overlay or surface treatment, the contractor is responsible for taking corrective measures.
Aggregate coating	Good coating is essential but complete coating is not feasible. Squeeze test: Squeeze a handful of mix and observe how it rolls apart. Check for emulsion stains on hand. Rolling method: Place coated aggregate under wheel and observe the movement of the material.

Testing of recycled mix.	Verify graduation from windrow. All milled material must pass a 1.25-in. screen. Sample two per mile. Take minimum of two moisture tests per day from windrow. Verify and check the project rideability specification.
Lay down and compaction	Use a minimum 127 KW powered paver to lay recycled mix (Higher power required for 3-in. depths). Keep paving screed clean to reduce drag.
Lay down and compaction	Wait for cold recycled mixture to break before compaction (early compaction can cause moisture to be trapped and thus cause stripping). Establish a rolling pattern test section and update as required throughout the day. Mark the pavement every ½ h at the paver to establish time of placement. Delay compaction for 1 to 2 h depending upon temperature, wind, and humidity. Use a compaction of steel wheeled, under static and/or vibratory mode, and pneumatic rollers to establish what combination will provide maximum compaction without cracking the recycled surface along the test section. Use a thin lift nuclear gauge to measure the compaction at three locations along the test section. Remember to allow a minimum of 3 h before sunset each day to complete the initial compaction, fag seal, sand blotter, and open the surface to traffic. Perform recompaction between 3 to 15 days after initial compaction. Use caution if the steel wheeled roller is using vibratory mode. Do not perform recompaction when the surface temperature is below 90ºF. Allow the cold recycled mixture to cure a minimum of 10 days before overlaying or placing surface treatment (varies by project).
Fog seal	Fog seal the recycled mat at 0.10 gal/yd^2 to prevent raveling of the surface under traffic. The emulsion used for the fog seal should be diluted 50/50 with water.
Sand blotter	Apply sand blotter on the fog seal before the surface is opened to traffic.

NDOT [9] performed a 20-year life-cycle cost analysis (LCCA) using net present worth of their CIR rehabilitation strategy compared to conventional rehabilitation options. All options received an open graded wearing course. NDOT data was used to determine expected treatment lives. The results are shown in Table D-3. CIR was shown to be the most cost effective rehabilitation strategy.

Table D-3. Results of Life Cycle Cost Analysis for NDOT CIR Rehabilitation Strategies.

Option	Treatment	Rehabilitation	Net Present Worth
CIR	3-in. CIR 2.5-in HMA OGWC	Year 12	$306,000
HMA Overlay	2-inch HMA OGWC	Years 6 and 16	$418,000
Mill & Fill	3-inch mill 3-inch HMA OGWC	Year 12	$415,000

Iowa

The Iowa DOT has sponsored several performance studies on CIRP. The Iowa DOT philosophy of using CIR is as a crack relief layer and designs CIR mixtures differently than many agencies. Jahren et al. [25,26] selected 18 CIR projects from 97 that had been constructed prior to 1996. The selected projects had an AADT from 300 to 2,000 vpd with 5 to 18 percent trucks. Pavements were evaluated using the US Army Corps of Engineers pavement condition index (PCI). The pavements were also evaluated using AASHTO's qualitative method and were averaged with the PCI values to determine a pavement serviceability index (PSI). The performance results are shown in Figure D-6. Using regression techniques, the mean predicted service life (95% confidence limit), was predicted. Based on PCI, the mean predicted service life was 14-19 years and based on PSI, the mean predicted service life was 14-38 years.

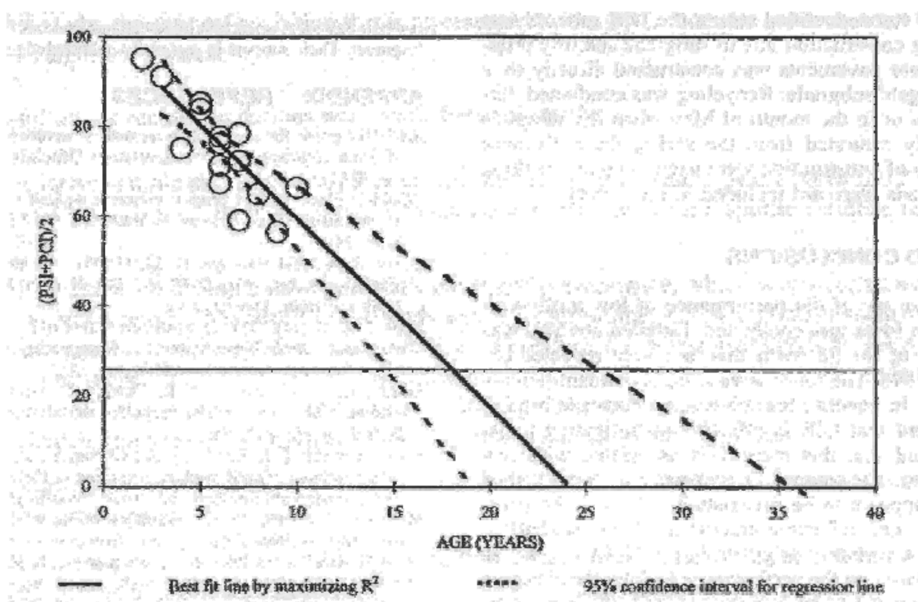

Figure D-6. Scatter Plot. Regression of (PSI + PCI)/2 Versus Age [25].

Kim et al. [27] performed a second performance assessment of CIR pavement in Iowa between 1986 and 2004. This study evaluated 26 pavements, including the 18 evaluated by Jahren [25,26]. The PCI was determined using roughness measurements and an Automated Image Collection System. Soil support was determined using an FWD. Pavements were divided into four categories based on subgrade support and traffic. Subgrade support was listed as either good or poor using a resilient modulus of 5,000 psi as the threshold and high and low traffic using 2000 AADT as the threshold. Regression analysis was performed on PCI to predict pavement service life. Figure D-7 shows the results for all CIR pavements. Predicted service life was 21 to 25 years before reaching a poor service condition (PCI between 40 and 55). Pavements with good subgrade support, as shown in Figure D-8, had an average predicted service life of 34 years compared to 22 years for CIR pavements with poor subgrade support, as shown in Figure D-9. Traffic did not have a significant effect on the results.

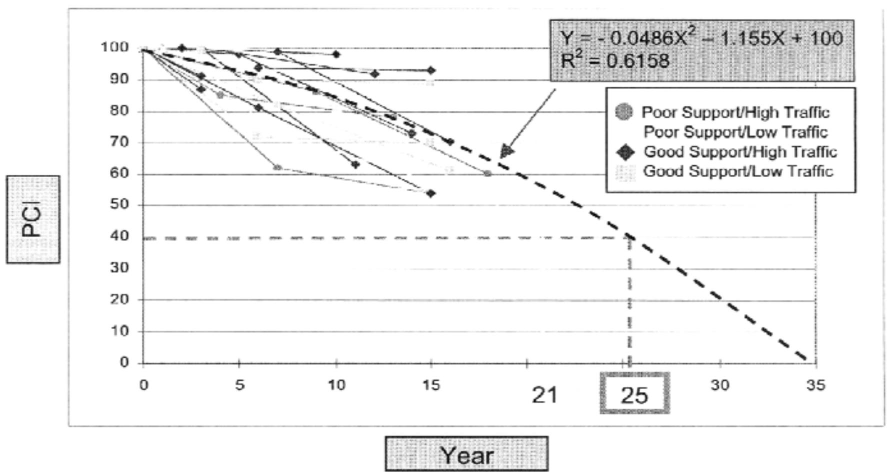

Figure D-7. Scatter Plot. PCI Performance vs. Age, Based on Distress Surveys [27].

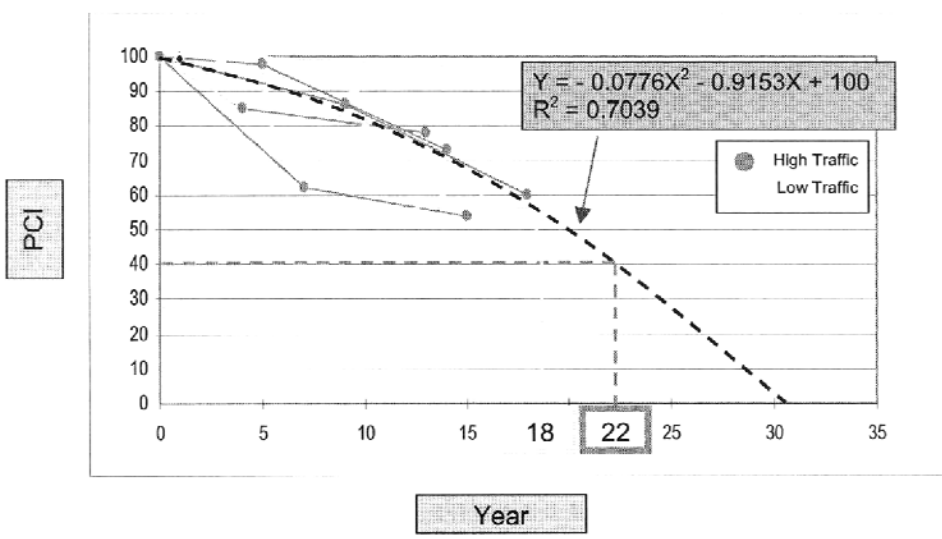

Figure D-8. Scatter Plot. PCI Performance vs. Age, Pavements with Poor Subgrade Support [27].

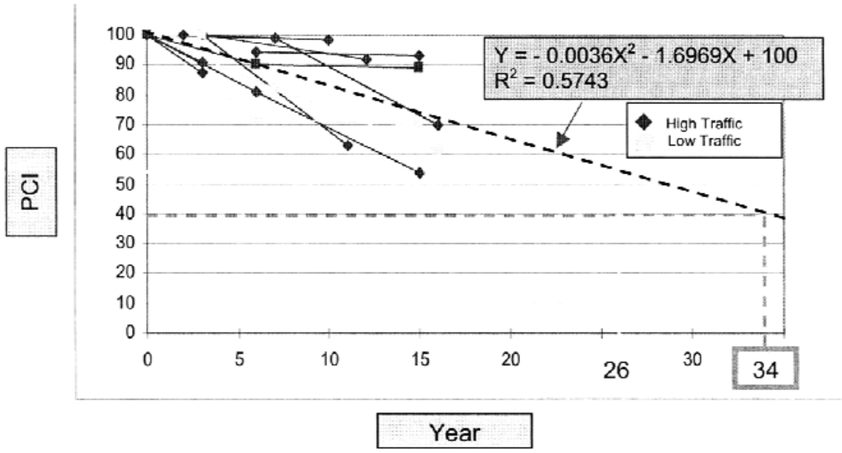

Figure D-9. Scatter Plot. PCI Performance vs. Age, Pavements with Good Subgrade Support [27].

Arizona

Mallela et al. [28] evaluated 17 CIR projects in Arizona. CIR pavements ranged in age from 3 to 21 years with truck traffic up to 1,500 trucks per day. Treatment options consisted of 2 to 3 inches of CIR with either a double chip seal or 2 to 3 inches of an HMA overlay. Performance was based on cracking, rutting and Mays ride meter. Cracking resistance was reported as adequate, 10 to 30 percent cracking, after 10 years of service. Rutting resistance was also reported as adequate after 10 years of service. Pavements with HMA overlays generally had rut depths less than 0.25 inches. CIR with double chip seals had adequate ride (93 to 143 in./mile) with very little change in ride noted for projects greater than 10 years of age. CIR pavements with HMA overlays had less than 93 in. /mile of roughness.

Ontario

Lane and Kazmieroski [29] evaluated CIR pavements for the Ministry of Transportation (MOT), Ontario. To date, the authors reported the MOT has completed 43 CIR projects, 40 with asphalt emulsions and three with expanded asphalt or foamed asphalt. The traditional CIR treatment consists of cold recycling to a depth of 4 inches and placing a single lift of HMA as a wearing surface. Traditional rehabilitation consists of milling to 4 inches and placing 5 inches of HMA in three lifts. Performance was measured using PCI and IRI values. The performance of the two rehabilitation techniques was reported as similar with the conventional mill and overlay technique being marginally smoother to start, resulting in marginally better performance. Service life for the mill and overlay treatment was estimated at 18 years compared to 15 years for CIR. Life cycle cost analysis over a 50 year analysis period indicated CIR was more cost effective.

Pennsylvania

Morian et al. [30] evaluated the performance characteristics and cost effectiveness 23 CIR pavements in Pennsylvania. Eleven of the CIR projects were performed over concrete pavements and the remaining 12 were over HMA or aggregate bases. Traffic ranged from 1,000 to 17,000

ADT with estimated daily ESALs of 5 to 584. Typical CIR depth was three inches. Analysis was based on the service life of the pavements and number of years before resurfacing.

For CIR sections on concrete pavements, reflective cracking did not appear until the third year after construction and significant cracking did not appear until six years after construction. The authors reported that CIR sections had two to three times less cracking than conventional HMA overlaid pavements of similar age and traffic [30].

CIR was reported to be a proven effective means of extending pavement life for highways with up to 13,000 ADT and 200,000 annual ESALs. Projects were reported to have documented service lives of up to 160% (16 years) of the 10-year design life provided by conventional mill and HMA resurface projects in the same geographical area. CIR was reported as a cost effective rehabilitation strategy, approximately one to two thirds the local cost of conventional HMA materials while providing superior performance [30].

In a second study, Morian et al. [31] compared the performance and cost effectiveness of four different rehabilitation procedures from 49 pavements in northwestern Pennsylvania. The rehabilitation procedures consisted of 1) milling and placement of an HMA overlay, 2) leveling and placement of an HMA overlay, 3) application of a stress-absorbing membrane interlayer (SAMI) and placement of an HMA overlay, and 4) CIR with placement of an HMA overlay.

Analysis was based on pavement condition and roughness (IRI). The analysis indicated that CIR sections had performed significantly better than the other three methods. CIR was also shown to be the most cost effective treatment followed by SAMI sections. CIR sections exceeded the pavement life of SAMI sections by four years and of the other treatments by six years [31].

Montana

Hill [32,33,34] reported on the performance of 14 CIR pavements in Montana. Six CIR pavements received a chip seal, six received an HMA overlay and two received both. Of the eight CIR projects that received an HMA overlay, six were reported to have performed well and one performed poorly. The poor performing project was reported to be related to poor construction and harsh climate. Based on 10 years of performance, Hill reported that CIR pavements with HMA overlays tend to stay smooth and have rut depths equivalent to conventional HMA mill and fill treatments. CIR projects with a chip seal did not perform as well as those with HMA overlays. Of the eight projects that received a chip seal, 4 were reported to be performing well and 3 performed poorly due to rutting.

Washington

Uhlmeyer [35] reported on CIR performance in Washington. Washington State DOT (WSDOT) has performed CIR on 16 projects since 1981. CIR is typically performed on low volume roads with ADTs less than 5,000 vehicles per day (vpd) in Washington; however, a 4-lane divided highway with an ADT of over 10,000 vpd was successfully recycled as well. Five of the 16 pavements were rehabilitated prior to the study but only three due to distress in the CIR layer. Of the 11 remaining pavements, seven had a pavement structural condition (PSC) in the 90s, three were between 79 and 89 and one had a rating of 43. PSC scores range from 0 (poor) to 100 (good). CIR pavements in Washington were reported to show excellent performance with no failures in any of the 16 projects which ranged in age from 3 to 15 years. A life cycle cost analysis indicated CIR was a cost effective rehabilitation procedure.

Central Federal Lands Highway Division

Voth [(36)] reported on the performance of CIR projects performed by the Central Federal Lands Highway Division (CFLHD) of FHWA. CFLHD has been performing CIR for over 30 years. Although no formal reports exist, Voth reported excellent performance on their CIR projects. All CIR projects were expected to exceed their 20 year design life with preventative maintenance. Of the 25-30 projects constructed to date, only one was not still in service [(36)].

PROJECT SELECTION

Most performance studies reviewed considered project selection a key to pavement performance. The BARM [(1)] recommends CIR for reconstruction of any flexible pavement where the need arises from structural failures, including transverse cracking, wheel rutting, potholes, surface irregularities, or a combination of the above. With the exception of wheel path rutting, most practitioners would classify the above distresses as functional failures. NDOT [(9)] recommends CIR be limited to pavements with functional failures such as non wheel path longitudinal cracking, block cracking, transverse cracking, poor ride quality, flushing and raveling and that drainage issues always be addressed.

Cross et al. [(21)] reported that CIR treated pavements had less functional distress but more structural distress than mill and fill or two course HMA overlay pavements. However, in the study the differences were not statistically significant. Numerous other agencies report CIR should be limited to repair of functional failures or have reported poor performance when structural failures brought on by weak subgrades were treated by CIR. Table D-4, adopted from ARRA, is a listing of pavement distresses that can be used to screen pavements eligible for CIR [(37)].

In addition, several agencies reported additional criteria for CIR project selection. Many agencies limit CIR to low and medium trafficked pavement and avoid routes with heavy truck traffic. NDOT [(9)] recommended CIR be limited to low to medium trafficked pavements initially until adequate experience is gained and then these traffic restrictions could be dropped. Chesner et al. [(22)] reported that CIR pavements in New York performed better on higher trafficked pavements than on lower trafficked pavements. They also reported that CIR pavements with thicker bases, better support, had longer service lives. The authors [(22)] concluded that a general policy of employing CIPR as a rehabilitation strategy on low AADT and lightly traveled pavements with low truck traffic may be misleading. They concluded that the primary factor is not traffic but the pavement support structure. Higher trafficked pavements tend to be designed with greater base and subbase thickness, thereby providing enhanced support to the CIR section, which increases the service life of the pavement.

Table D-4. Applicability of CIR to Pavement Distress.

Pavement Conditions		
Distress	**Criteria**	**CIR Applicability**
Ruts	< 3/8 inch	Yes
	> 3/8 inch	Possible[1]
Cracking	Fatigue	Possible[1]
	Longitudinal	Yes
	Transverse	Yes
Surface Defects	Dry	Yes
	Flushing	Yes
	Bleeding	Yes
	Variable	Yes
Raveling	All levels	Possible[1]
Potholes	All levels	Possible[1]
Stripping	High degree	Possible[1]
Ride	Poor	Possible[1]
Drainage	Poor	No

[1]Further investigation, described under each heading, would be necessary to determine CIPR applicability.

Several agencies use FWD testing or dynamic cone penetrometer testing during project selection to identify subgrade and drainage issues. Jahren (38) recommended using the dynamic cone penetrometer to evaluate subgrade conditions. For glacial till soils in Iowa, recommended minimum DCP blow counts for the top 12 inches of subgrade were 6 blows per inch. Glacial till subgrades with a DCP between 4 and 6 were considered marginal and less than 4 indicated soft subgrades unsuitable of CIR. This assumed a minimum of 2 inches of HMA be left in place to support the recycling equipment. Nevada (9) recommends a minimum of 1.5 inches of HMA remain in place after milling to support the recycling equipment.

Many states reported the importance of qualified contractors and require an experienced contractor representative be on site to adjust recycling additive contents as changing conditions dictate. Nevada (9) requires all CIR contractors attend a training course. The CIR checklist used at the training classes was shown in Table D-2.

MIX PROPERTIES

MEPDG Input Parameters

May (39) evaluated use of the MEPDG for cold mixes such as CIR and FDR. May reported several challenges to using the MEPDG with these mixes. First, May reported that measured dynamic modulus values for CIR were considerably higher, especially at higher temperatures and lower frequencies than the default value of 30,000 psi, leading to very conservative designs. May also reported there was no easy way of altering the fatigue damage equations and rutting equations for the different mix properties cold mix layers compared to HMA layers. This resulted in similar performance between mixes, contrary to observed results in some cases.

Resilient Modulus

McKeen [40] evaluated the performance of 28 CIR projects in New Mexico. Eighteen projects used HFE as the additive, seven used HFE with slaked lime slurry and three used cement-fly ash blends. The projects were evaluated for air voids, resilient modulus and indirect tensile strength. In-place air voids ranged from 6.3 to 14.8 percent. Resilient modulus from field cores ranged from 360,000 psi to 1,124,000 psi and indirect tensile strength ranged from 48.6 psi to 132.4 psi. McKeen [40] reported a good correlation between indirect tensile strength and resilient modulus, indicating that resilient modulus could be estimated from the much simpler indirect tensile strength test. Pavements with resilient modulus and indirect tensile strengths exceeding 870,000 psi and 116 psi, respectively, were found to be prone to excessive cracking. McKeen recommended mix design samples be long-term aged using the SHRP protocol and tested to be sure they do not exceed these threshold values.

Morian et al. [30] determined back-calculated resilient modulus of cold recycled projects in Pennsylvania. The authors reported that resilient modulus values ranged from 50,000 to 470,000 psi. Lower modulus values were attributed to central plant rather than CIR mixtures. On projects where CIR was used, resilient modulus values were reported to be similar to HMA with a range of 250,000 to 450,000 psi. The authors reported that modulus appeared to decrease slightly with time, as shown in Figure D-10. However, the authors pointed out that the relationship was not developed from one project tested over time but by comparing projects with different ages. The authors also reported that resilient modulus appeared to be stress sensitive with higher stress resulting in a stiffer response or modulus.

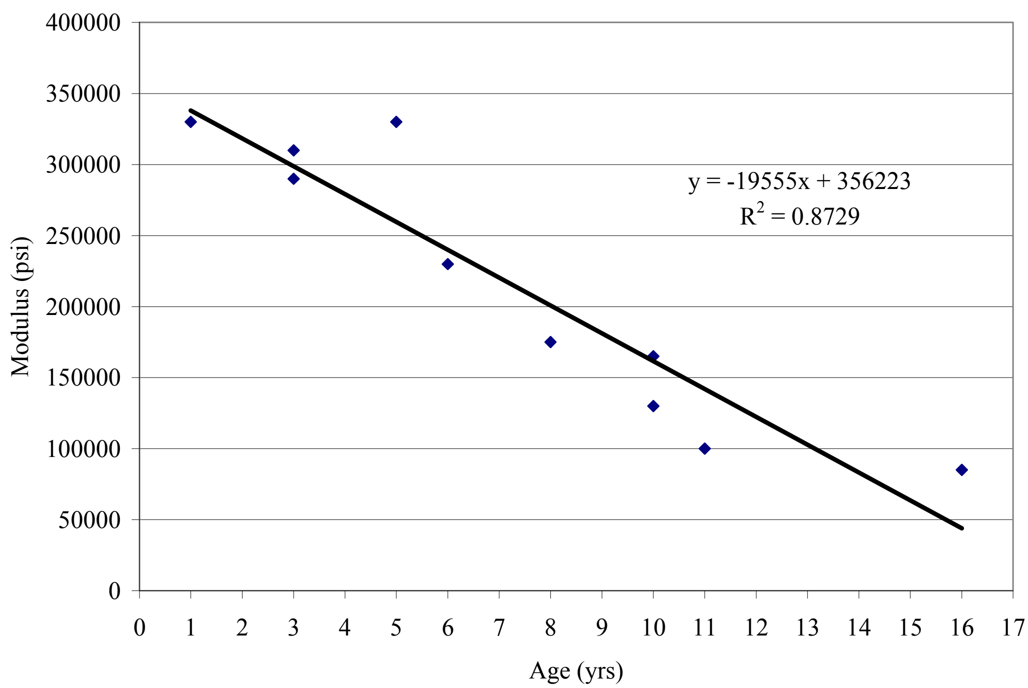

Figure D-10. Scatter Plot. Plot of Back-calculated Modulus by Age at 9,000-lb Load [30].

Chen [41] reported on back-calculated resilient modulus obtained from 24 CIR projects in Iowa. Pavements ranged in age from 3 to 21years. Resilient modulus values ranged from a low of 500,000 psi to a high of 14,500,000 psi. The values seem very high compared to the literature. Chen reported that CIR resilient modulus and air void content had a significant effect on pavement performance but that cumulative traffic did not. CIR mixtures with lower resilient modulus were reported to perform better. The stiffness of the foundation was not found to have a significant effect on pavement performance. This fits with the Iowa DOT philosophy of using CIR as a crack relief layer.

AASHTO Structural Layer Coefficient

One of the main reasons for determining CIR resilient modulus appears to be for helping determine an acceptable structural layer coefficient or "a" coefficient for use in thickness design. Epps [4] reported structural layer coefficients determined from resilient modulus of CIR cores ranged from 0.22 to 0.49 with an average value of 0.35. Reports indicate Kansas uses CIR structural layer coefficients between 0.25 and 0.28 for design [42]. Uhlmeyer [35] reported WSDOT uses a CIR structural layer coefficient of 0.30, the same as Oregon and Pennsylvania DOTs. The CFLHD takes a conservative approach and uses a structural layer coefficient of 0.28 but Voth [36] reported values of 0.35 were applicable based on field test results.

Nevada [9] reported elastic modulus values for CIR mixtures with 1.5% CMS-2s vary between 150 to 400 ksi. According to the 1993 AASHTO Design Guide for Pavement Structures, this corresponds to a structural layer coefficient of 0.25 to 0.41. NDOT [9,24] has adopted a structural layer coefficient for CIR of 0.26 based on backcalulation from extensive FWD testing and laboratory modulus testing.

McKeen [23] reported New Mexico DOT was using a structural layer coefficient of 0.25 for thickness design. Based on field performance evaluations and laboratory testing McKeen recommended the structural layer coefficient be immediately raised to 0.30 with consideration of increasing to 0.35. New Mexico DOT now uses a structural layer coefficient of 0.30 [42].

Air Voids

Chen [41] reported on in-place air voids from 24 CIR projects in Iowa. The pavements ranged in age from 3 to 21years. Air void contents of the CIR layer ranged from 4.5 to 14.3 percent. Chen reported that CIR mixtures with air voids between 6 and 12 percent performed better than CIR mixtures with air voids outside this range.

McKeen [40] reported in-place void contents from 28 CIR projects in New Mexico. The pavements ranged in age from 5 to 11 years. Eighteen projects used HFE as the additive, seven used HFE with slaked lime slurry and three used cement-fly ash blends. The original density or air void contents were compared to current density and air void contents obtained from cores of the pavements. In-place air voids ranged from 6.3 to 14.8 percent. McKeen found that pavement density or air voids had not changed over time and that there was no correlation between density and pavement age. McKeen concluded that densification under traffic was not occurring and that current compaction requirements of 96% of laboratory molded density was adequate.

Cross [43] reported on CIR mix properties and their relationship to performance from 11 projects in Kansas ranging in age from 1 to 5 years. In-place air voids of the CIR layer ranged from 6.9 to 13.3 percent. Compaction of the CIR layer was considered adequate based on the USACE recommendation of less than 14% voids for CIR mixtures [44]. Air void contents of HMA overlays ranged from 3.3 to 9.2 percent. Air void contents of CIR were not related to pavement performance but pavements with high in-place air voids in the HMA overlay performed poorly.

RECYCLING ADDITIVES

Many different types of recycling agents are available for use with CIR. Muncy [45] stated that recycling agents for CIR using a recycling train requires an emulsion that allows immediate compaction, high water resistance, flexibility to reduce reflective cracking, mix quickly and soon become tacky for adequate densification and opening to traffic. The most popular recycling agents for CIR are polymer and non-polymer high float emulsions and cationic medium and slow set emulsions. Two relatively new additions are engineered emulsions which are typically modified cationic slow set emulsions and expanded or foamed asphalt [45,46].

Conventional Asphalt Emulsions

According to Croteau [47], CIR mixtures may be produced with the addition of only 1.0% added residual bitumen whereas comparable mixtures produced using uncoated aggregates require at least 4.0 to 5.0% residual binder. Thus the aged binder in the RAP contributes to the buildup of cohesion in the mix. The contribution of old asphalt to the buildup of cohesion in the new CIR mix depends on the fluxing characteristics of the recycling binder, weather, gradation of the RAP, asphalt content of the RAP and softness of the asphalt. Old binders with a penetration of less than 20 are inert and will not react whereas binders with a penetration above 45 are still active and will contribute significantly to the cohesion of the new mixture.

Croteau [47] states that high float emulsions are always manufactured with a small amount of fluxing agent to promote coating and consequently, soften the old aged binder. Coating of dense graded material with high float emulsions tends to be selective with the smaller particles coated with a thick film of asphalt while the larger particles are partially coated. Cationic slow set emulsions tend to coat more of the fine portion of the mix with a more uniform, thinner film thickness. Cationic slow set emulsions can be made with or without a fluxing agent. For all bituminous additives, the coated fine material acts as a mortar that binds the material together. Pozzolanic material, lime or cement, can be added to cationic materials to act as a catalyst to accelerate the buildup of cohesion.

Polymer modification can enhance the positive characteristics of emulsions. O'Leary and Williams [6] reported that the use of polymer modification of high float emulsions in New Mexico resulted in higher cohesion of the binder and more rapid strength gain. Other advantages were listed as increased resistance to moisture damage, reduced raveling and reduced cracking. Polymer modification allowed the use of softer residual binders which are better able to soften the aged binder in the RAP.

Engineered Emulsions

All emulsions can be "engineered" to provide selective properties for a given project. Properties that are engineered include mixing and coating, breaking times, curing times, moisture resistance, softening ability of the emulsion and stiffness properties of the residual binder. Properties are adjusted by numerous techniques including varying the residual binder content, stiffness of the residual binder, polymer modification, ph, and adding a fluxing agent, to name a few. There are limits; however, as to how much modification can be accomplished with a given grade or classification of a recycling agent.

Although the term engineered emulsion does not really apply to a specific product or manufacturer, it has come to describe a modified or engineered emulsion commonly referred to as CSS(Special). Road Science, formally SemMaterials and Koch Materials, originally developed and marketed CSS(Special) under the trade name ReFlex®. According to Road Science [(48)], ReFlex® is a solventless emulsion that uses a new chemistry to improve coating, provide earlier breaking and increase early strength gain. The formulation allows higher total asphalt contents, thicker asphalt films, higher early strength, stronger mixtures and better coating, all of which are claimed to lead to better performing more durable CIR pavements.

Forsberg et al. [(49)] reported on a comparison between a conventional emulsion and an engineered emulsion (CIR-EE) supplied by Koch Materials on a project in Blue Earth County, Minnesota. The grade of the conventional emulsion was not identified. Emulsion contents were 1.5% for the conventional emulsion and 3.25% for the CIR-EE. Table D-5 shows the mix design information for a medium RAP gradation. The CIR-EE mix easily passed the raveling test where the conventional emulsion failed the test. The CIR-EE mix has lower air voids, lower Marshall stability and higher retained stability. It is interesting to note that the conventional emulsion did not meet current moisture susceptibility requirements and lime would typically be added. Lime could increase the early strength of the conventional emulsion.

Table D-5. Mix Design Results, Medium RAP Gradation [(49)].

	Emulsion Content, %	Air Voids, %	Stability, lbs	Retained Stability, lbs	Percent Retained
Conventional CIR	1.3	14.3	2093	876	42
	2.0	13.2	2112	1060	50
CIR-EE	2.0	11.9	1827	1428	78
	2.7	10.6	1824	1680	92
	3.4	9.1	1635	1361	83

Results from field stiffness gauge measurements indicated the CIR-EE was 34% stiffer after one day compared to the conventional emulsion after one week. FWD testing performed immediately after placement of the CIR and 10 months later, after placement of a 2-inch HMA overlay, indicated that CIR-EE was 41 and 36% stiffer than the conventional emulsion section, respectively [(49)].

The authors [49] reported that the CIR-EE section did not ravel, was compacted earlier, gained strength quicker and had better thermal cracking properties than the conventional emulsion section.

Lime

Croteau [47] reported that pozzolanic materials such as lime or cement are added to cationic materials to act as a catalyst to accelerate the buildup of cohesion. Increased cohesion improves moisture resistance. The benefits of lime as an anti-strip agent are well documented. Nevada DOT [9] reported the average anticipated life expectancy of CIR with lime slurry is 15 to 20 years compared to 10 to 12 years without lime slurry.

Cross [50] evaluated the effect of hydrated lime, added as slaked or hot lime slurry, and as dry hydrated lime on properties of CIR mixtures. Samples were made with 1.5% CMS-1, 1.5% CMS-1 with 1.5% hydrated lime (CMS+HL) and 1.5% CMS-1 with 1.5% hydrated lime produced by slaking 1.14% quicklime (CMS+QL). Hydrated lime was mixed with mix water prior to adding the emulsion and mixing the ingredients. Mixtures were tested for tensile strength, conditioned tensile strength and APA wet and dry rut depth.

Figures D-11 and D-12 show the results of the indirect tensile strength testing and tensile strength ratios. Lime increased the dry tensile strength and TSR. Introducing lime as slaked lime slurry improved mix properties more than adding the same amount of hydrated lime as slurry. Wet and dry APA rut depths are shown in Table D-6. Lime significantly increased the CIR resistance to permanent deformation and wet rut depths. Again, lime introduced as slaked lime slurry improved mix properties more than adding the same amount of hydrated lime as slurry [50].

A second study by Cross [51,52] was performed evaluating the effectiveness of hot or slaked lime slurry (HLS) on CIR mixtures made with CMS-1, CSS-1 and HFE-150. Mix properties evaluated included resilient modulus, conditioned resilient modulus and AASHTO T 283. Test results showed similar tensile strengths for the mixtures, regardless of base emulsion with a tensile strength of 225 kPa. HFE-150 had the highest resilient modulus (270 MPA) followed by CSS-1 (185 MPa) and CMS-1 (97 MPa). Figure D-13 shows the percent increase in tensile strength and resilient modulus with the use of 1.5% hydrated lime added as hot lime slurry. Hydrated lime, added as hot slurry, significantly increases the tensile strength and stiffness of CIR mixtures. APA testing showed that hydrated lime significantly reduced both wet and dry APA rut depths, as shown in Figure D-14.

Table D-6. Effect of Hydrated Lime on Wet and Dry APA Rut Depths [50].

Additive	APA Dry Rut Depth (mm)	APA Wet Rut Depth (mm)
CMS-1	6.5	13.9
CMS-1 + Hydrated lime	4.5	5.6
CMS-1 + Quicklime slurry	3.8	4.8

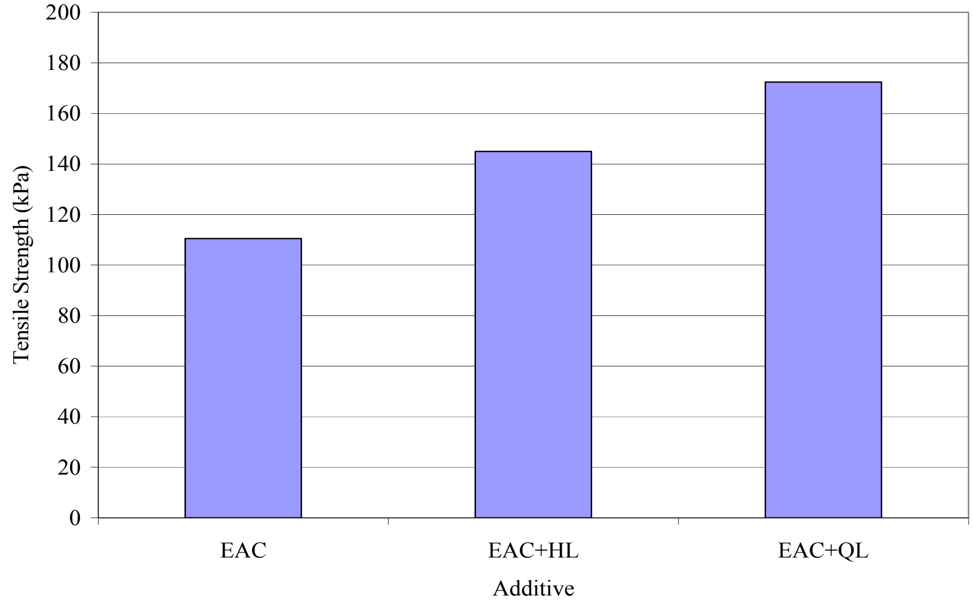

Figure D-11. Bar Chart. Indirect Tensile Strength vs. Recycling Additive [50].

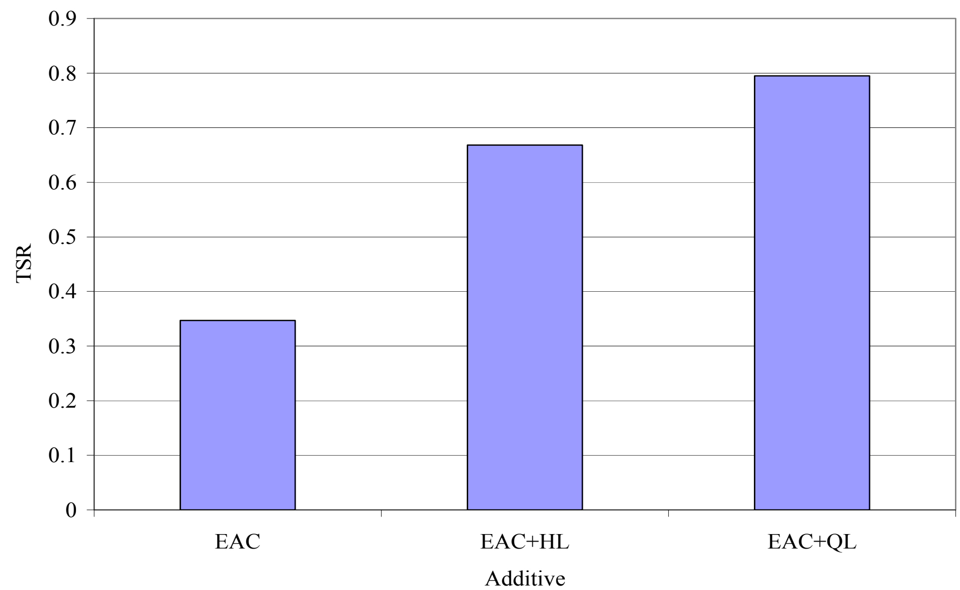

Figure D-12. Bar Chart. AASHTO T 283 Tensile Strength Ratio vs. Recycling Additive[50].

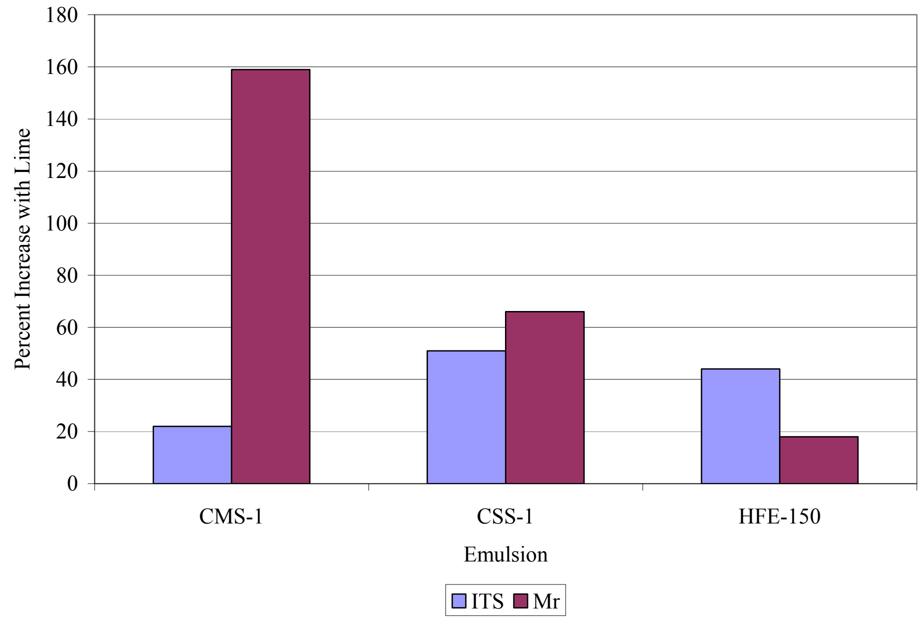

Figure D-13. Bar Chart. Percent Increase in Indirect Tensile Strength and Resilient Modulus, with Lime [51].

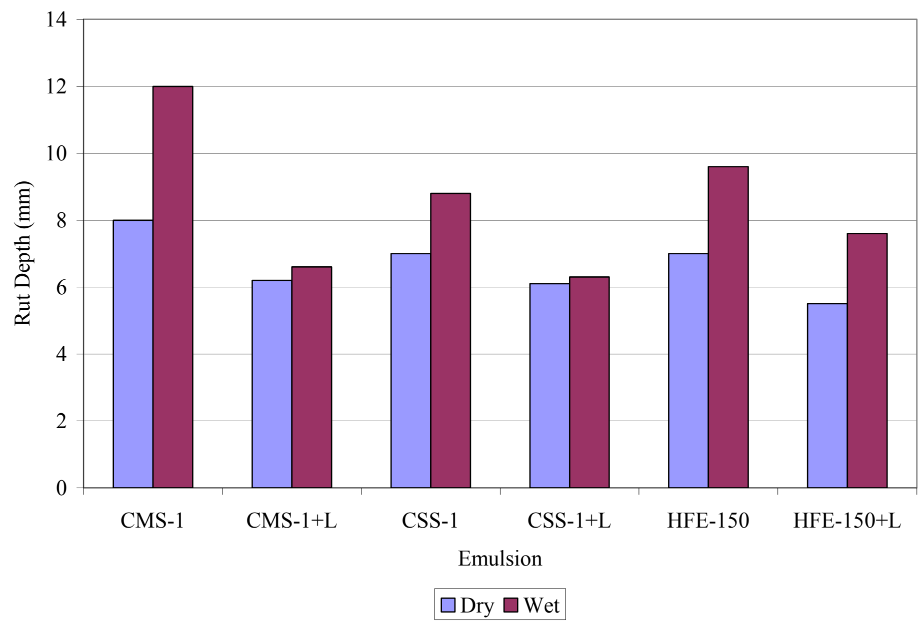

Figure D-14. Bar Chart. Effect of Lime on Wet and Dry APA Rut Depth [51].

Virgin Aggregates

Uncoated aggregates are sometimes added to CIR mixtures to improve gradation. Croteau [47] reported that the gradation of the mineral aggregate has a direct influence on the mechanical properties of CIR mixtures. According to Croteau, coarse aggregate gradations are not usually found whereas sandy or fine gradations are more common. Sandy gradations tend to produce tender mixtures that are susceptible to permanent deformation and dense gradations produce excellent results. Croteau assessed gradation of mineral aggregate in the RAP using the No. 4 sieve. The following recommendations were made [47].

% Passing No. 4 Sieve	Action
45 -65	No corrective aggregate
65-75	Corrective aggregate if compacted voids < 9.0%
> 75	Corrective aggregate required

From a study evaluating CIR mix properties and their relationship to performance from 11 CIR projects in Kansas, sandy mixtures (79 to 91% passing No. 4 sieve) performed poorly with rutting being a major distress mode [43]. The pavements ranged in age from 1 to 5 years. Aggregate gradation was found to have an impact on performance. However, it should be noted that the aggregate consisted of a sand-gravel mixture with an average of 15 percent coarse aggregate (+No.4) and an average uncompacted void content of 39.1% of the recovered fine aggregate [43].

Croteau [47] also stated that corrective aggregate is often required for RAP with more than 5.5 to 6.0% asphalt prior to recycling. Dense graded aggregates, rather than chips, were recommended to correct excessive asphalt contents due to increased surface area.

MIX DESIGN PROCEDURES

Lack of a nationally recognized mix design procedure for CIR is often listed as a deterrent to the expanded use [53]. However, it should not be interpreted that there are no CIR mix design procedures available. Epps [4] reported the results of a survey on CIR mix design practice conducted by ARRA. From the ARRA survey, Epps reported that 20 of 30 agencies using mix design procedures for CIR used Marshall and the remainder used Hveem procedures. Prior to Superpave, there were 38 states reported as using some form of the Marshall method and 10 states reported as using some form of the Hveem method for HMA mix designs [54].

One of the first published mix design procedures was developed for the Oregon DOT. The procedure was described by Rogge et al. [55] and later updated by Scholz et al. [56]. Oregon reported that CIR mixtures with less than 1.2% emulsion tended to ravel and mixtures with more than 2% tended to rut. Therefore, 1.2% was used as a starting point for optimum emulsion content and adjustments were made based on the softness of extracted asphalt, gradation of millings and percent recovered asphalt. Adjustments for softness ranged from 0 to 0.3%, adjustments for gradation ranged ±0.3% and adjustments for asphalt content ranged from 0 to -0.3%. Tests on compacted CIR samples were not performed [55].

Epps [4] reported the most developed CIR mix design procedures were those by California, Chevron, USACE, Nevada, New Mexico, Oregon, Pennsylvania, Purdue University, Texas and the Asphalt Institute. Epps reported that the basic procedures for these methods were similar and many of these methods are discussed in detail in Epps' report [4]. These methods/procedures are not heavily used today and will not be discussed further.

The BARM [(1)] contains a chapter on mix design practice; however, a specific procedure is not provided but the basic steps are discussed. The basic steps are listed below [(1)].

1. Obtain samples of RAP from field.
2. Determine RAP gradation, RAP binder content, gradation of extracted aggregate and aged binder properties.
3. Select amount and gradation of additional granular material, if required.
4. Select type and grade of recycling additive.
5. Estimate recycling additive demand.
6. Determine pre-mix moisture content for adequate coating.
7. Test trial mixtures; initial cure properties, final cure properties and moisture sensitivity.
8. Establish job mix formula.
9. Make adjustments in field.

In 1998, a joint task force from AASHTO, AGC and ARTBA conducted a review of CIR practice. As a part of their review [(11)], the task force published recommended mix design procedures using both Marshall and Hveem equipment. The procedures are basically the same with allowances for differences in the respective equipment. These procedures are rarely used today having been replaced with more recent methods that use Superpave technology and will not be further discussed.

Lee et al. [(57)] determined if Superpave equipment and procedures could be used for CIR mixtures and found that the voids analysis methods of Superpave were applicable to CIR mixes if mixing and compaction temperatures were adjusted. Lee recommended samples be compacted at ambient temperatures with emulsions heated to typical delivery temperatures (140^{o}F). Optimum moisture or total liquids were determined using typical emulsion content and varying total liquids. Samples were compacted in the SGC and the optimum total liquids determined, usually at maximum dry density. Next, samples were compacted at varying emulsion contents with adjustments in water to keep total liquids constant. A voids analysis was then performed and the optimum emulsion content determined.

Cross [(58,59)] extended Lee's work by determining recommended N_{design} compactive efforts for CIR mixtures. Recommended values of N_{design} were 30 gyrations for samples that are compacted immediately after mixing and prior to the emulsion breaking and 35 gyrations for samples compacted after the emulsion begins to break. Shape of RAP particles were reported to influence results.

Road Science Procedure

Road Science developed a mix design procedure to go along with their ReFlex® emulsion cold in-place recycling system. The mix design method has been changed slightly over the years based on their continued evaluation and research efforts. Many state DOTs have adopted the basic procedure and published standard methods. However, as a part of Road Science ReFlex® system, they supply the mix design to the contractor. There are slight differences in the procedures found on agency web sites. Two sources are the Kansas DOT [(60)] and the Missouri DOT [(61)]. The basic procedure is described below.

The mix design is performed on RAP obtained from crushed cores from the project site. The RAP is dried to a constant mass at 40°C. Mix designs are performed on the middle gradation and either the coarse or fine gradation, as shown in Table D-7. Four 100 mm diameter by 61 to 66 mm high samples are prepared at each emulsion content. Moisture that is expected to be added at the milling head is added to each mix, typically 1.5-2.5%. Water and any other additives are mixed thoroughly and then emulsion is added and mixed for a maximum of 60 seconds. All materials are at ambient temperatures with the exception of the emulsion, which is heated to a maximum of 50°C [61].

Table D-7. Mix Design RAP Gradations [61].

Sieve Size	Fine	Medium	Coarse
	Percent passing		
1.25"	100	100	100
1"	100	100	85-100
3/4"	95-100	85-96	75-92
No. 4	55-75	40-55	30-45
No. 30	15-35	4-14	1-7
No. 200	1-7	0.6-3	0.1-3

Samples are immediately compacted using the SGC. A 100 mm mold is used and samples are compacted to 30 gyrations. After compaction, samples are placed in a 140°F forced draft oven and cured to a constant mass, less 0.05% change in mass two hours, but for at least 16 hours and no more than 48 hours. Rice specific gravity is performed on two loose mix samples at the highest emulsion content using the dry-back procedure of AASHTO T 209. The Gmm at other emulsion contents is calculated [60,61].

After curing, two samples at each emulsion content are tested for Marshall stability after temperature conditioning for 2 hours at 104°F in accordance with AASHTO T 245 or sealed in a leak-proof bag for 1 hour in a 104°F water bath. Corrected stability is determined based on sample height [60,61]. It should be noted that if volume is used to get correction factors rather than height, different correction factors can result because AASHTO T 245 height correction factors appear to be based on the volume of a 4-inch (101.6 mm) diameter sample, not a 100 mm sample.

The remaining samples are tested for moisture susceptibility. The same conditioning and volumetric measurements are performed on the moisture-conditioned samples as the other samples. Samples are moisture conditioned by vacuum saturation to 55 to 70 percent saturation in accordance with AASHTO T 283. Samples are then soaked in a 77°F water bath for 23 hours followed by one-hour soak at 104°F. Optimum emulsion content is the highest emulsion content that meets minimum stability and retained stability. Next, samples are compacted at optimum emulsion content using the medium gradation and tested for raveling (ASTM D7196) and thermal cracking (AASHTO T 322) [60,61].

The raveling test is performed on 150 mm diameter by 70 mm high samples batched to the medium gradation. Samples are compacted at optimum emulsion content to 20 gyrations using the SGC. Immediately after compaction the samples are allowed to cure. Curing conditions have varied; however, curing at 10°C for 4 hours at 50% relative humidity is seen in more current specifications [61]. After curing, samples are abraded in a modified slurry seal wet track abrasion device, as shown in Figure D-15, for 15 minutes. A minimum weight loss of 2 percent is required to pass the test [60,61]. The test was developed to go with Road Science Reflex® emulsion. It is doubtful that any emulsion, other than a solventless emulsion using a chemical induced break or with the addition of lime, could pass this test.

Figure D-15. Photo. Raveling Test Apparatus.

Thermal cracking potential is evaluated by modifying AASHTO T 322 for CIR samples. Testing is performed on two rather than three samples compacted at optimum emulsion content to the design air void content. Samples are 150 mm and 155 mm tall. Samples are cured to constant dry mass as previously described and for a minimum of 48 hours and a maximum of 72 hours. After curing, the samples are cut to test size. Samples are tested at the critical design temperature and at ±10°C. Critical design temperature is the 98% reliability minimum pavement temperature at the top of the CIR layer, determined using LTPPBind. The critical cracking temperature is determined at the intersection of the thermal stress curve with the tensile strength curve [60,61]. Typical mix design requirements are shown in Table D-8 and emulsion properties are shown in Table D- 9.

Table D-8. CIR Mix Design Requirements [61].

Property	Criteria
Compaction effort, SGC	1.16° int. angle, 600 kPa stress, 30 gyrations
Density, AASHTO T 166	Report
Gradation for Design Millings, AASHTO T 27	Report
Marshall Stability, AASHTO T 245 Section 4, 40 C	1,250 lb (5.56 kN) min.
Retained stability based on cured stability[a]	70 % min.
Indirect Tensile Test, AASHTO T 322, Modified in Appendix 2	See Note in Appendix 2
Raveling Test, ASTM D7196, Cure 4 hr. ± 5 min., 50 F (10 C) and 50% humidity, Test 15 min.	2% max.

[a] Cured stability tested on compacted specimens after 140 F (60 C) curing to constant weight (mass).

Table D-9. Emulsion Requirements [61].

Test	Specification	Minimum	Maximum
Residue from distillation, %	AASHTO T 59	64.0	66.0
Oil distillate by distillation, %	AASHTO T 59		0.5
Sieve Test, %	AASHTO T 59		0.1
Penetration (To be determined), 25°C, mm	ASTM D5	-25%	+25%

Pacific Coast Conference on Asphalt Specifications (PCCAS)

Escobar [62] reported on a new mix design procedure recently adopted by the 36th Pacific Coast Conference on Asphalt Specifications (PCCAS) as guidelines for optional use for design of CIR mixtures. The procedure appears to be the Road Science mix design procedure without AASHTO T 322, the thermal cracking test.

The procedure recommends batching samples to the coarse and medium gradations shown in Table D-10. Tolerances are slightly more restrictive than the Road Science procedure [61]. Recommended mix design parameters are shown in Table D-11. The thermal cracking test (AASHTO T 322) was replaced with a requirement that the PG grade of the asphalt cement used to make the asphalt emulsion meet the bending beam requirements of AASHTO M 320 where the CIR project is constructed [62]. Emulsion requirements are similar to the Road Science procedure [60,61] and are shown in Table D-12.

Table D-10. PCCAS CIR Mix Design RAP Gradations [62].

Sieve Size	Medium	Coarse
	% Passing	
1"	100	100
3/4"	95±2	85±2
No. 4	50±2	40±2
No. 30	10±2	5±2
No. 200	0.8±0.3	0.3±0.3

Table D-11. PCCAS CIR Mix Design Requirements [62].

Property	**Criteria**
Compaction effort, SGC	1.16° int. angle, 600 kPa stress, 30 gyrations
Gradation of RAP	Report
Asphalt Content	Report
Bulk Specific Gravity	Report
Maximum Theoretical Specific Gravity	Report
Air Voids (Normally 10-16%)	Report
Marshall Stability (40°C), Cured Specimen at 60°C 16-48 hours	1,250 lb min.
Retained Marshall stability	70.0 % min.
Raveling Test, ASTM D7196, Cure 4 hr. ± 5 min., 50 F (10 C) and 50% humidity, Test 15 min.	2% max.

Table D-12. PCCAS CIR Mix Design Emulsion Requirements [62].

Test	**Specification**	**Minimum**	**Maximum**
Residue from distillation, %	AASHTO T 59	60.0	
RAP Coating Test	AASHTO T 59	Report	
Sieve Test, %	AASHTO T 59		0.1
Test on Residue Penetration & Absolute viscosity		Target values	

Use of the raveling test (ASTM D7196) will probably prohibit the use of anything other than solventless emulsions using a chemical break or emulsions with lime. The replacement of AASHTO T 322 with the requirement that the base binder for the emulsion have the correct PG grade for the location will lower the cost of the mix design considerably and allow more agencies and laboratories to provide CIR mix designs.

EXPANDED ASPHALT (FOAM)

Background

Expanded asphalt or foam is a mixture of air, water and hot asphalt. When cold water, 60 to 77°F, comes in contact with hot asphalt cement, 320 to 390°F, spontaneous foaming occurs (63). The foaming or expansion occurs as the water changes states from a liquid to a vapor, a process that is accompanied by an expansion of 1500 times its original volume. When water particles come in contact with hot asphalt, the result is a thin filmed asphalt bubble filled with water vapor. The reduced viscosity allows for dispersion or mixing at ambient temperatures.

The foam process was first realized as a stabilizing agent in the 1950's but used sparingly until the mid 1990s. Foam has traditionally been used to stabilize granular materials or RAP mixed with granular materials, a process usually called FDR in the US. Using foam in partial depth CIR is relatively new, although FDR with foam has been around for over 50 years. Only recently has there been published literature on CIR using foam. There is a wealth of information on using foam with FDR. Caltrans recently funded a very comprehensive study with the University of California at Davis of FDR using foam if one wants more information on FDR with foam.

The major difference between FDR and CIR with foam is the amount of fines (-No. 200) present. FDR literature recommends a minimum fines content greater than what is usually found in 100% RAP (CIR) mixtures. The second issue is wet or retained strengths. Retained strength testing for CIR with emulsions has usually followed a vacuum saturation procedure where much of the CIR with foam work evolved from FDR work and follows a wet soak procedure only. Therefore, direct comparisons between the two are problematic. This review is limited to the use of foam in partial depth CIR.

Asphalt temperatures need to be high for the foaming process to work. At such high temperatures (> 320°F), asphalt cement can be lethal if not handled properly. Asphalt manufacturers work with hot asphalts on a daily basis and are aware of the dangers and proper safety precautions. Recycling contractors and agency personnel need to be made aware of the safety concerns and proper handling procedures (64).

Construction Methods

CIR using foam generally requires similar equipment and construction specifications as CIR with asphalt emulsions. In the recycling train the asphalt emulsion tanker is replaced with a hot asphalt tanker that injects the asphalt into the recycler or pugmil through a special spraybar. Due to the high heat of the asphalt cement, > 320°F, special equipment and safety precautions are often required (64).

Laydown and compaction procedures for CIR with foam are similar to conventional CIR. Weather restrictions are similar and foam operations should not be undertaken when the material drops below 50°F [64] or 60°F [65]. Final curing requirements are the same as conventional CIR with overlays placed after the moisture content is 0.5% above the residual moisture content or less than 2.0%. Foam usually takes less than 48 hours to cure whereas emulsions can take considerably longer [29,64,65].

Performance Studies

Using foam in partial depth CIR is relatively new; therefore, very little published literature on CIR using foam was available until recently. There is a wealth of information on using foam with FDR. Iowa and Ohio DOTs both reported performing CIR using foam and expressed initial satisfaction with the process. However, no published reports were found in the literature. Ontario [20] reported completing 43 CIR projects on over 500 lane km of pavements. Forty of these projects used conventional asphalt emulsions and three used expanded asphalt or foam. A comparison of foam and conventional emulsions were reported on one project.

Lane [29] reported on a comparison of CIR using convention asphalt emulsion and foam on HWY 7 in Ontario, Canada. A portion of the project was recycled 110 mm deep using 1.2% HF 150MP emulsion with 3.5% mix water. The remainder was recycled 110 mm deep using 1% PGAC 58-28 with 4.0% mix water. The mixtures were placed to a minimum of 96% of the laboratory compacted density. Contract documents allowed the foam section to be overlaid after 48 hours. The emulsion section required a minimum moisture content of 2.0%. Several weeks were required before the emulsion section could be overlaid [29].

Field samples of both materials were obtained and tested in the laboratory for indirect tensile strength. Laboratory testing indicated that strength was related to bulk density and that foam samples compacted more readily in the laboratory than emulsion samples. Cores obtained from the field eight months after construction indicated no difference in density, so the researchers concluded that tensile strengths were similar. Resilient modulus testing was performed on cores and results ranged from 1,141 to 1,629 MPA for emulsion cores and 1,194 to 1,704 MPa for foam cores. The difference was reported as not statistically different. FWD testing, rut depth testing and roughness measurements were also performed with no significant differences reported. A field review performed one year after construction indicated both sections were performing well with the emulsion section being rated slightly better for ride quality than the foam section [29].

Chan et al. [66] reported a follow up study to the HWY 7 project report by Lane [29] after 5 years of traffic. Chan [66] reported that initial roughness was higher for the foamed section compared to the emulsion section but after five years, there was no statistical difference in roughness. Rutting data indicated that the foam section had statistically lower rut depths than the emulsion. Average rut depths were 2.6 mm for the foam and 2.9 mm for the emulsion, a difference of 0.3 mm which can hardly be considered a practical difference. Chan concluded similar rutting performance after 3 years performance. FWD testing and laboratory testing for tensile strength and resilient modulus had the same trends. Initially the emulsions had lower strengths but when fully cured, there was no significant difference in mix properties. The authors concluded there was no difference in performance between the two sections.

The limited literature indicated comparable performance between CIR using foam and conventional asphalt emulsions. Wirtgen [64] listed the advantages and disadvantages of recycling using emulsions and foam. They are shown in Table D-13.

Table D-13. Comparison of Emulsions vs. Foam [64].

Asphalt Emulsions	
Advantages	**Disadvantages**
Flexibility: A visco-elastic material with improved flexibility and resistance to rutting.	Cost**.** Emulsions are not manufactured on site, requiring haul costs that are inflated by hauling the water component.
Ease of Application: A bulk tanker is coupled to the recycler and emulsion is injected through a spraybar.	When the moisture content in the material is high, saturation can occur when the emulsion is added.
Acceptance: Emulsions are well known and standard test methods exist.	Curing can take a long time and strength gain is dictated by moisture loss.
	The required formulation may not always be available.
Foamed Asphalt	
Advantages	**Disadvantages**
Flexibility: A visco-elastic material with improved flexibility and resistance to rutting.	Foamed asphalt demands that the asphalt is hot, usually above 320^{o}F. This often requires special heating facilities and additional safety precautions.
Ease of Application: A bulk tanker is coupled to the recycler and hot asphalt is injected through a special spraybar.	Material type and condition. Saturated material and material deficient in material passing No. 200 sieve can not be treated without pre-treatment or the addition of new material.
Foamed asphalt uses standard grade asphalt with no additional manufacturing costs.	
Rate of Strength Gain: Material can be trafficked immediately after placing and compaction	

Mix Properties

Foam Generation

The foam process was first realized as a stabilizing agent in the 1950's but used sparingly until the mid 1990's. Foam asphalt is characterized by two primary properties, expansion ratio and half life. Expansion ratio is the ratio of the maximum volume relative to its original volume and is a measure of viscosity or how well it will mix. Half life is the time in seconds it takes for the foam to collapse to half its maximum expansion and is a measure of the stability of the foam. Minimum acceptable values are 10 times for expansion ratio and 8 seconds for half-life. Foam properties are influenced by water addition, asphalt hardness, asphalt source, temperature, pressure and additives [64]. The relationship between expansion, half-life and water addition is shown in Figure D-16.

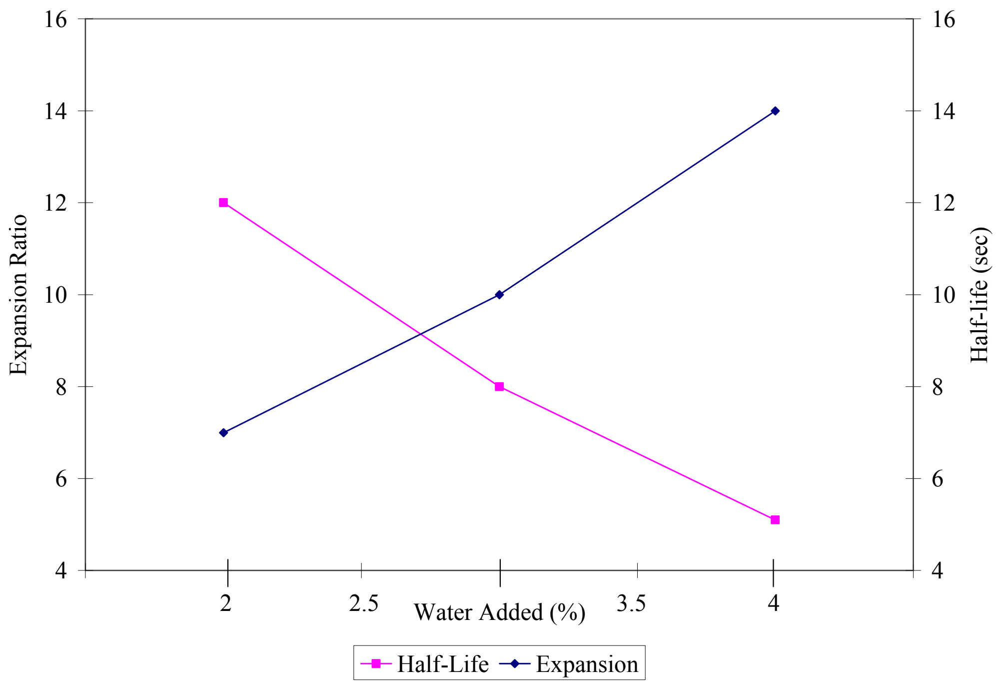

Figure D-16. Scatter Plot. Relationship Between Foaming Properties [64].

Asphalt Cements

Conventional PG graded asphalt cements are used for foam generation, no special additives or properties are required. However, not all asphalts foam equally as well. Softer asphalts, 80 to 150 pen, are usually preferred [64].

RAP Gradation

Foam has traditionally been used to stabilize granular materials or RAP mixed with granular materials, a process usually called FDR in the US. However, 100% RAP mixtures have been successfully stabilized using foam. According to Wirtgen [64], a certain amount of fines are generally considered necessary to promote good mixing with foam asphalt. Too few fines, less than 5% passing No. 200 sieve, can result in poor dispersion of the foamed asphalt resulting in stringers of asphalt in the mix. These stringers act as a lubricant in the mix and lead to reduced strength and stability. Most 100% RAP mixtures have little to no minus No. 200 material. Overcome this deficiency in fines generally requires addition of filler. Another approach is to add cement or lime. Cement should be limited to less than 1.5% to avoid negative effects on flexibility of the stabilized layer. Figure D-17 shows the recommended gradation limits for using foam.

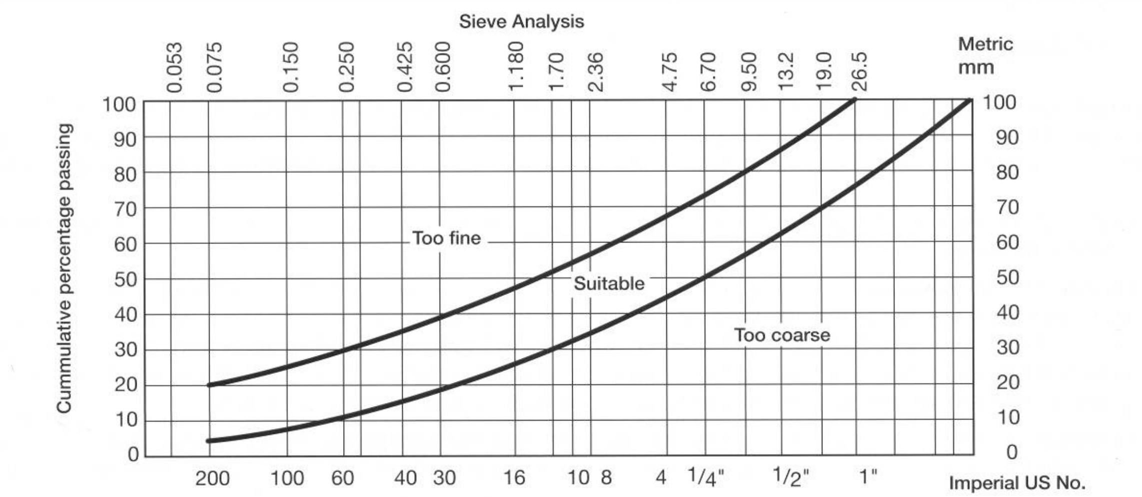

Figure D-17 Scatter Plot. Suitability of Material for Foamed Bitumen Treatment [64].

Moisture Resistance

Moisture resistance is often listed as an advantage of CIR with foam compared to emulsion. However, test conditions are not always similar making direct comparisons problematic. There is a need for a well planned study comparing conditioned strengths. Fu et al. [67] evaluated four FDR-foamed projects in California that showed early distress. The distress was found to be related to water damage. Fu concluded that soaked tensile strengths were required to adequately predict field performance of FDR-foam mixtures. In a second study [68], Fu reported that portland cement was very effective in improving strength, stiffness and permanent deformation resistance of foamed asphalt mixes, especially in the early stages when the foamed asphalt has not fully cured.

Mix Design Procedure

Wirtgen [64] has published a mix design procedure for foam mixtures. The procedure was developed for FDR mixtures but may be applicable to CIR mixtures using 100% RAP as well. Foam mix designs require the use of a laboratory foam generator, as shown in Figure D-18. The procedure is briefly described below.

Wirtgen [64] recommends that all foam projects contain a small amount of active filler. For mixtures with a PI < 10 (100% RAP mixtures) 1% portland cement is recommended. Optimum fluid content and maximum dry density for treated material are determined by assuming they are the same as the optimum moisture content and maximum dry density determined on representative samples of untreated materials using AASHTO T-180, modified Proctor compaction.

Figure D-18. Photo. Wirtgen WLB10 Laboratory Foam Generator [69].

Water content and asphalt temperature for optimum expansion ratio and half-life is determined using a foam generator. Water (2%) is injected into heated asphalt (320^{o}F) using the foam generator and the maximum expansion and time in seconds for the foam to collapse to half its maximum volume is recorded. The expansion ratio and half-life are recorded. The procedure is repeated for two additional water contents (usually 3 and 4%) and then the whole process repeated at two additional asphalt temperatures (usually 340^{o}F and 360^{o}F). The optimum water content is the average of the two water contents that meet the minimum expansion ratio of 8 and minimum half-life of 6 seconds. The temperature and water content that produces the best form is used in the mix design [64].

For the mix design, samples are mixed at optimum water content with different asphalt contents; an allowance is made for water in the foam and in the material. Materials are compacted using either 75-blow Marshall compaction or Superpave gyratory compaction. After compaction, samples are extruded from the molds and cured for 72 hours at 104^{o}F in a forced draft oven. The bulk specific gravity of each sample is determined and the dry and soaked indirect tensile strength and tensile strength ratio is determined. Dry samples are tested at 77^{o}F. Wet samples are tested after a 24-hour soak in a 77^{o}F water bath. The results of the wet and dry indirect tensile strength tests are plotted versus asphalt content. The asphalt content that best meets the desired properties is the optimum asphalt content [64]. Ohio DOT CIR mix design specification requirements using foam are shown below in Table D-14.

Table D-14. Ohio DOT Foam CIR Mix Design Requirements [65].

Property	Minimum Requirement
Dry Tensile Strength (kPa)	300
Wet Tensile Strength (kPa)	150
Tensile Strength Ratio (TSR %)	50

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