AN ABSTRACT OF THE THESIS OF

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Title: Evaluation of Cold In-Place Recycling of Asphalt Concrete Pavements in Oregon

Redacted for privacy

Abstract Approved: ______________________________
R. Gary Hicks

Significant use of cold in-place recycling (CIR) has occurred in Oregon since 1984 as an alternative to other rehabilitation techniques or to the reconstruction of distressed asphalt concrete (AC) pavements. Due to the initial success of the projects cold recycled during 1984-85, the Oregon Department of Transportation (ODOT) and Oregon State University (OSU) undertook, in 1986, an intensive study to investigate cold in-place recycling. The objectives of this study were to:

1. develop an improved mix design procedure for cold in-place recycled pavements,
2. evaluate the structural contribution of the cold recycled pavements, and
3. develop improved construction guidelines and specifications for these pavements.

Presented in this thesis are two papers summarizing the efforts and advancements made to achieve the first two objectives. Also included is a paper that evaluates the repeatability of two test methods which were, in part, used to evaluate the structural contribution of cold recycled pavements.

EVALUATION OF COLD IN-PLACE RECYCLING OF ASPHALT CONCRETE PAVEMENTS IN OREGON

by

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Head of Department of Civil Engineering

Redacted for privacy

Dean of Graduate School

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TABLE OF CONTENTS

1.0 INTRODUCTION 1

1.1 Background 1

1.2 Purpose 2

2.0 USE OF COLD IN-PLACE RECYCLED ASPHALT MIXES FOR ROAD SURFACES 4

ABSTRACT 4

2.1 Introduction 7

2.1.1 Background 7

2.1.2 Purpose 9

2.2 Cold Recycling in Oregon 10

2.2.1 Mix Design Process 10

2.2.2 Structural Design 20

2.3 Construction Process 25

2.3.1 Roto-Mill-Grader Laydown 25

2.3.2 Recycling Train 26

2.3.3 Single Train Unit 27

2.3.4 Evaluation of Construction Process 27

2.4 Field Performance 29

2.4.1 Performance Condition 29

2.4.2 Ride 31

2.4.3 Mix Properties 31

2.5 Conclusions 37

2.6 References 39

3.0 MIX DESIGN PRACTICES FOR COLD IN-PLACE RECYCLED PAVEMENTS 40

TABLE OF CONTENTS (CONTINUED)

ABSTRACT 40

3.1 Introduction 42

3.2 History of Cold Recycling in Oregon 45

3.2.1 Project Information 45

3.2.2 Construction Process 45

3.3 Evolution of Mix Design Process 51

3.3.1 Initial Design Process: 1984-86 51

3.3.2 Improved Design Process: 1987-88 56

3.4 Expected Ranges in Strength Properties 71

3.4.1 Modulus and Fatigue 71

3.4.2 Stability and Flow 74

3.5 Conclusions and Recommendations 77

3.5.1 Conclusions 77

3.5.2 Recommendations 78

3.6 References 79

4.0 REPEATABILITY OF TESTING PROCEDURES FOR RESILIENT MODULUS AND FATIGUE 80

ABSTRACT 80

4.1 Introduction 82

4.1.1 Problem Statement 82

4.1.2 Purpose 83

4.2 Laboratory Study 85

4.2.1 Description of Laboratory Study 85

4.2.2 Test Equipment and Procedures 91

4.2.3 Test Results 96

TABLE ON CONTENTS (CONTINUED)

4.2.4 Analysis of Results 104
4.2.5 Discussion of Results 106
4.3 Conclusions and Recommendations 111
4.3.1 Conclusions 111
4.3.2 Recommendations 111
4.4 References 113
5.0 CONCLUSIONS 114
6.0 BIBLIOGRAPHY 116
APPENDICES
A DETAILED MIX DESIGN PROCEDURE FOR CIR 119
B FATIGUE TEST PROCEDURE 125

LIST OF FIGURES

Figure   Page

2.1 Softness of Asphalt in Old Pavement 16

2.2 Range of Milling (RAP) Gradations from 41 cm (16 in.) and 3.8 m (150 in.) Mill 18

2.3 Adjustment for Residual Asphalt Content 19

2.4 Suggested Mix Design Process - Future Projects 21

2.5 Resilient Modulus and Fatigue Test Results (1 ksi = 6.9 MPa) 34

2.6 Marshal Stability and Flow Test Results (1 lb = 4.45 N and 1 mil = 0.025 mm) 35

3.1 Softness of Asphalt Cement in Old Pavement 60

3.2 Range of Milling (RAP) Gradations from the 41 cm (16 in.) Mill and the 3.8 m (150 in.) Mill on the Recycling Train . . 62

3.3 Adjustment for Residual Asphalt Cement Content 64

3.4 Proposed Mix Design Process - Future Projects 67

3.5 Resilient Modulus and Fatigue Test Results for Selected 1986 Projects 73

3.6 Marshall Stability Test Results for Selected 1986 Projects . 76

4.1 Flowchart Showing Scope of This Study 84

4.2 Original and Modified Failure Criteria for Fatigue 88

4.3 Failure Criterion as Determined by Measurement of Permanent Deformation 89

4.4 Typical Strip Chart for Diametral Resilient Modulus of an Asphalt Concrete Specimen 95

LIST OF APPENDIX FIGURES

Figure		Page
A.1	Emulsion Content Adjustments for Gradation, Asphalt Content, and Asphalt Softness	123
A.2	Determination of the Adjusted Gradation (for the 150-in Mill) from the Gradation on the 16-in Mill	124
B.1	Loading of a Cylindrical Asphalt Concrete Specimen in the Repeated Load Indirect Tensile Test	127
B.2	Asphalt Concrete Specimen with Foil Tape Having Loops on Both Sides of the Specimen	130
B.3	Fatigue Specimen Set Up	131

LIST OF TABLES

Table		Page
2.1	Projects Constructed in Oregon (1984-1986)	11
2.2	Cost Comparison with 5 cm Overlay	13
2.3	Proposed Adjustments from Base Design of 1.5%	15
2.4	Suggested Sample Preparation Procedure for CIR	22
2.5	Suggested Mix Design Criteria	23
2.6	Comparison of Fatigue Lives for CIR and Conventional Hot Mixes	23
2.7	Pavement Condition of Selected CIR Projects (Spring 1988)	30
2.8	Before and After Ride Data for Selected CIR Projects	32
2.9	Criteria to Rate the Smoothness of a Pavement Using the Mays Ride Meter	32
2.10	Summary of Mix Property Test Results	33
3.1	Projects Constructed in Oregon (1984-88)	46
3.2	Mix Design Procedure Used for the 1985 CIR Projects	52
3.3	Oregon's 1986 Mix Design Guidelines for CIR (Revised Mid-Summer 1986)	54
3.4	Mix Design Criteria for 1986 Projects; a) Initial 1986 Criteria, b) Revised Mix Design Criteria (mid-1986)	55
3.5	Specifications for the CMS-2S and HFE-150 Emulsions	57
3.6	Proposed Adjustments from Base Design; a) Adjustments, b) Example	59
3.7	Suggested Sample Preparation Procedures for CIR Mixtures	68
3.8	Suggested Mix Design Criteria (1987-88)	69
3.9	Summary of Modulus and Fatigue Test Results (Selected 1986 Projects)	72
3.10	Summary of Marshall Stability Test Results (Selected 1986 Projects)	75

LIST OF TABLES (CONTINUED)

Table | Page

4.1 Job-Mix Formula for Laboratory-Fabricated Test Specimens 86

4.2 Original Test Schedule for the Laboratory Study 86

4.3 Revised Test Schedule for the Laboratory Study 90

4.4 Summary of OSHD Modulus Test Results for the Warm Temperature (25°C) Samples 97

4.5 Summary of OSHD Modulus Test Results on the Cold Temperature CA(P)-1 Samples Which Were Tested at 25°C 98

4.6 Summary of OSU Modulus and Fatigue Test Results for the Warm Temperature (25°C) Samples 100

4.7 Summary of OSU Modulus and Fatigue Test Results for the Cold Temperature CA(P)-1 Samples Which Were Tested at 25°C . . . 101

4.8 Summary of OSU Modulus and Fatigue Test Results for the Cold Temperature AC20 Samples Tested at 25°C 102

4.9 Summary of OSU Modulus and Fatigue Test Results for Both Mix Types at 0°C 103

4.10 Summary of Statistical Analysis (Student's t-test) of Modulus Test Results at 25°C Between Laboratories 105

4.11 Statistical Summary of Resilient Modulus Test Results Within Each Laboratory 107

4.12 Statistical Summary of Fatigue Test Results 108

PREFACE

This thesis is a compilation of three papers written for separate publication and inclusion herein. The first paper was published in the proceedings for the 5th Conference on Asphalt Pavements for Southern Africa presented in June, 1989, the second in the proceedings for the American Society for Testing and Materials (ASTM) Symposium on Asphalt Emulsions presented in December, 1988, and the third will be submitted for presentation and publication at the annual meeting of the Transportation Research Board in January, 1990. Each paper is complete by itself. However, some duplication of material appears in the first two papers (Chapters 2 and 3). Citations are referenced at the end of each paper and a comprehensive bibliography is included at the end of this report.

The co-authors listed in each paper contributed their expertise with respect to the scope and details of the research efforts presented herein. More specifically, in the first two papers (Chapters 2 and 3), Dr. R. Gary Hicks formulated and oversaw the scope of the research efforts while Mr. Dale Allen and Dr. Hicks, together, provided necessary guidance, details, and editorial comments throughout the development of the two papers. In the third paper (Chapter 4) Mr. Lewis Scholl formulated the scope of the study as well as provided his expertise as an advisor throughout the study and an editor of the contents of the paper. Similarly, Dr. Hicks provided his expertise as an advisor and editor in addition to overseeing the study.

Todd Scholz

June 1989

EVALUATION OF COLD IN-PLACE RECYCLING OF ASPHALT CONCRETE PAVEMENTS IN OREGON

1.0 INTRODUCTION

1.1 Background

Across the nation, state departments of transportation (DOTs) and other agencies are placing significant emphasis on the rehabilitation and preservation of existing roadway pavements. The rationale behind this strategy (as opposed to allowing pavements to deteriorate to the point that costly reconstruction is required) is to protect the investment already made in the roadway systems. That is, by improving a moderately distressed pavement, through various rehabilitation techniques, significant savings can be realized in the cost of reconstruction of a severely distressed pavement. Rehabilitation/ preservation, however, can only delay the eventual need for more expensive modernization or reconstruction of pavements.

Rehabilitation techniques which have been widely accepted and commonly used in the past have ranged from pothole patching and crack sealing for minor distress to chip sealing and overlaying for moderate distress to inlaying (removal and replacement) for severely distressed pavements. One approach, which has evolved since 1980 and is rapidly gaining wide acceptance, is the cold in-place recycling of the existing materials in the pavement. In this technique, existing materials are removed, processed, and replaced; thus conserving costly construction materials.

Pavement recycling can incorporate all materials (e.g., surface, base, and subbase) down to, and sometimes including, subgrade material depending on the intended purpose of the recycled mixture. For example, if the intended use of the recycled mix is that of a base course, all materials down to and including the subgrade may be used. If, on the other hand, the intended purpose of the recycled mix is that of a wearing course, the existing surface course may be the only material used in the recycled mixture.

Oregon has, since 1984, utilized pavement recycling techniques to rehabilitate distressed asphalt concrete (AC) pavements. The purpose of these recycled mixtures has been to provide an improved wearing course. Thus, Oregon has exclusively recycled only the surface of the pavement (i.e., base, subbase, and subgrade materials have not been incorporated). In addition, Oregon has exclusively utilized cold in-place recycling (CIR) techniques in all recycling efforts. This has provided the benefit of substantial savings in energy costs and a reduced impact on the environment.

1.2 Purpose

Presented in this thesis are three papers written for separate publication and inclusion herein. The first two papers present detailed information regarding the cold in-place recycling efforts in Oregon since 1984. Together, these two papers describe and evaluate the mix design procedure, the construction process, and the field performance of CIR pavements constructed in Oregon between 1984 and 1988.

The third paper presents an evaluation of the repeatability (precision) of two test methods used, in part, to determine the performance of cold recycled mixes. The results of this third paper provide a general idea of the reliability that can be placed on the two test methods.

2.0 USE OF COLD IN-PLACE RECYCLED ASPHALT MIXES FOR ROAD SURFACES

by

Todd Scholz[1], R.G. Hicks[2], and Dale Allen[3]

ABSTRACT

Currently, the United States is experiencing a national trend toward rehabilitation of distressed asphalt concrete (AC) pavements as opposed to new construction. This, compounded by the inflationary trend in the cost of construction materials, is requiring highway agencies to pursue alternative approaches to the preservation of existing highway systems. One approach, which is proving to be one of the most promising and cost-effective alternatives, is cold in-place recycling (CIR).

Advantages attributed to the use of cold in-place recycling include significant cost savings realized through savings in energy, conservation of costly construction materials, and a reduction in the impact on the environment, as well as the ability to limit mitigation to the distressed lane. Although significant savings are realized through the use of CIR, there remains an absence of proven and simple mix and thickness design procedures—standard design procedures currently do not exist. Furthermore, due to the lack of long-term per-

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[1]Research Assistant, Department of Civil Engineering, Oregon State University, Corvallis, OR 97331

[2]Professor of Civil Engineering, Oregon State University, Corvallis, OR 97331

[3]Region Engineer, Oregon State Highway Division, Bend, OR 97708

formance data and adequately documented field engineering studies, many agencies remain skeptical of the use of CIR as a viable alternative to pavement restoration and rehabilitation.

However, several major research efforts are currently underway to summarize current and/or develop new technical data associated with cold in-place recycling. These efforts are expected to address the need that exists for realistic and defensible procedures for the design and use of CIR asphalt pavements. One of the major efforts researching cold in-place recycling was a joint study between the Oregon Department of Transportation (ODOT) and Oregon State University (OSU). This study was initiated in April 1986 and completed in June 1988 with the overall objective to develop a simple and reliable mix design procedure for cold in-place recycled pavements in the state of Oregon.

This paper summarizes the efforts undertaken by ODOT and OSU to accomplish this objective. Specifically, this paper presents:

1) A review of the projects that were studied. These encompass nearly 322 km (200 miles) of cold in-place recycled AC pavements constructed during the period of 1984 to 1986.
2) The process used to construct the pavements. CIR was accomplished employing two different methods: a "recycling train" and a "single unit" machine. Both methods utilized standard paving equipment for laydown.
3) The performance of the cold in-place recycled pavements. Field and laboratory data were collected and evaluated in

order to assess the performance of the pavements as well as to investigate the effects of emulsion content, curing time, and compactive effort. Field performance evaluations indicate the pavements are performing very well. Laboratory test results indicate the mix properties differ only slightly from those of conventional mixes.

4) Recommendations for a simple and reliable mix design procedure for cold in-place recycled asphalt materials as well as improved guidelines and specifications for the construction of CIR pavements.

2.1 Introduction

2.1.1 Background

The national trend away from new construction to preservation of the highway system is requiring highway agencies to seek alternative approaches to rehabilitating distressed pavements. One of the most promising and cost effective approaches is cold in-place recycling (CIR). Recycling is defined as the re-use, after processing, of a material that has already served its intended purpose.

Although cold in-place recycling of asphalt pavements has been used in the United States in some form since the 1920s, the methods discussed in this report have evolved since 1980. During this period, spurred by the development of milling and reclaiming equipment, CIR has evolved into one of the fastest growing pavement rehabilitation procedures. Many agencies, however, remain skeptical of the use of CIR because of the lack of long-term performance data and adequately documented field engineering studies.

Further compounding the problem is that the term CIR is frequently misunderstood. The reason for this is because of the different processes used with substantially different design concepts and end results. For the purposes of this paper, these different processes include the following:

1) Class I. This recycling treatment is performed on a uniform pavement designed and built to specifications. It is expected that a rational CIR mix design can be

prepared and produced. The treatment could handle medium to high traffic volumes, either as a base or wearing course. Treatment width is normally 3.6 m (12 ft).

2) Class II. This recycling treatment is performed on a pavement with significant maintenance patches over a uniform pavement or a pavement with minimal design used in the original construction. The finished mixture may be used as a base or wearing course. Treatment width is normally 3.6 m (12 ft).

3) Class III. This treatment is used on low volume highways where considerable variation in pavement structure exists, and it may incorporate base aggregate. The design of the mix is limited. Normally, the treatment is used as a base. Treatment width varies from 1.2–3.6 m (4–12 ft).

In 1986–87, three major research efforts were initiated to summarize and publish currently available technical data. These projects include:

1) An NCHRP synthesis on cold recycling design methods and practices.

2) An ARRA research study concerned with the "Evaluation of Design and Performance Criteria for CIR Asphalt Pavement."

3) A study, funded by Oregon Department of Transportation, concerned with the "Development of Improved Mix Design Procedures for CIR Asphalt Pavement."

These research efforts were initiated with the expectation of addressing the need that exists for defensible and realistic design procedures.

2.1.2 Purpose

This paper presents a description of the projects, including mix and thickness design procedures, involved in the ODOT/OSU study. Additionally, methods to construct these projects and the performance of the projects are given.

2.2 Cold Recycling In Oregon

Oregon first experimented with partial depth CIR work in 1984. This work, totaling about 22.5 km (14 miles), was done with state forces and rented equipment. Due to initial success, 138 km (86 miles) were cold recycled in 1985. Table 2.1 summarizes construction information for these projects. The ultimate objectives of these initial projects were to determine costs on an actual major contract, the quality of the product achieved, and to advance the state of the art of CIR in the state of Oregon.

Encouraged by substantial cost savings (Table 2.2), high production rates, and performance, 249 km (155 miles) of highway were recycled in 1986 (Table 2.1), 121 km (75 miles) were recycled in 1987, and another 80 km (50 miles) in 1988. Data are being collected for the 1987 and 1988 projects.

2.2.1 Mix Design Process

In 1984, formal mix designs were not available. Emulsion and water contents were established in the field using trial-and-error procedures by experienced paving personnel. In general, the emulsion contents were about 1.5% by dry weight of RAP (recycled asphalt pavement) while the water contents varied from 2 to 4%.

For the 1985 projects, Oregon DOT attempted to use a formal mix design procedure for CIR which was basically a modification of the existing hot-mix procedure used by Oregon DOT. This procedure was also used at the onset of the 1986 construction season. However, in most cases the procedure resulted in emulsion contents that were too

Table 2.1. Projects Constructed in Oregon (1984-86).

Year	Highway	Project Name	Traffic Volume (ADT)	Length[1] (km)	Depth of Cut[2] (cm)	Emulsion Type (Content)	Method of Construction	Chip Seal	Performance Spring 1988
1984	OR 372	Sand Shed-Mt. Bachelor (Intermittent)	820	7.7	4	CMS-2S (1-2%)	State forces, Class III treatment, grader laid	Surface left open winter of 1984, chip sealed in 1985	Fair to Good
	Misc.	Bend area	Up to 2,000	14.5	4	CMS-2S (1-2%)	State forces,Class III treatment, grader laid	About 50% chip sealed	Fair
1985	US 26	Sisters-Redmond	1,450-8,300	30.2	4	CMS-2S (1-2%)	Class II treatment	Chip seal placed on about 75% of work	Good
	US 395	Harney Co. Line-Hogback Summit	220	49.4	4-5	CMS-2S[3] (1-2%)	Class I treatment	Entire section chip sealed	Good
	US 140	Drews Gap-Lakeview	1,000	16.6	4-5	CMS-2S (1-2%)	Class I treatment	Entire section chip sealed	Fair to Good
	Misc.	Bend area	up to 23,000	19.3	4-5	CMS-2S (1-2%)	Class I treatment	80% chip sealed	Good
1986	US 26	Warm Springs	2,850	27.8	5-10	CMS-2S (1%)	Class I treatment	Surface left open winter of 1986, chip sealed in 1987	Fair to Poor
	OR 41	Powell Butte-Prineville	3,600	15.8	5	CMS-2S HFE-150	Class I treatment	Entire section chip sealed	Fair to Good
	OR 270	Lake of the Woods	1,750	10.2	6-10	CMS-2S (1.4%)	Class I treatment	Entire section chip sealed	Very Good
	US 20	Bend-Powell Butte	4,800	5.1	4-5	CMS-2S (1.5%)	Class I treatment	Entire section sand sealed	Fair
	OR 371	MP 18.0-Powell Butte	2,200	29.0	4-5	CMS-2S (1.1-1.3%)	Class II treatment	Entire section chip sealed	Good

[1] 1 mile = 1.609 km
[2] 1 inch = 2.54 cm
[3] HFE-150 and HFE-150S were also used, but only for test.

Table 2.1. Projects Constructed in Oregon (1984-86) (continued).

Year	Highway	Project Name	Traffic Volume (ADT)	Length[1] (km)	Depth of Cut[2] (cm)	Emulsion Type (Content)	Method of Construction	Chip Seal	Performance Spring 1988
1986	US 26	Ochoco Dam-MP 35.0	1,100	17.0	4-5	CM-2S (1.1-1.6%)	Class II treatment	Entire section chip sealed	Good
	US 26	MP 73.4-MP 81.6	600	13.2	4-5	CMS-2S (1.8-2.6%)	Class II treatment	8 cm overlay	Good
	US 26	MP 89.6-Jct. OR 19	600	14.0	4-5	CMS-2S (1.4-1.5%)	Class I treatment	Entire section chip sealed	Very Good
	US 20	MP 75.0-MP 84.0	1,000	14.5	4-5	CMS-2S (1.5-1.6%)	Class I treatment	2 cm oil mat	Fair
	OR 423	US 97-OR 39	800	11.3	4-5	CMS-2S (1.5%)	Class I treatment	Entire section chip sealed	N/A†
	OR 140	Dairy-Ritter Rd.	2,000	9.6	4-5	CMS-2S (1.2-1.9%)	Class II treatment	Entire section chip sealed	Fair
	OR 140	Sprague River Rd.-Bly	2,700	28.6	4-5	CMS-2S (1.5%)	Class II treatment	Entire section chip sealed	Good
	US 97	MP 235.3-Spring Creek	3,400	9.6	4-5	CMS-2S (0.9%)	Class I treatment	Entire section chip sealed	Fair

[1] 1 mile = 1.609 km
[2] 1 inch = 2.54 cm
†Not Available

Table 2.2. Cost Comparison with 5 cm Overlay 1985 Prices (After Ref. 1).

	Cold Recycle[1]	Overlay[2]	Difference
Cost/square meter	$1.44[3]	$4.78	330%
Cost/kilometer	$10,560	$58,420	550%
Metric tons processed/day	3,600	3,200	12%
Kilometers/day	6.4	4	160%
Cost/24 mile project	$255,000	$1,410,000	$1.15 million

[1]Based on 5 cm (2 in.) depth, 7.3 m (24 ft) wide of 12.2 m (40 ft) roadway pavement
[2]Based on 5 cm (2 in.) overlay, 12.2 m (40 ft) roadway
[3]Cost without seal; $2.21 with chip seal, $1.67 with sand seal

high. Subsequent modification of the procedure corrected these problems. The modified procedure has been used on all projects constructed after the mid-1986 construction season.

The proposed mix design described in this paper consists of estimating the design emulsion content and preparing and testing samples at the design oil content and at design ±0.4%. These procedures are described in detail in the following section.

Estimating Design Emulsion Content. Estimation of the design emulsion content begins with establishing a base design emulsion content and making adjustments based on the results of the laboratory findings. Oregon has found, through experience with the CMS-2S emulsion, that a base design emulsion content of 1.5% by dry weight of RAP is a good starting point. Adjustments are then made to this base content according to softness of extracted asphalt, gradation of the millings (41 cm (16-in.) mill), and the percent of recovered asphalt. Table 2.3 shows the calculations to be made with the adjustments. The final estimated design emulsion content can be as low as 0.4% and as high as 2.6%. The adjustments are discussed in detail below:

1) Softness of Asphalt. The penetration and absolute viscosity laboratory test results are used to determine the softness of the extracted asphalt. Figure 2.1 indicates the ranges in these values that have been found in CIR completed to date. By plotting the values obtained from the laboratory on this figure, an adjustment of up to ±0.5% can be selected. Thus, for a hard asphalt an

Table 2.3. Proposed Adjustments from Base Design of 1.5%.

Base Design	1.5%
Adjustment for Softness	±0.5%
Adjustment for Gradation	±0.3%
Adjustment for % Asphalt	±0.3%
Final Estimated Design	____%
Lowest Design	0.4%
Highest Design	2.6%

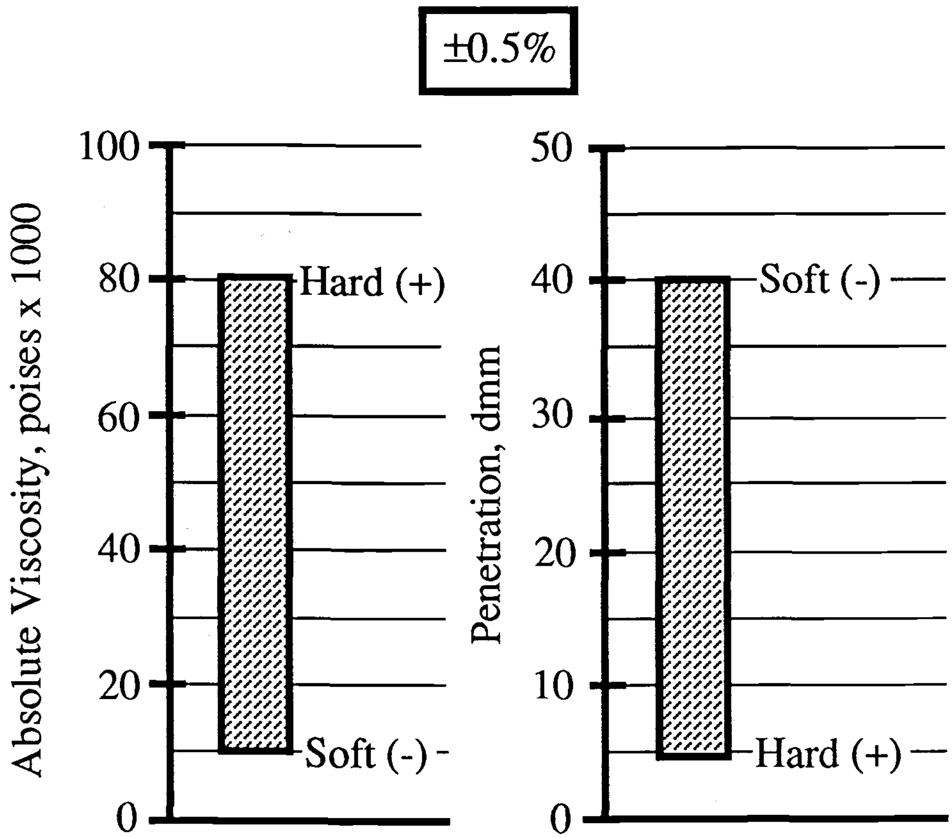


Figure 2.1. Softness of Asphalt on Old Pavement.

emulsion content adjustment of up to 0.5% would be added to the base design emulsion content. Conversely, an adjustment of up to 0.5% would be subtracted from the base content for a soft asphalt.

2) Gradation Adjustment. By plotting the RAP gradations from CIR completed to date, a range of values was obtained for the percent passing the 12.7 mm (1/2 in.), 6.4 mm (1/4 in.), and 2.0 mm (#10) screens. Figure 2.2 indicates the range of values when the sampling is performed with a 41 cm (16 in.) mill and the expected RAP gradation when using a 3.8 m (150 in.) mill. By using this graph, a maximum adjustment of ±0.3% can be made to the base design emulsion content. RAP with a coarse gradation would result in adding an adjustment of up to 0.3% to the base design emulsion content while up to 0.3% would be subtracted for RAP with a fine gradation. Findings to date indicate that if a RAP gradation is fine on the 12.7 mm (1/2 in.) screen, it will also indicate a fine gradation on the 6.4 mm (1/4 in.) and 2.0 mm (#10) screens. The same holds true for a coarse or average gradation (2,3).

3) Asphalt Adjustment. The percent of asphalt recovered from the RAP was plotted giving the expected range of asphalt content. Figure 2.3 shows this range as well as the adjustment range of ±0.3. RAP with a high residual asphalt content would result in subtracting up to 0.3%

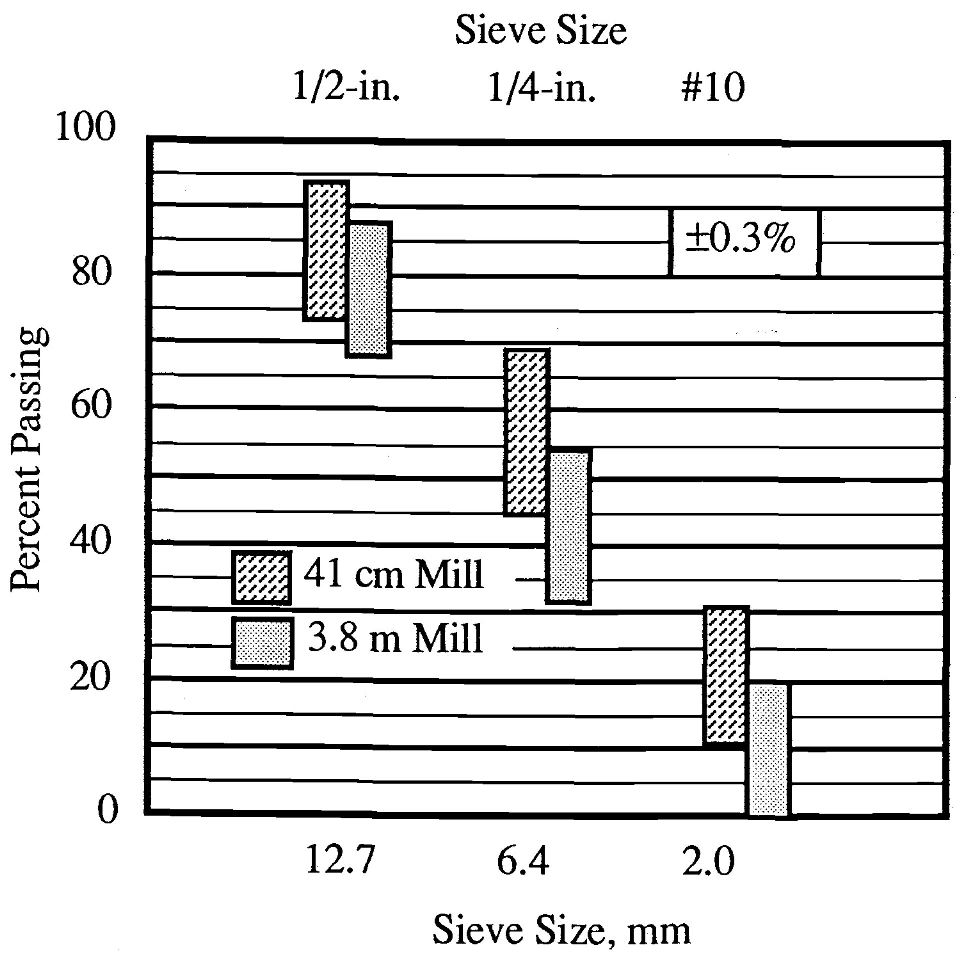


Figure 2.2. Range of Milling (RAP) Gradations from 41 cm (16 in.) Mill and 3.8 m (150 in.) Mill.

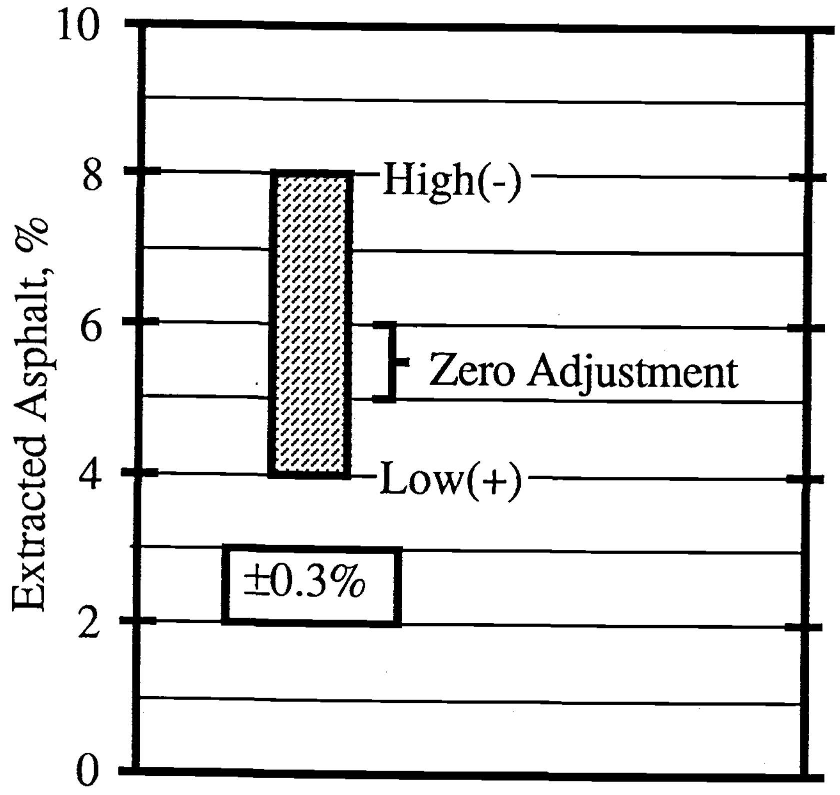


Figure 2.3. Adjustment for Residual Asphalt Content.

from the base design emulsion content while up to 0.3% would be added for RAP with a low residual asphalt content. No adjustment is necessary if the residual asphalt content falls between 5 and 6%.

The final estimated design emulsion content is determined as shown in Table 2.3.

Final Design. Although not yet implemented, it is proposed that the final design emulsion content be determined from tests on samples prepared at the final estimated design emulsion content (determined above) and at the final estimated content ±0.4%. Figure 2.4 summarizes the proposed steps to select a final design emulsion content where the CIR pavement will become part of the structural design to upgrade the surface. The samples should be prepared using either the Hveem or Marshall compaction method. Table 2.4 gives a suggested sample preparation procedure for either the Hveem or Marshall method. Once compacted and cured as prescribed the samples are tested for stability, resilient modulus, and fatigue. Suggested criteria (as of 1988) for selection of the final emulsion content are given in Table 2.5.

2.2.2 Structural Design

One of the objectives of this study was to develop structural layer coefficients for CIR mixtures. These coefficients would be used to determine the required thickness of the CIR pavement. To date the results would indicate that CIR mixtures are essentially equivalent to conventional hot-mix. This is supported when the fatigue lives of CIR

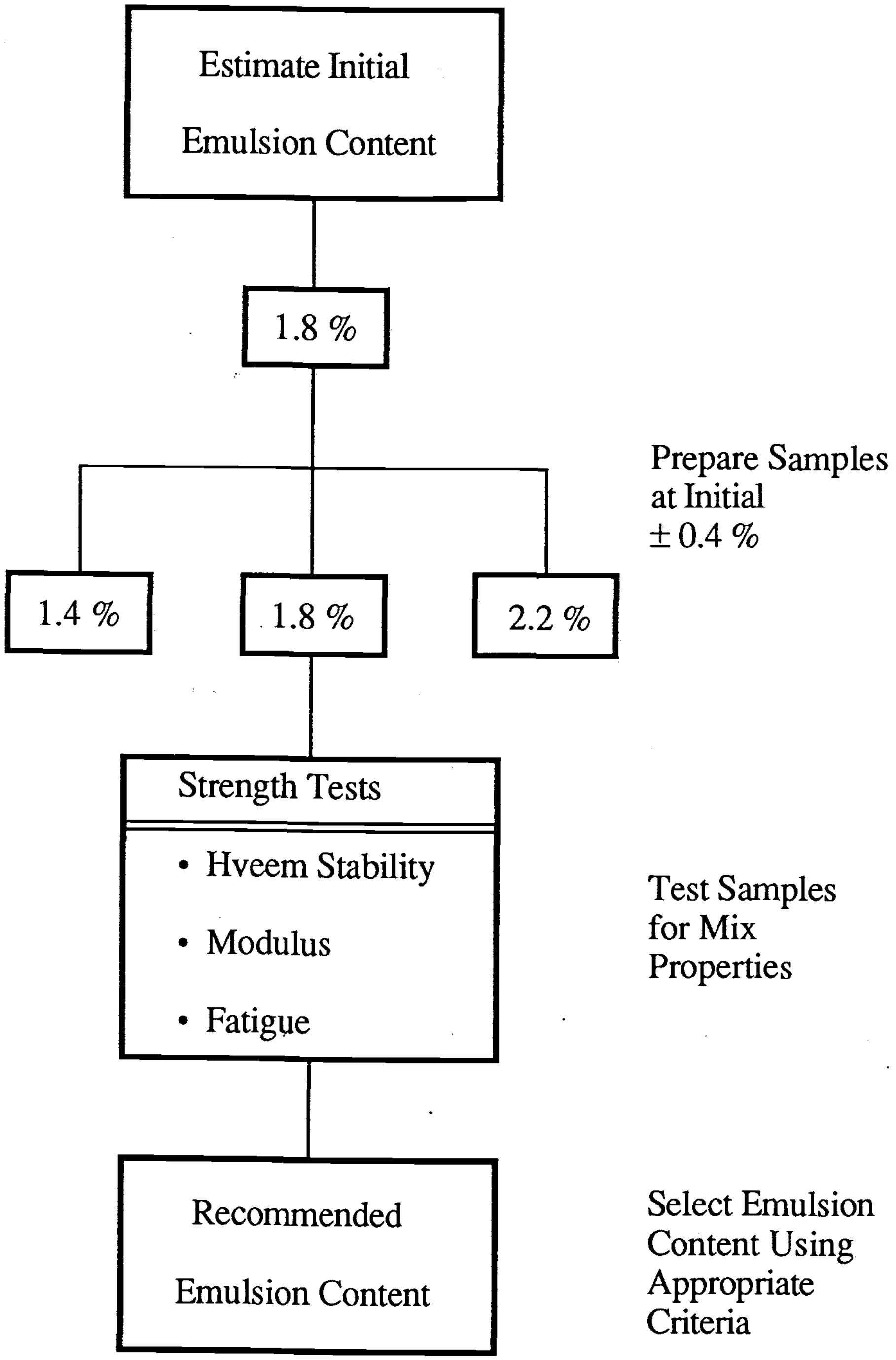


Figure 2.4. Suggested Mix Design Process - Future Projects.

Table 2.4. Suggested Sample Preparation Procedure for CIR.

1		Millings are split into approximately 5500 g batches; this size sample provides sufficient material for four 6.4 cm (2.5 in.) in.) high specimens with an 1100 gm sample for moisture determination.
2		Sample is screened on the 2.5 cm (1 in.) sieve. The material re-retained on the 2.5 cm (1 in.) sieve is reduced in size to 100% passing 2.5 cm (1 in.) sieve using 3-lb hammer. This is because the retained 2.5 cm (1 in.) is too large for 10.2 cm (4 in.) molds.
3		Batch five 1100 gm samples of millings at the average gradation.
4		Determine moisture content of one batch by drying 24 hrs at 110°C (230°F).
5		Samples are heated to 60°C (140°F) ± prior to mixing (1-2 hrs).
6		Water is added to the millings in the appropriate proportion based on the dry weight of the millings: % water = 4.5 total liquid - % added emulsion. Water is thoroughly mixed into millings by hand.
7		Emulsion is added to the premoistened millings after water addition using the recommended content. The added emulsion is based upon the dry weight of the millings. The emulsion is preheated to 60°C (140°F) ± (1 hr) and mixed thoroughly into the batch by hand or using a mechanized mixer.
8		The material is spread into a 30.5 cm x 41.2 cm (12 in. x 17 in.) baking pan and allowed to cure for 1 hr at 60°C (140°F) ± to simulate average time elapsed between paver laydown and initial compaction during actual construction.
9		Samples are molded using standard Marshall or Hveem procedures to produce 6.4 cm (2.5 in.) ± high briquets as described below:
	a	Molds are preheated to 60°C (140°F) ±.
	b	Compact samples using standard 50 blow compactive effort for Marshall procedure or 150 blows at 3.1 MPa (450 psi) for the Hveem procedure.
	c	Cure overnight at 60°C (140°F) and recompact using 25 blows per side for the Marshall procedure and 75 blows at 3.1 MPa (450 psi) for the Hveem procedure.
	d	The molds are laid on their side and the briquets are cured for 24 hrs at 60°C (140°F) ± prior to extrusion.
	e	Briquets are extruded with the compression testing machine.
	f	Briquets are laid on their side to maximize surface exposure and cured for 72 hrs at ± room temperature prior to testing.
10		Specimens are tested for stability, modulus, and fatigue at 25°C (77°F).

Table 2.5. Suggested Mix Design Criteria.

Property	Recommended Value
Hveem stability	> 10 after 2nd compaction
Resilient modulus @ 25°C (77°F)	1.0–2.1 GPa
Modulus ratio @ 25°C (77°F) after saturation	> 0.60
Fatigue life @ 100 $\mu\epsilon$ @ 25°C (77°F)	> 5,000

Table 2.6. Comparison of Fatigue Lives for CIR and Conventional Hot Mixes.

Mix Type	Fatigue Life at 100 $\mu\epsilon$		
	Minimum	Maximum	Mode*
CIR	21958	61337	22000–33000
Hot Mix	6890	46600	15000–27000

*Most frequently occurring range.

mixes are compared to conventional hot mixes. Table 2.6 shows a comparison of the fatigue lives of CIR mixes to those of conventional hot mixes. The fatigue values for both mix types were obtained under the same test conditions. The hot mix is a Class "B" mix with 4.5 to 7% of AR4000 and AR8000 grades of asphalt (4,5,6).

As indicated, CIR mixes generally have about the same fatigue lives as the hot mixes. Thus, the structural layer coefficients can be expected to be about the same as those for conventional hot mixes. Further field work is needed to verify whether the long-term performance of CIR mixes is similar to that of conventional hot mixes.

2.3 Construction Process

Three methods of construction were used for the 1984-88 projects. The 1984 work was accomplished using a roto-mill-grader laydown operation, a Class III treatment. The 1985 through 1987 work was accomplished using both the "recycling train" and a "single unit" machine. Valentine Construction Company of Vancouver, Washington, furnished the recycling train and most of their work would be classified as Class I or II treatments. The Oregon DOT maintenance team, on the other hand, relied on the use of a "single unit" machine (Class I and II treatments). Each construction method is discussed below.

2.3.1 Roto-Mill-Grader Laydown

This operation involved the use of a CMI 450 and an Ingersoll Rand MT 6520. Both these machines have 2.0 m (6-1/2 ft) milling heads. The millings were discharged to the side of the 2.0 m (6-1/2 ft) wide cut, an application of CMS-2S as a tack coat was applied to the milled area with a conventional distributor. The windrow was furrowed to retain the water applied from a water tanker. Windrow was rolled over, furrowed again, and emulsion applied. Two motor graders road mixed the material into the cut, and bladed to grade. Trimming the final surface was required to correct the surface profile. The trimmed surface, however, resulted in tire noise that was objectionable to the public. This process was used on 11.3 km (7 miles) in 1984 with fair to good results.

2.3.2 Recycling Train

In the train method, the train was led by a water tank, and then a CMI 1000 rotomill having a 3.8 m (12 ft-6 in.) milling head. The mill pulled a trailer mounted screen deck, roll crusher, and pugmill followed by a nurse tanker for the emulsion.

The existing pavements were milled to depths of between 4 and 6 cm (1-1/2 and 2-1/4 in.). The millings were screened on a 4 cm (1-1/2-in.) screen and the oversized millings were crushed. CMS-2S emulsion was added and mixed in the pugmill. This mixture was deposited in a windrow on the roadway about 33.5 m (110 ft) from where it was removed. Tack was applied to the milled surface using a spray bar attached to the rear of the train. Laydown was accomplished with standard paving equipment.

The train has controls to monitor the quantity of emulsion and water. To avoid difficulties in handling of the mixture, the paving machine was normally operated within 30.5 m (100 ft) of the train. After laydown, a two-stage compaction was specified. The initial compaction was accomplished using a rolling pattern of one pass vibratory and one pass static with an Ingersoll Rand model DA-50 double drum vibratory roller and one pass static using a Hyster model 15-7 tandem steel wheel roller. The mat was opened to traffic immediately following initial compaction. The second compaction followed within 3-15 days. This consisted of two passes of a Hyster 71 kN (7.2 metric ton) double drum roller in static mode and two passes with the Ingersoll Rand vibratory roller.

After initial compaction, the recycled pavement surface was opened to traffic. If humps or rough spots existed, they were removed by texturing with a milling machine. Following second compaction, the pavement was covered with a 1 cm (3/8 in.) single chip seal using a CRS-2 or a polymer modified (HFE-150S) emulsion.

2.3.3 Single Train Unit

The single unit process involved the use of a RAYGO Barco Mill 800. This unit has a 3.8 m (12-1/2 ft) milling head and was serviced by a water and emulsion tanker. The modification was made to the unit to include a spray bar for applying tack immediately ahead of the windrow. Placement was accomplished using a conventional laydown machine. The unit was used on approximately 32 centerline kilometers (20 miles) of recycling projects in 1984-85 with good results.

2.3.4 Evaluation of Construction Process

The most significant findings from the 1984-86 experiences included:

1) Two to three hours of cure with surface temperatures > 32°C (90°F) are required prior to uncontrolled traffic.
2) Mix density will increase when rolled after 3 to 10 days cure.
3) Excess emulsion will cause an unstable product.
4) Excess water will cause asphalt to flush to surface.
5) Low emulsion causes the material to ravel.
6) Low water results in mix segregation, raveling under traffic and/or high void contents.

7) Fine gradations (less than 2.5 cm (1 in.) maximum size) reduces tolerance for water and emulsion deviations.

8) Coarse gradations (excess of 5 cm (2 in.) maximum size) may cause problems with laydown, dragging, and excess voids.

2.4 Field Performance (1984-1988)

Beginning in 1986, the following field and laboratory data were collected in order to evaluate the field performance of the CIR projects:

1) Pavement condition (visual surveys)
2) Ride (Mays Meter)
3) Mix properties (resilient modulus, fatigue, Marshall stability, and flow).

These data are summarized in the following sections.

2.4.1 Performance Condition

Visual condition surveys were conducted on several of the projects in the fall of 1986 and 1987 and in the spring of 1988. Pavements were rated on a scale of 1 to 5 with 1 being a condition rating of very good as prescribed by the Oregon State Highway Division's (OSHD) rating procedure (8). A summary of the condition of selected projects is given in Table 2.7. As indicated, the projects with no mix design (1984) and those with the initial mix design (1985) are in good condition. However, those with the modified mix design (1986-) are performing fair to very good. It should be noted that conventional mixes placed in these areas also deteriorate at the same rate. This is due to a great extent to the extremely severe weather conditions which prevail in central to eastern Oregon. Also, corrective work was required in the southbound lane of the Warm Springs project due to

Table 2.7. Pavement Condition of Selected CIR Projects (Spring 1988).

Project	Year Constructed	Condition	Comments
Century Drive	1984	Fair to Good	Surface not sealed before winter
Sisters-Redmond	1985	Good	75% of work chip sealed
Harney Co. Line-Hogback Summit	1985	Good	Some potholes
Drews Gap-Lakeview	1985	Fair to Good	Thermal cracking
Warm Springs	1986	Fair to Poor	NB Good; SB Poor
Powell Butte-Prineville	1986	Fair to Good	Areas with inadequate seal show block cracking
Lake of the Woods	1986	Very Good	

sealing of the surface before the water content in the mix had sufficiently dropped.

2.4.2 Ride

Ride data were collected on several of the CIR projects using the Mays ride meter. Data were obtained immediately before and after construction as well as each year after construction. These data are summarized in Table 2.8 while Table 2.9 indicates the criteria used to rate the smoothness of the pavement. As indicated, ride was markedly improved on the two projects that were rated rough before construction. However, for the two projects (Warm Springs and Lake of the Woods) that had a smooth rating before construction, the CIR work retained the ride rating.

2.4.3 Mix Properties

Field cores were extracted from several of the CIR projects beginning in 1986. Tests conducted on these cores were as follows:

1) bulk specific gravity,
2) resilient diametral modulus and fatigue, and
3) Marshall stability and flow.

These data are summarized in Table 2.10. Figure 2.5 displays the modulus and fatigue results graphically while Figure 2.6 graphically displays the Marshall stability and flow results.

As indicated, the modulus values increased with time for most sections. These increases were expected owing to the additional curing time and densification due to traffic. The modulus for the Warm Springs and Lake of the Woods projects, however, decreased slightly

Table 2.8. Before and After Ride Data for Selected CIR Projects.

Project	Year Constructed	Average Ride (m/km)		Score Rating	
		Before	After	Before	After
Harney Co. Line-Hogback Summit	1985	2.8	1.0	Rough	Smooth
Warm Springs	1986	1.1	1.1	Smooth	Smooth
Powell Butte-Prineville	1986	2.6	1.8	Rough	Slightly Rough
Lake of the Woods	1986	1.1	0.98	Smooth	Smooth

Note: 1 in./mile = 16 mm/km

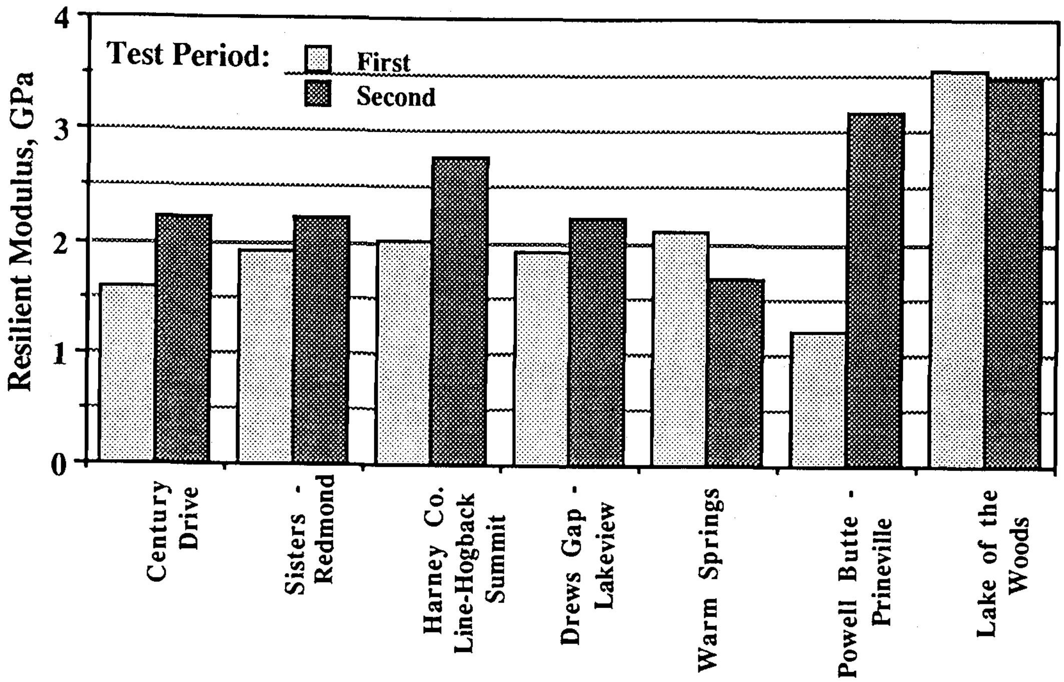
Table 2.9. Criteria to Rate the Smoothness of a Pavement Using the Mays Ride Meter.

Mays Reading (m/km)	Rating
3.2+	Very Rough
2.4-3.2	Rough
1.6-2.4	Slightly Rough
1.2-1.6	Average
0-1.2	Smooth

Table 2.10. Summary of Mix Property Test Results.

Project	Test Period (Months After Construction)	Average Bulk Specific Gravity	Average Resilient Modulus* (GPa)	Average Fatigue Life*	Average Marshall Stability** (kN)	Average Flow (mm)
Century Drive (1984)	15	–	1.59	–	–	–
	17	2.203	2.22	77800	–	–
Sisters-Redmond	3	2.160	1.92	17077	–	–
	5	2.207	2.23	65930	–	–
Harney Co. Line-Summit Hogback	3	–	2.02	–	–	–
	5	1.946	2.78	35072	–	–
Drews Gap-Lakeview	3	1.940	1.92	3424	–	–
	5	2.005	2.23	19317	–	–
Warm Springs	3	2.160	2.10	11030	3.09	1.50
	12	2.333	1.67	50010	3.83	0.51
Powell Butte-Prineville	3	2.121	1.21	8110	1.00	0.51
	12	2.316	3.16	32325	3.25	0.46
Lake of the Woods	3	2.059	3.54	5860	2.69	0.74
	12	2.092	3.47	34261	2.73	0.51

*Tests run at 23°C, 100 microstrain, and at a load frequency of 1 hertz
**ASTM D4123



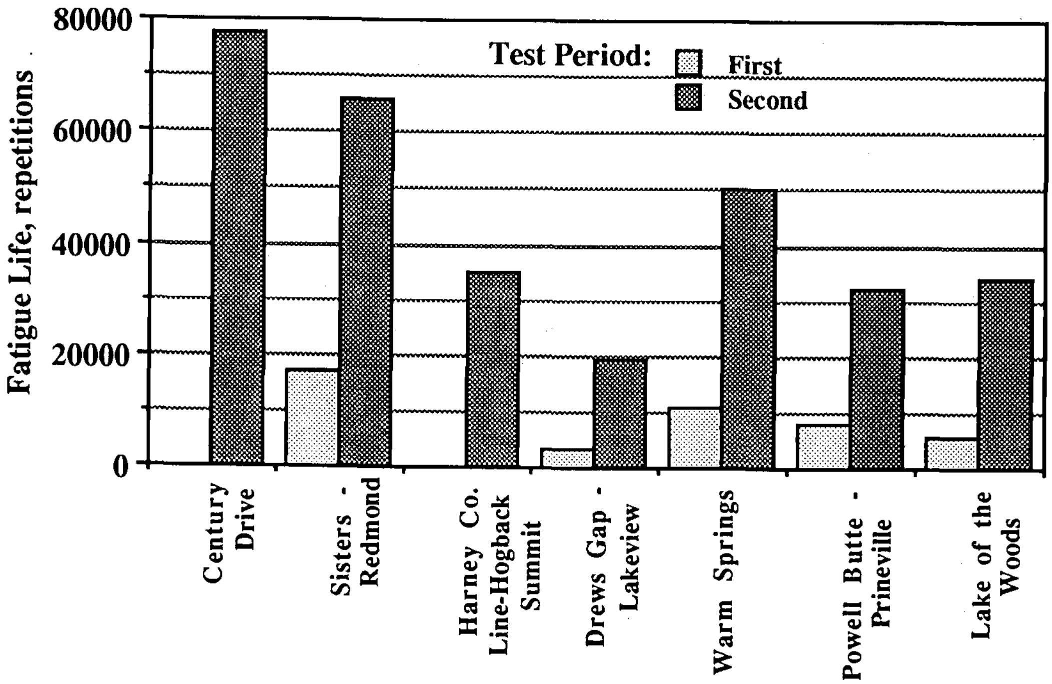
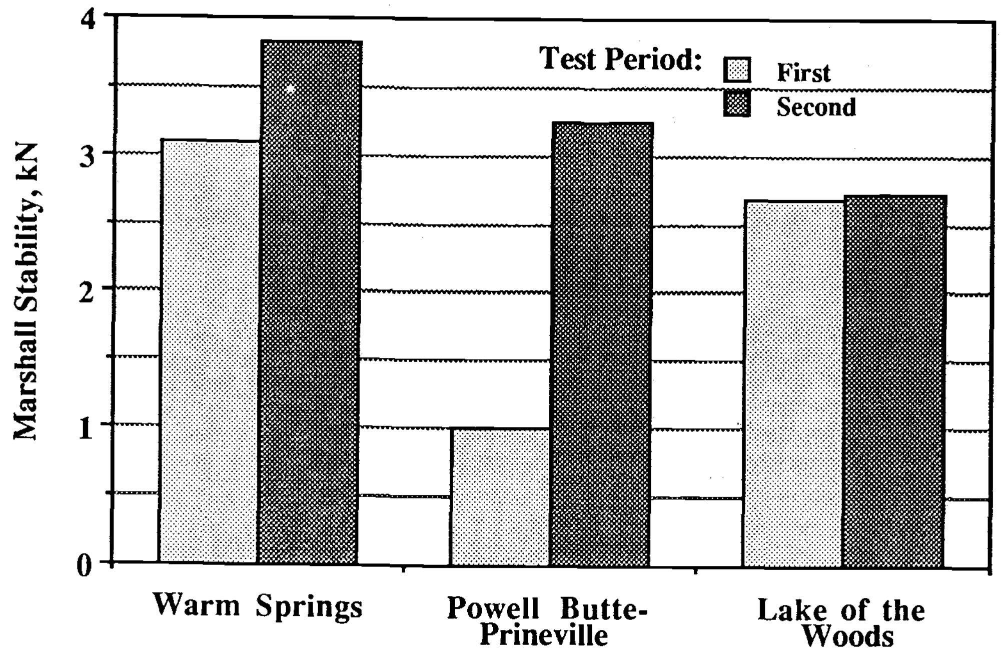


Figure 2.5. Resilient Modulus and Fatigue Test Results (1 ksi = 6.9 MPa)



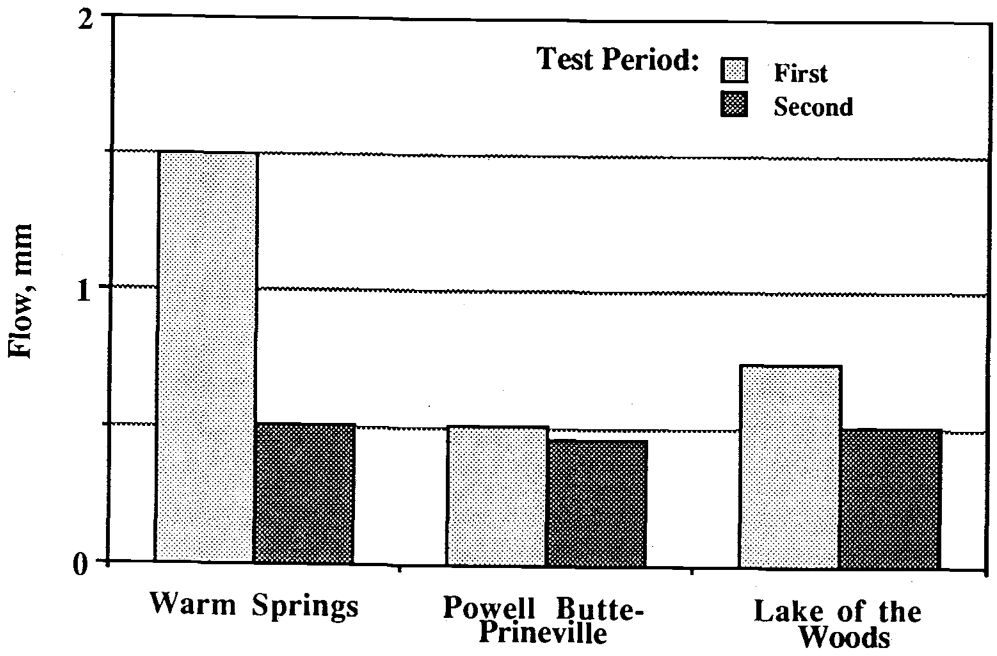


Figure 2.6. Marshall Stability and Flow Test Results
(1 lb = 4.45 N and 1 mil = 0.025 mm)

over time. In all cases where fatigue was monitored over time, the fatigue lives increased significantly. This may also be attributed to the additional cure time and densification. Also, the fatigue lives are comparable to conventional mixes (i.e., 10,000-50,000 repetitions to failure).

In addition to modulus and fatigue, Marshall stability and flow were monitored over time for the 1986 projects. In all cases the stabilities increased but remained slightly low (less than 3.5 kN) and the flow values decreased but remained slightly high (greater than 0.5 mm). These results generally reflect, and thus support, the modulus and fatigue test results.

2.5 Conclusions

The results presented in this paper appear to warrant the following conclusions:

1) The mix design procedure provides a rapid and simple method of determining emulsion content. Further refinement of the adjustment values is continuing and will be available in the 1989 reports.
2) The laboratory tests used in the mix design procedure are widely used and accepted.
3) The mix design procedure eliminates the necessity to fabricate, compact, and cure test briquets in the laboratory which reflect actual field conditions - one of the more controversial design issues for CIR (2).
4) The mix design results generally produce the optimum emulsion content within a fraction of a percent.
5) For most recycle projects where preservation and restoration of an existing pavement is the primary objective, the estimated design emulsion content would be adequate for the final recommended design.
6) Thickness design for CIR pavements may be accomplished using standard procedures for conventional hot mix overlays (7).
7) Both the recycling train and the single train unit are effective in producing a quality CIR base or wearing course.

8) Most sections are generally performing fair to good. The exceptions are all associated with using too high an emulsion content or stripping caused by sealing the surface before curing was completed.

9) Cold in-place recycling markedly improves the ride on rough surfaces.

10) Mix property test results on cores indicate the mix properties are comparable to those expected from conventional hot mixes (7).

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3.0 MIX DESIGN PRACTICES FOR COLD IN-PLACE RECYCLED PAVEMENTS

By

Todd Scholz,[1] R.G. Hicks,[2] and Dale Allen[3]

ABSTRACT

Since 1984, Oregon State Highway Division has constructed 724 km (450 centerline miles) of cold in-place recycled pavements. During this period, an intensive study was undertaken by Oregon Department of Transportation (ODOT) and Oregon State University (OSU) with the following purposes:

1) develop an improved mix design procedure for cold in-place recycled pavements,
2) evaluate the structural contribution of the cold in-place recycled pavements, and
3) develop improved guidelines and specifications for the construction of cold in-place recycled pavements.

This paper summarizes the development of a mix design process and describes the early procedures used and their limitations.

The first section of the paper describes the recycling process used during the 1984-88 period. The second part of the paper describes the evolution of the mix design process used during this period. It de-

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[1]Research Assistant, Department of Civil Engineering, Oregon State University, Corvallis, OR 97331

[2]Professor of Civil Engineering, Oregon State University, Corvallis, OR 97331

[3]Region Engineer, Oregon State Highway Division, Bend, OR 97708

scribes the steps needed to estimate the emulsion content from properties such as RAP gradation, residual asphalt content, and asphalt properties (penetration and viscosity). Also described are the mix design criteria developed to date (stability, modulus, etc.). The third part describes the expected ranges in strength properties (stability, modulus, etc.) for cold recycled mixes.

The last part of the paper presents the significant conclusions from the work performed to date as well as recommendations for further study.

Key Words: Cold in-place recycling, mix design, mix properties, modulus, fatigue, stability, emulsion.

3.1 Introduction

With the national trend away from new construction to preservation of existing pavements, several highway agencies are turning to cold in-place recycling (CIR) as an approach to rehabilitating distressed pavements. However, many agencies remain skeptical of the use of CIR due to the lack of long-term performance data and adequately documented field engineering studies. Furthermore, due to variability in construction processes with substantially different design concepts and end results (1,2,3), the term CIR is often misunderstood.

Recycling may be defined as the re-use, after processing, of a material that has already served its intended purpose. For the purposes of this paper, the different construction processes for cold in-place recycling are defined as follows:

1) Class I. This recycling treatment is performed on a uniform pavement designed and built to specifications. It is expected that a rational CIR mix design can be prepared and produced. The treatment could handle medium to heavy traffic volumes, usually as a base on high volume roads or as a wearing course on low volume roads. The recycling train method would normally be used; however, depending on the degree of distress, a single-unit train could also produce a Class I treatment. Treatment width is normally 3.6 m (12 ft).

2) Class II. This recycling treatment is performed on a pavement with significant maintenance patches over a uni-

form pavement or a pavement with minimal design used in the original construction. Either the recycling train or the single-unit train can produce millings of sufficient quality for reasonable mix designs. The finished mix can be used as a base or wearing course as in the case of the Class I process. Treatment width is normally 3.6 m (12 ft).

3) Class III. This treatment is used on low volume highways where considerable variation in pavement structure exists and it may incorporate additional aggregate. The design of the mix is limited. Various milling and pulverizing units can be used to perform this operation. The treatment is normally used as a stabilized base course. Treatment width varies from 1.2 to 3.6 m (4 to 12 ft).

The Oregon State Highway Division is one of the several agencies that has attempted cold in-place recycling as an approach to rehabilitating distressed asphalt concrete (AC) pavements. Oregon first experimented with partial depth CIR work in 1984 totaling 22 km (14 miles). An additional 110 km (68 miles) of AC pavement was cold recycled in 1985. Spurred by the initial success of these projects and recognition of the need for a formal mix design procedure, ODOT and OSU, in 1986, undertook a joint study of cold in-place recycling. The study involved investigating seven of 13 projects cold recycled in 1986 in order to develop an improved understanding of the relationship between mix design and field performance of cold recycled pavements. The specific objectives of the study were to develop an improved mix design, evaluate the structural contribution and durability, and develop improved construction guidelines and specifications for cold in-place recycled pavements.

This paper describes the history of CIR in Oregon from 1984 to 1988 including project information and the construction process used on the projects; the evolution of the mix design process for CIR; and ranges of strength properties (modulus, stability, etc.) to be expected from cold recycled mixes. Also presented are significant conclusions from the work completed to date as well as recommendations for further research.

3.2 History of Cold Recycling in Oregon

Project information associated with the 1984-88 CIR work as well as the process used to construct the projects are described in this section.

3.2.1 Project Information

To date 724 km (450 miles) have been cold recycled in Oregon since 1984. All projects were constructed in Regions 2, 3, 4, and 5. Table 3.1 summarizes construction information for all of the Region 4 projects totaling 619 km (385 miles) constructed between 1984 and 1988. Information for the projects constructed in Regions 2, 3, and 5 was unavailable at the time this paper was written.

3.2.2 Construction Process

Oregon's first efforts (1984) at CIR involved exclusively Class III recycling. The construction process was accomplished with a roto-mill having a 2.0 m (6.5 ft) milling head and a motor grader. The surface was milled with the roto-mill which discharged the millings into a windrow to the side of the cut. Water and CMS-2S emulsion was then applied to the windrow. The windrow was then mixed with the motor graders and bladed into the cut.

All subsequent work (1985-88) was accomplished using either the "recycling train" or a "single-unit" machine. The work done with the recycling train was contracted out to a construction company that owned the equipment and most of their work would be classified as Class I or Class II treatments. The Oregon DOT maintenance team, on

Table 3.1. Projects Constructed in Oregon (1984-88).

Year	Highway	Project Name	Traffic Volume (ADT)	Length[1] (km)	Depth of Cut[2] (cm)	Emulsion Type (Content)	Class of Treatment	Surface Treatment	Performance Spring 1988
1984	OR 372	Sand Shed-Mt. Bachelor (Intermittent)	820	7.7	4	CMS-2S (1-2%)	Class III, State forces, grader laid	Surface left open winter of 1984.	Good
	Misc.	Bend area	Up to 2000	14.5	4	CMS-2S (1-2%)	Class III, State forces, grader laid	About 50% chip sealed	Good
1985	US 26	Sisters-Redmond	1450-8300	30.2	4	CMS-2S (1-2%)	Class II	Chip seal placed on about 75% of work	Good
	US 395	Harney Co. Line-Hogback Summit	220	49.4	4-5	CMS-2S[3] (1-2%)	Class I	Chip seal	Good
	US 140	Drews Gap-Lakeview	1000	16.6	4-5	CMS-2S (1-2%)	Class I	Polymer chip sealed	Fair
	Misc.	Bend area	Up to 23,000	19.3	4-5	CMS-2S (1-2%)	Class I	80% chip sealed	Good
1986	US 26	Warm Springs	2850	27.8	5-10	CMS-2S (1%)	Class I	Polymer chip sealed	Fair-Poor
	OR 41	Powell Butte-Prineville	3600	15.8	5	CMS-2S (1.2%) HFE-150 (1.2%)	Class I	Chip seal	Fair-Poor
	OR 270	Lake of the Woods	1750	10.2	6-10	CMS-2S (1.4%)	Class I	Chip seal	Very Good
	US 20	Bend-Powell Butte	4800	5.1	4-5	CMS-2S (1.5%)	Class I	Chip seal	Good
	OR 371	MP 18.0-Powell Butte	2200	29.0	4-5	CMS-2S (1.1-1.3%)	Class II	Chip seal	Good
	US 26	Ochoco Dam-MP 35.0	1100	17.0	4-5	CMS-2S (1.1-1.6%)	Class II	Chip seal	Good
	US 26	MP 73.4-MP 81.6	600	13.2	4-5	CMS-2S (1.8-2.6%)	Class II	8 cm overlay	Good
	US 26	MP 89.6-Jct. OR 19	600	14.0	4-5	CMS-2S (1.4-1.5%)	Class I	Chip seal	Very Good
	US 20	MP 75.0-MP 84.0	1000	14.5	4-5	CMS-2S (1.5-1.6%)	Class I	2 cm oil mat	Fair
	OR 423	US 97-OR 39	800	11.3	4-5	CMS-2S (1.5%)	Class I	Chip seal	N/A†
	OR 140	Dairy-Ritter Rd.	2000	9.6	4-5	CMS-2S (1.2-1.9%)	Class II	Chip seal	Fair
	OR 140	Sprague River Rd.-Bly	2700	28.6	4-5	CMS-2S (1.5%)	Class II	Chip seal	Fair
	US 97	MP 235.3-Spring Creek	3400	9.6	4-5	CMS-2S (0.9%)	Class I	Chip seal	Poor
	OR 7	W-Horse Ridge-Crooked River Hwy	900	14.8	8	CMS-2S (1.7%)	Class I	Chip seal	Good
	OR 372	Kiwa Springs-Sand Shed	880	9.0	5	CMS-2S (1.0%)	Class I	Chip seal	Good

Table 3.1. Projects Constructed in Oregon (1984-88) (continued).

Year	Highway	Project Name	Traffic Volume (ADT)	Length[1] (km)	Depth of Cut[2] (cm)	Emulsion Type (Content)	Class of Treatment	Surface Treatment	Performance Spring 1988
1987	OR 41	Antone-MP 89.6	520	12.9	4	CMS-2S (1.7%)	Class I	Chip seal	Good
	OR 293	Jct. US 97-Tub Springs Rd.	200	14.5	5	CMS-2S (2.8%)	Class I	Chip seal	Good
	OR 360	Jct. US 97-SE Rammes Rd.	1000	14.5	5	CMS-2S (1.6%)	Class I	Chip seal	Good
	OR 380	Conant Basin Rd.-Shotgun Rd.	180	14.6	5	CMS-2S (1.0%)	Class I	Chip seal	Good
	OR 4	Fuego Rd.-Forge Rd.	3350	15.9	5	CMS-2S (1.2%)	Class I	None	Poor
	OR 427	Modoc Secondary	450	18.0	2.5	CMS-2S (1.3%)	Class I	Chip seal	Poor
1988	OR 4	Shaniko Jct.-Quaale Rd.	390	20.6	5	HFE-150 (0.8-1.2%)	Class I	3/4 in. cold mix overlay	Good
	OR 41	Prineville-Ochoco Dam	2200	11.3	5	HFE-150 (1.8%)	Class I		Good-Very Good
	OR 41	Ochoco Ranger Sta.-Ruch Creek	800	31.7	5	HFE-150 (0.6%)	Class I	Chip seal	Very Good[4] Poor[5]
	OR 380	Jct. Ochoco Hwy-Conant Basin Rd.	3100	33.3	5	HFE-150 (2.6%)	Class I		Good
	OR 50	Merill Jct.-Hatfield Hwy	3600	4.2	5	CMS-2S (1.2%)	Class I	None	Good
	OR 426	Jct. Klamath Falls-Malin Hwy to Calif. Line	2350	4.8	5	CMS-2S (0.5%)	Class I	None	Good
	OR 42	DeMoss Springs-Moro	1800	8.0	5	HFE-150 (1.0%)	Class I	Chip sealed with HFE	Very Good
	OR 19	Cogswell Creek-New Pine Creek	600	9.2	5.5	CMS-2S (1.7%)	Class I	Chip sealed with HFE	Good
	OR 20[6]	Beatty-Ivory Pine Rd.	980	15.3	5.5	CMS-2S (0.3%)	Class I	None	N/A
	OR 22	Fort Klamath-Crooked Creek	550	8.7	5	HFE-150 (1.1%)	Class I	Sand seal	Very Good
	OR 49	Lake Abert-Valley Falls	260	6.4	5	CMS-2S (1.0%)	Class I	Chip seal	Good
	OR 22	Crater Lake Hwy-Frontage Rd.	520	5.6	5	CMS-2S (1.7%)	Class I	Chip seal	Good

[1] 1 mile = 1.609 km
[2] 1 in. = 2.54 cm
[3] HFE-150 and HFE-150S were also used, but only for test
[4] 15.3 km
[5] 16.4 km
[6] One lane only
†Not Available

the other hand, relied on the use of a single-unit machine (Class I or Class II treatments). Both construction methods are discussed below.

Recycling Train. In the train method, the train was led by a water tanker, and then a CMI 1000 roto-mill having a 3.8 m (12.5 ft) milling head. The mill pulled a trailer-mounted screen deck, roll crusher, and pugmill followed by a nurse tanker for the emulsion.

The existing pavements were milled using the CMI 1000 to depths between 3.8 and 5.7 cm (1.5 to 2.25 in.). The millings were transferred via conveyor belt to the screen deck and screened over 3.8 to 5 cm (1.5 to 2 in.) screens. The oversized millings were crushed such that 100% passed the 5 cm (2 in.) screen. Emulsion was added and mixed with the millings in the pugmill. This mixture was deposited in a windrow on the roadway. A diluted CMS-2S tack was applied to the milled surface using a spray bar attached to the rear of the train.

The train has controls to monitor the quantity of emulsion and water. To avoid difficulties in handling of the mixture, the paving machine was operated within 30 to 60 m (100 to 200 ft) of the train. After laydown, a two-stage compaction was specified. The initial compaction was accomplished using a rolling pattern of one pass vibratory and one pass static with an Ingersoll Rand model DA-50 double drum vibratory roller and one pass static using a Hyster model 15-7 tandem steel wheel roller. The mat was opened to traffic immediately following initial compaction. The second stage compaction followed within 3 to 12 days. The variation in days elapsed until second compaction is due to the amount of cure the recycled pavement has undergone which is dependent primarily on pavement temperature and moisture content. That is, with high pavement temperatures and low moisture content, second compaction may be appropriate after only three days following pavement recycling while up to 12 days may be appropriate for low pavement temperatures and high moisture contents following pavement recycling. This consisted of at least two passes of a Hyster 71 kN (8 ton) double drum roller in static mode and at least two passes with a 178 kN (20 ton) pneumatic roller. It should be noted that the second stage compaction is more effective than the initial compaction. This is because second stage compaction results in a mat at the same (or nearly the same) density that exists in the wheel tracks which have been compacted under traffic since initial compaction. That is, the second stage compactive effort merely "levels" the surface to match the compaction in the wheel tracks due to traffic.

If humps or rough spots existed in the recycled mat after second compaction, they were removed with a milling machine or corrected with skin patches before sealing. Two weeks or more after recycling, the pavement was covered with a 10x2 mm (3/8 in.x #10) single chip seal (using a CRS-2 or a polymer modified (HFE-150) emulsion) or a fog/sand seal. Through experience it has been found that a fog/sand seal is best for pavements with a relatively tight surface and having soft asphalt properties. A chip seal, on the other hand, would be appropriate for a cold recycled mat with an open texture.

Single-Unit Train. The single-unit process involved use of a RAYGO Barco Mill 800. This unit has a 3.8 m (12.5 ft) milling head and was serviced by a water and emulsion tanker. A modification was made to the unit to include a spray bar for applying tack immediately ahead of the windrow. Placement was accomplished using a conventional paver. Initial compaction was the same as for the recycling train, but the second stage compaction was normally done with only a vibratory roller since a 178 kN (20 ton) pneumatic roller was not available.

3.3 Evolution of Mix Design Process

This section describes the evolution of the mix design process used for cold in-place recycled pavements in Oregon.

3.3.1 Initial Design Process: 1984-86

Formal mix designs were not available for the 1984 projects. Emulsion and water contents were established in the field using trial-and-error procedures by experienced paving personnel and this work would be classified as Class III recycling. In general, the emulsion contents were about 1.5% while the water contents varied from 2 to 4%.

A formal mix design procedure (Table 3.2) for Class I and Class II recycling was first attempted by Oregon in 1985. This procedure used the existing Oregon standard open-graded mix design procedure which is basically a modification of Oregon's hot-mix procedure. Design criteria used for the 1985 projects were:

Film thickness	sufficient-thick
Hveem Stability:	
After 1st compaction at 25°C (77°F)	> 20
After 2nd compaction at 60°C (140°F)	> 10
Voids after 2nd compaction	5-8%
Index of Retained Strength (IRS)	60% minimum
Modulus Ratio	not used

Using the above criteria, recommended emulsion and water contents were given to field personnel. The recommended values, however, were almost always too high and were adjusted in the field to eliminate rutting and flushing.

Table 3.2. Mix Design Procedure Used for the 1985 CIR Projects

1) Determine gradation of millings from reclaimed asphalt pavement (RAP).

2) Extract asphalt using hot reflux and recover asphalt using modified Abson procedure and determine penetration at 25°C (77°F), kinematic viscosity at 135°C (275°F), and absolute viscosity at 60°C (140°F).

3) Determine percent asphalt in the RAP and gradation of the Aggregate after asphalt extraction.

4) Determine mix design moisture content that would provide saturated surface damp (SSD) millings.

5) Make 4 trial mixes by varying the CMS-2S emulsion content (1, 2, 3, and 4%) increments while holding the water content constant. Record the coating (film) thickness for each emulsion content.

6) Place mix in bread pan at 25°C (77°F) for 24+ hours.

7) Place mix in compaction mold at 25°C (77°F) and apply 20 tamping blows at 1.7 MPa (250 psi) pressure and then compact with 150 blows at 3.4 MPa (500 psi) pressure.

8) Cure compacted specimen at 60°C (140°F) for 15 to 24 hours in mold.

9) Determine 1st Hveem stability at 25°C (77°F) and bulk specific gravity.

10) Return specimen to mold and compact using 6.9 MPa (1000 psi) static load and determine 2nd Hveem stability at 60°C (140°F) and bulk specific gravity.

11) Return specimen to mold and cure at 116°C (240°F) for 3 to 4 hours and continue compaction with 150 blows at 3.4 MPa (500 psi) pressure.

12) Determine 3rd Hveem stability at 60°C (140°F) and bulk specific gravity.

13) Determine Rice specific gravity and percent voids.

14) Determine dry and wet unconfined compressive strength by AASHTO T165 procedure and calculate Index of Retained Strength.

The same procedure was used at the onset of the 1986 construction season. However, it was found in most cases that the recommended values still resulted in emulsion and/or water contents that were too high resulting in unstable mixes. Also, a review of Oregon's 1985 mix design procedure compared with those used by other agencies (1985) found that significant differences existed in the various methods used to: (1) determine the amount of recycling/reclaiming agent (emulsion) to be added, (2) cure the laboratory samples, (3) compact the samples, (4) evaluate the strength of CIR mixes, and (5) evaluate the mixes for stability (4).

Because of these differences and having to field adjust the recommended design significantly, ODOT developed revised mix design guidelines and criteria (mid-summer 1986) for Class I and Class II recycling. A summary of the revised guidelines is given in Table 3.3 while Table 3.4 summarizes the initial and revised design criteria for the 1986 projects. The significant differences between this design procedure and that used in 1985 include:

1) The 1985 procedure calls for the initial laboratory sample cure to be at 25°C (77°F) for 24+ hours while the revised procedure calls for 60°C (140°F) for 15 hours.
2) The revised procedure calls for a second cure (after first compaction) at 60°C (140°F) for 24 hours while the 1985 procedure calls for 60°C (140°F) for 15 to 24 hours.
3) The revised procedure replaces the unconfined compressive strength (AASHTO T165-Effect of Water on Cohesion of Compacted Bituminous Mixtures) used in the 1985 procedure

Table 3.3. Oregon's 1986 Mix Design Guidelines for CIR (Revised Mid-Summer 1986).

1) Determine gradation of reclaimed asphalt pavement (RAP) millings.

2) Extract asphalt using hot reflux and recover asphalt using modified Abson recovery to determine penetration at 25°C (77°F), kinematic viscosity at 135°C (275°F), and absolute viscosity at 60°C (140°F).

3) Determine percent asphalt content in the RAP and gradation of aggregate after asphalt extraction.

4) Determine mix design moisture content to provide saturated surface damp (SSD) condition.

5) Prepare trial mixes at 1.0, 2.0, 3.0, and 4.0% CMS-2S emulsion based on dry weight while holding the moisture content obtained in (4) constant.

6) After mixing for 2 minutes, place mix in bread pan and cure in the oven for 15 hours at 60°C (140°F).

7) Place mix in compactor and apply 20 tamping blows at 1.7 MPa (250 psi) and then compact with 150 blows at 60°C (140°F).

8) Cure compacted specimen at 60°C (140°F) for 24 hours in the mold.

9) Determine 1st Hveem stability at 25°C (77°F) and bulk specific gravity.

10) Return specimen to mold and compact specimen using 6.9 MPa (1000 psi) static load and determine 2nd Hveem stability at 60°C (140°F) and bulk specific gravity.

11) Return sample to mold and cure sample at 116°C (240°F) for 3 hours and continue compaction with 150 blows at 3.4 MPa (500 psi).

12) Determine 3rd Hveem stability at 60°C (140°F) and bulk specific gravity.

13) Determine Rice specific gravity and percent voids.

14) After 1st compaction of resilient modulus specimens, put samples in air bath at 25°C (77°F) for 24 hours and determine unconditioned resilient modulus.

15) Vacuum saturate samples for 30 minutes at 68.6 cm (27 in.) of Hg, rest samples for 30 minutes, and place in 25°C (77°F) water bath for 3 hours, and then determine saturated resilient modulus.

16) Vacuum saturate for 30 minutes, double wrap sample, and place in freezer for 15 hours; remove and place in 60°C (140°F) bath for 30 minutes, remove wrapping, and re-place sample in 60°C (140°F) bath for 24 hours; place in 25°C (77°F) bath for 3 hours, and then determine freeze-thaw resilient modulus.

17) Determine index of retained modulus after vacuum saturation and after one freeze-thaw cycle.

Table 3.4. Mix Design Criteria for 1986 Projects.

a) Initial 1986 Criteria

Item	Criteria
1) Film thickness	Sufficient
2) Coating	70% min
3) Moisture	Surface Damp
4) Void content after:	
1st stability	6-10%
2nd stability	5-8%
3rd stability	1-3%
5) Unconditioned Resilient Modulus (M_R)	1.0 GPa (150 ksi) min
6) M_R Ratio (vac. sat.)	.70 min
7) M_R Ratio (freeze-thaw)	.50 min

b) Revised Mix Design Criteria (mid-1986)

Item	Criteria
1) 2nd stability	10 min
2) Void content after 3rd stability	4-6%
3) Film thickness	Dry-sufficient (60% coating)
4) Minimum emulsion content	1%

with the Indirect Tension Test for Resilient Modulus of Bituminous Mixtures (ASTM D4123) and modulus ratios (5) (freeze-thaw and vacuum saturated).

In reviewing the design emulsion and water contents with those actually used during construction, it was found that the revised design procedure also recommended emulsion and/or water contents that were better but still too high.[4] It was clear that neither the 1985 nor the revised design procedures could accurately recommend emulsion and/or water contents and that an improved procedure was needed.

3.3.2 Improved Design Process: 1987-88

Due to the limitations of the 1985 and revised design procedures (namely the inability to recommend accurate emulsion and/or water contents), Oregon developed an improved design procedure for Class I and Class II recycling (6) prior to the 1987 construction season. The improved procedure used on the 1987 projects consists of estimating the design emulsion (CMS-2S) and water content.

Further refinement of the procedure allowing the use of a high float emulsion (HFE-150), was developed prior to the 1988 construction season. Prior to 1988 the CMS-2S emulsion was used exclusively except for one experimental project. Table 3.5 summarizes the specifications for the CMS-2S and HFE-150 emulsions. Most 1988 CIR work utilized the HFE-150 emulsion because Oregon had (by 1988) considerable experience with the CMS-2S emulsion and was interested in finding out if

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[4]Test sections of 60 to 90 m (200 to 300 ft) were used with emulsion contents ±1 and ±2 from design. At the design emulsion content, or above, the sections became unstable within one week or flushed immediately after rolling.

Table 3.5. Specifications for the CMS-2S and HFE-150 Emulsions.

Test	CMS-2S	HFE-150
Viscosity at 50°C (122°F)	50-450	50 min
Sieve	0.1 max	0.1 max
1 day storage stability, %	1 max	1 max
Residue at 250°C (500°F), %	60 min	65 min
Charge	+ Pass	- Pass
Residue Tests	**CMS-2S**	**HFE-150**
Penetration	100-250	100-300
Oil Distillate, %	5-15	0-7
Float Sec 60°C (140°F)	---	1200 min
Solubility, %	97.5	97.5

the high floats would work as well. The specific details of the improved design procedures are described in the following paragraphs. Reference to the CMS-2S emulsion corresponds to the 1987 projects while reference to the HFE-150 emulsion corresponds to the refinements developed and utilized on the 1988 projects.

Estimating Design Emulsion Content. Estimation of the design emulsion content begins with establishing a base design emulsion content and making adjustments based on the results of the laboratory findings. Oregon has found, through experience with the CMS-2S emulsion, that a base design emulsion content of 1.5% (1.2% for HFE-150) by dry weight of recycled asphalt pavement (RAP) is a good starting point. Adjustments are then made to this base content according to softness of extracted asphalt, gradation of millings (41 cm (16 in.) mill), and the percent of recovered asphalt. Table 3.6 shows the calculations to be made with the adjustments. The final estimated design emulsion content for CMS-2S can be as low as 0.4% and as high as 2.6% (0.6 to 1.8% for HFE-150). The adjustments are discussed in detail below:

1) Softness of Asphalt. The penetration (ASTM D5-Penetration of Bituminous Materials) and absolute viscosity (ASTM D2171-Viscosity of Asphalts by Vacuum Capillary Viscometer) laboratory test results are used to determine the softness of extracted asphalt. Figure 3.1 indicates the ranges in these values that have been found in CIR completed to date. By plotting the values obtained from the laboratory on this figure, an adjustment of up to ±0.5% can be selected for

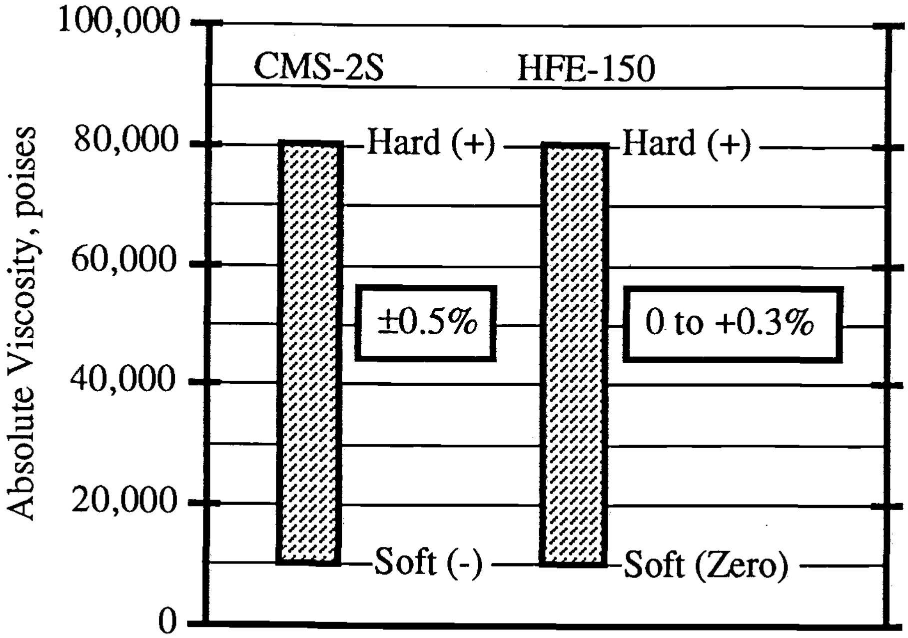
Table 3.6. Proposed Adjustments from Base Design.

a) Adjustments

	CMS-2S	HFE-150
Base Design, %	1.5	1.2
Adjustment for <u>Softness</u>, %	±0.5	0 to +0.3
Adjustment for <u>Gradation</u>, %	±0.3	±0.3
Adjustment for <u>Asphalt</u>, %	±0.3	0 to -0.3
Final Estimated Design, %	______	______
Lowest Design, %	0.4	0.6
Highest Design, %	2.6	1.8

b) Example

	CMS-2S	HFE-150
Base Design, %	1.5	1.2
Softness is average with absolute viscosity 50,000 poises and/or penetration 25 dmm	0.0	+0.1
Gradation is very coarse with 5% passing the #10 and 30% passing the 1/4 in. screens	+0.3	-0.2
Extracted asphalt content is 7% or higher	-0.2	-0.3
Final Estimated Design, %	1.6	0.8



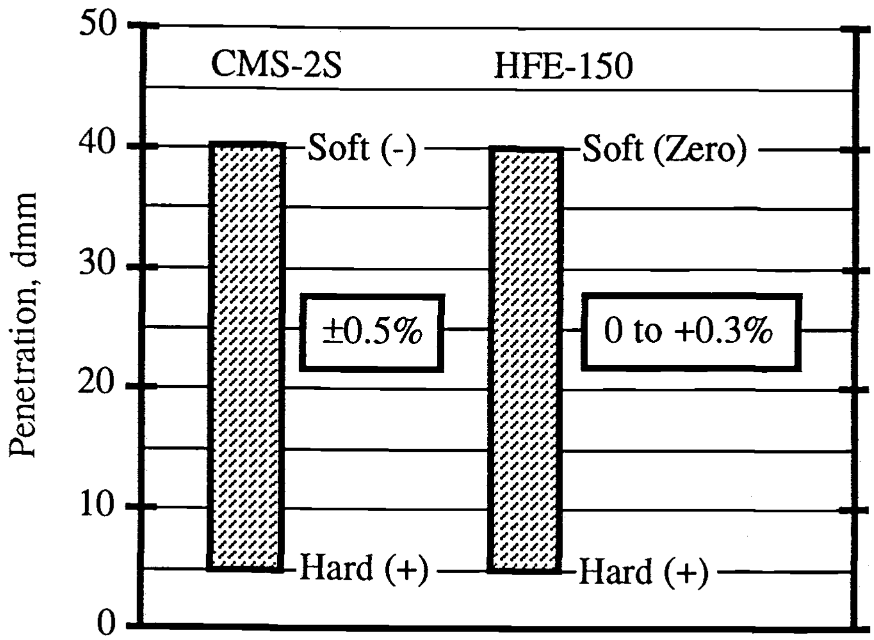


Figure 3.1. Softness of Asphalt Cement in Old Pavement.

the CMS-2S emulsion while an adjustment from 0 to +0.3% would be selected for an HFE-150 emulsion. For example, the adjustments made for an HFE-150 emulsion would be +0.3% for a hard asphalt (> 80,000 poises) while no adjustment (0%) is made for a soft asphalt (10,000 to 35,000 poises). Note that the adjustments for the CMS-2S emulsion range from +0.5% for a hard asphalt to -0.5% for a soft asphalt.

2) Gradation Adjustment. Measurements of RAP gradations from CIR completed to date resulted in a wide range of values for percent passing the 12.7 mm (1/2 in.), 6.4 mm (1/4 in.), and 2.0 mm (#10) screens. Figure 3.2 indicates the range of values when sampling is performed with a 41 cm (16 in.) mill and the expected RAP gradation when using a 3.8 m (150 in.) mill. By using this graph, a maximum adjustment of ±0.3% is made to the base design emulsion content. RAP with a coarse gradation would result in adding an adjustment for the CMS-2S emulsion of up to 0.3% to the base design emulsion content while up to 0.3% would be subtracted for RAP with a fine gradation. It was initially felt that the mixes treated with CMS-2S behaved like an open-graded mix; hence thicker films would be permitted with coarser gradations. Further, whenever fine RAP materials were used, early failures (e.g., rutting) were noted. This could be due to the type of cutter (e.g., naphtha) used in the CMS-2S emulsion.

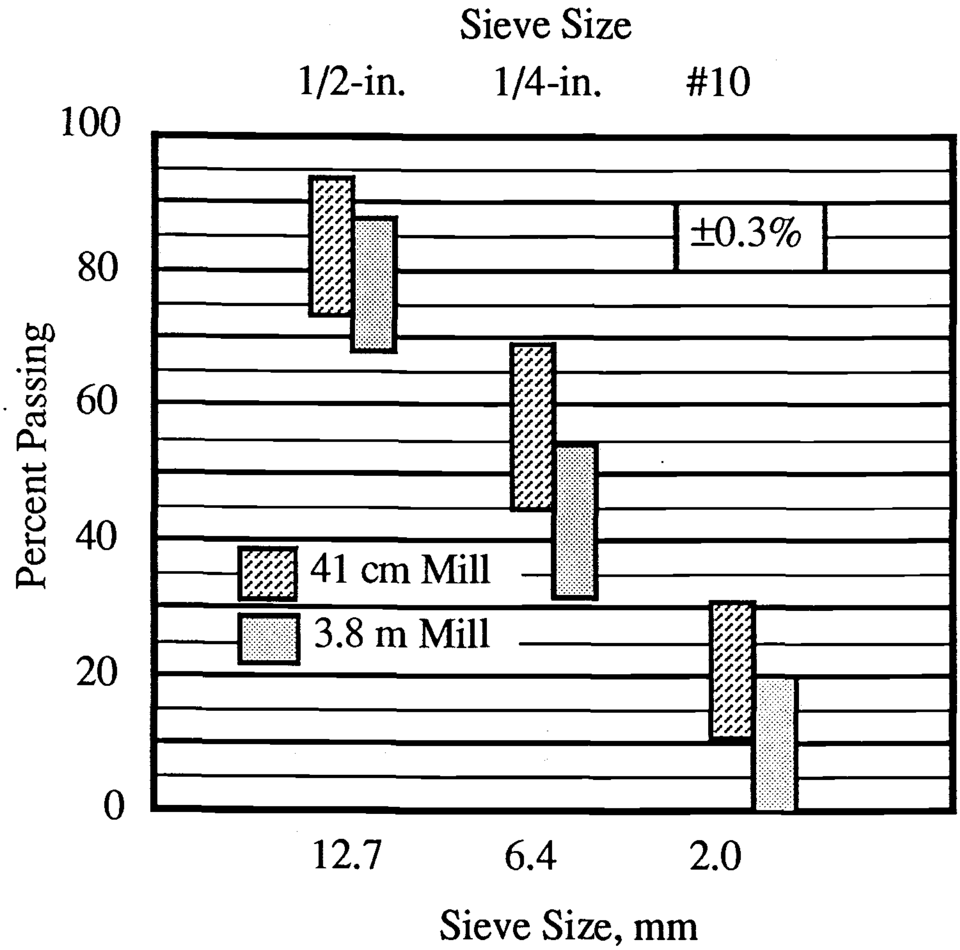


Figure 3.2. Range of Milling (RAP) Gradations from the 41 cm (16 in.) Mill and the 3.8 m (150 in.) Mill on the Recycling Train.

For the 1988 projects (using HFE-150), RAP with a coarse gradation would result in adding an adjustment of up to 0.3% while up to 0.3% would be subtracted to the base design emulsion content for RAP with a fine gradation. Findings to date indicate that if a RAP gradation is fine on the 12.7 mm (1/2 in.) screen, it will also indicate a fine gradation on the 6.4 mm (1/4 in.) and 2.0 mm (#10) screens. The same holds true for a coarse or average gradation (4,6).

3) Asphalt Adjustment. The percent of asphalt recovered from the RAP (ASTM D1856-Recovery of Asphalt from Solution by Abson Method) was plotted giving the expected range of asphalt content. Figure 3.3 shows this range as well as the adjustment range of ± 0.3% for the CMS-2S emulsion and from 0 to -0.3% for the HFE-150 emulsion. RAP with a high residual asphalt content would result in subtracting up to 0.3% (for both the CMS-2S and the HFE-150 emulsions) from the base design emulsion content while up to 0.3% CMS-2S (0% HFE-150) would be added for RAP with a low residual asphalt content.

The final estimated design emulsion content is determined as shown in Table 3.6.

Estimating Total Fluid Content. Once the final estimated design emulsion content is determined, it is necessary to estimate the total fluids (water and emulsion) content. This is accomplished with the modified Oregon State Highway Division test method OSHD TM-126 (CTB

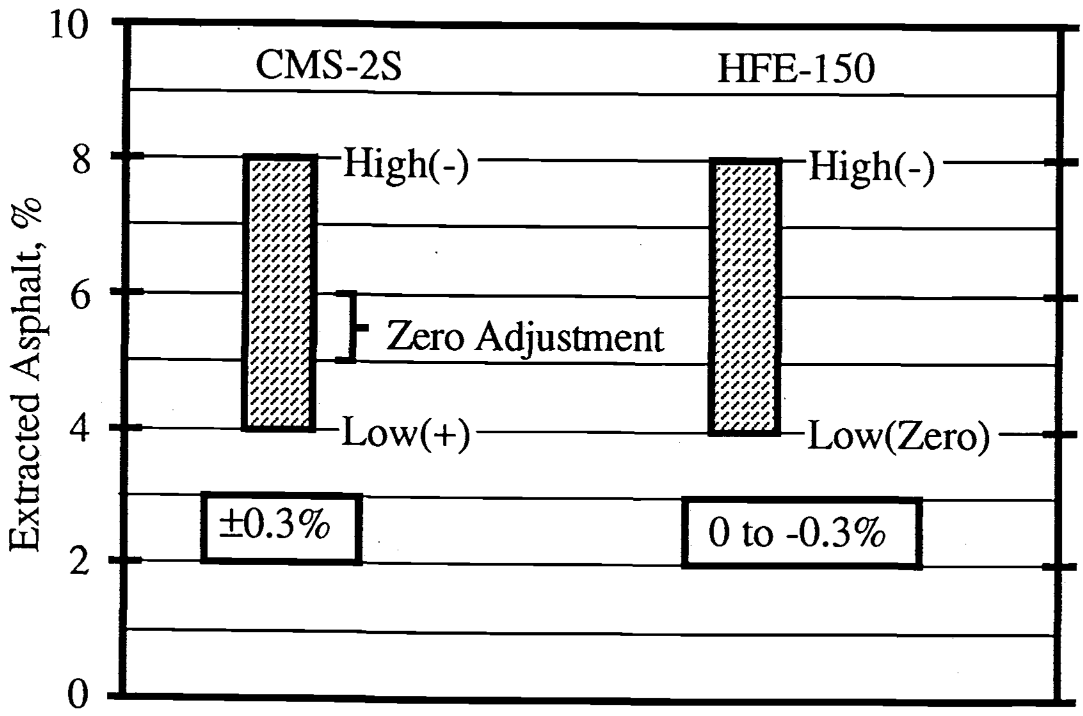


Figure 3.3. Adjustment for Residual Asphalt Cement Content.

test) (4). The objective of the test is to determine the amount of water (total fluids content minus emulsion content) required for the mix. The test may also be used in the field to spot-check the mix for total fluids content. Briefly, the modified CTB test is conducted as follows:

1) Samples are prepared at the final estimated design emulsion content and at incremental water contents (e.g., 0.5, 1.0, 1.5%) and each sample weight is recorded.
2) Each sample is placed and rodded in a split mold in two lifts.
3) Each sample is gradually compressed to a total load of 111 kN (25 kips)—one minute to achieve 89 kN (20 kips) plus one half minute to achieve the additional 22 kN (5 kips). The 111 kN (25 kip) load is held for one minute.
4) The specimen weights are then determined. The difference between the initial sample weight and the weight of the compacted specimen is the liquid loss.

The total fluids content that results in a liquid loss of 0 to 4 ml (0 to 4 grams) is used as the design total fluids content. From this the water content can be calculated (total fluids content minus emulsion content). It should be pointed out that this test is valid for determining total fluids and cannot directly determine the water content (i.e., the water must be calculated).

Final Design. Up to the present time, the emulsion and water contents determined above were used in construction. It is proposed, however, that for future projects the final emulsion content be determined from tests on samples prepared at the final estimated design emulsion content (determined above) and at the final estimated content ± 0.4%. Figure 3.4 summarizes the steps to select a final design emulsion content where CIR pavement will become part of the structural design to upgrade the surface. The samples should be prepared using either the Hveem (ASTM D1561–Preparation of Bituminous Mixture Test Specimens by Means of California Kneading Compactor) or Marshall (ASTM D1559–Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus) compaction method. Table 3.7 gives a suggested sample preparation procedure for either method. Once compacted and cured as prescribed, the samples are tested for stability, resilient modulus, and fatigue. Suggested criteria (as of 1988) for selection of the final emulsion content are given in Table 3.8. A detailed mix design procedure for CIR mixtures is given in Appendix A.

Field Adjustments. In addition to the calculated adjustments to the base design emulsion content (described above) field adjustments are made to the final estimated design emulsion content. These include adjustments for (1) differences in RAP gradation, (2) isolated fat spots or unstable mixtures, and (3) visual appearance of the mat 2 to 3 hours after rolling. The field adjustments to the final estimated design emulsion content are described in detail below:

1) RAP Gradation. RAP gradation is checked frequently during construction. If differences from the RAP gradation used to estimate the emulsion content occur, the emulsion content is changed as described above (i.e., using Figure 3.2).

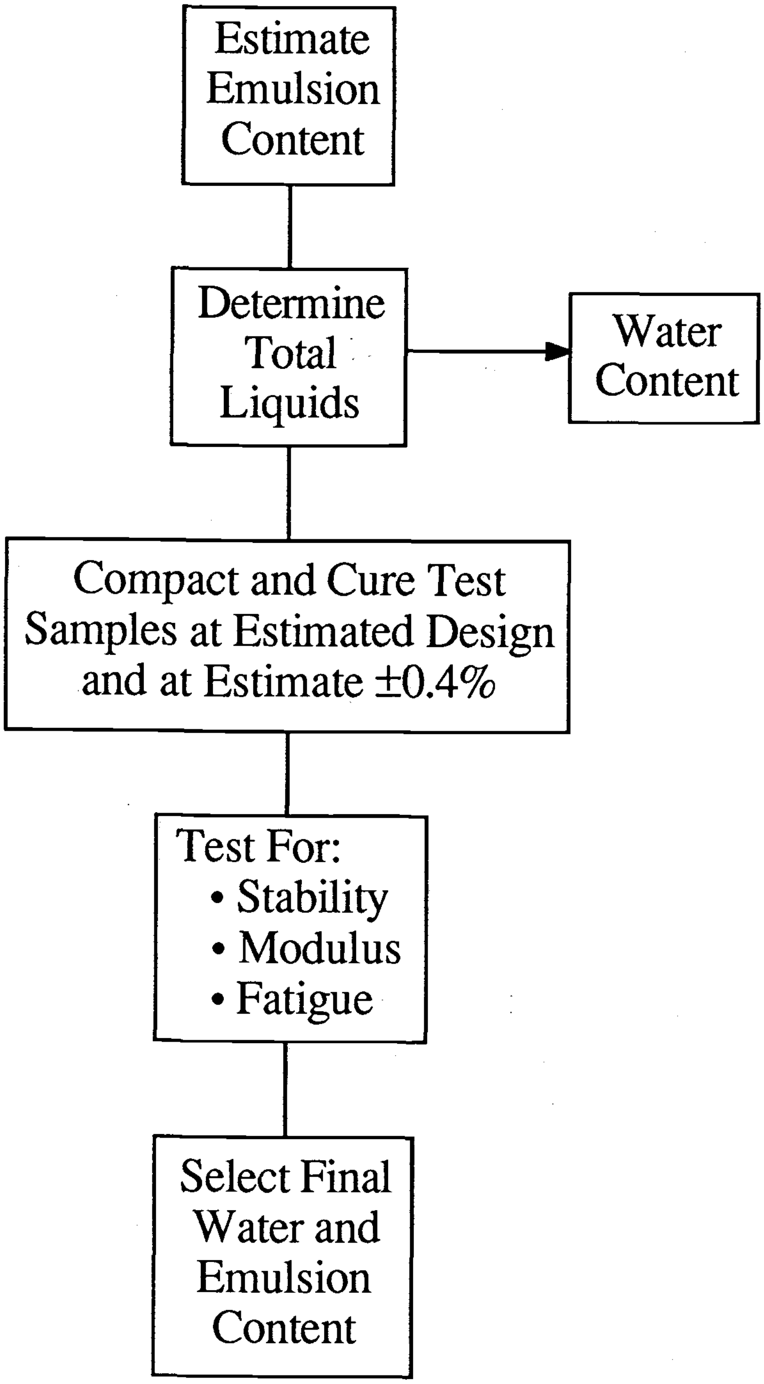


Figure 3.4. Proposed Mix Design Process – Future Projects.

Table 3.7. Suggested Sample Preparation Procedure for CIR.

1) Millings are split into approximately 5500 g batches; this size sample provides sufficient material for four 6.4 cm (2.5 in.) high specimens with an 1100 gm sample for moisture determination.

2) Sample is screened on the 2.5 cm (1 in.) sieve. The material retained on the 2.5 cm sieve is reduced in size to 100% passing 2.5 cm sieve using 13.4 N (3 lb) hammer. This is because the retained 2.5 cm is too large for 10.2 cm (4 in.) molds.

3) Batch five 1100 gm samples of millings at the average gradation.

4) Determine moisture content of one batch by drying 24 hrs at 100°C (230°F).

5) Samples are heated to 60°C (140°F) ± prior to mixing (1-2 hrs).

6) Water is added to the millings in the appropriate proportion based on the dry weight of the millings: % water = 4.5 total liquid - % added emulsion. Water is thoroughly mixed into millings by hand.

7) Water is added to the premoistened millings after water addition using the recommended content. The added emulsion is based upon the dry weight of the millings. The emulsion is preheated to 60°C (140°F) ± (1 hr) and mixed thoroughly into the batch by hand or using a mechanized mixer.

8) The material is spread into a 30.5 cm x 41.2 cm (12 in. x 17 in.) baking pan and allowed to cure for 1 hr at 60°C (140°F) ± to simulate average time elapsed between paver laydown and initial compaction during actual construction.

9) Samples are molded using standard Marshall or Hveem procedures to produce 6.4 cm (2.5 in.) ± high briquets as described below:

   a) Molds are preheated to 60°C (140°F) ±.
   b) Compact samples using standard 50-blow compactive effort for Marshall procedure or 150 blows at 3.1 MPa (450 psi) for the Hveem procedure.
   c) Cure overnight at 60°C (140°F) and recompact using 25 blows per side for the Marshall procedure and 75 blows at 3.1 MPa (450 psi) for the Hveem procedure.
   d) The molds are laid on their side and the briquets are cured for 24 hrs at 60°C (140°F) ± prior to extrusion.
   e) Briquets are extruded with the compression testing machine.
   f) Briquets are laid on their side to maximize surface exposure and cured for 72 hrs at ± room temperature prior to testing.

10) Specimens are tested for stability, modulus, and fatigue at 25°C (77°F).

Table 3.8. Suggested Mix Design Criteria (1987-88).*

Property	Recommended Value
Hveem Stability	> 10 after 2nd comp.
Resilient Modulus @ 25°C (77°F)	1.0-2.1 GPa (150-300 ksi)
Modulus ratio @ 25°C (77°F) after saturation	> 0.6
Fatigue Life @ 100 $\mu\epsilon$ @ 25°C (77°F)	> 5,000

*Marshall Stability criteria are currently being developed.

2) Isolated Fat Spots and Unstable Mixes. Isolated fat spots and unstable mixes are noted ahead of the mill. The emulsion content is dropped 0.2% in areas that appear slightly fat and dropped 0.4% in areas that are obviously unstable and rutted. These adjustments are made only if field samples were not taken at the exact locations of the distress.

3) Visual Appearance. Minor adjustments of ±0.1% to ±0.2% are made by visual appearance of the mat 2 to 3 hours after initial compaction. Additional emulsion is added (up to +0.2%) if the mat remains brown and is prone to raveling. On the other extreme, the emulsion content is reduced 0.2% if the mat is very black and shiny and no raveling is apparent.

3.4 Expected Ranges in Strength Properties

Mix property tests were performed on cores taken from the 1986 projects as part of the ODOT/OSU study (7). These tests included:

1) Resilient modulus (ASTM D4123) and fatigue at 23°C (73°F), at an initial tensile strain of 100 microstrain (100×10^{-6}), at a load duration of 0.1 second, and at a dynamic loading frequency of 1 Hertz.
2) Marshall stability and flow (ASTM D1559) at 60°C (140°F) and at a static loading rate of 5 cm/min. (2 in./min.).

Six 10 cm (4 in.) diameter field cores were extracted from seven of the thirteen 1986 projects in the fall of 1986 (3 months after construction) and in the fall of 1987 (15 months after construction). Three cores from each project were used for the modulus and fatigue tests while the remaining three cores from each project were used for the Marshall stability tests. The results of these tests as well as expected ranges in these properties are discussed in the following paragraphs.

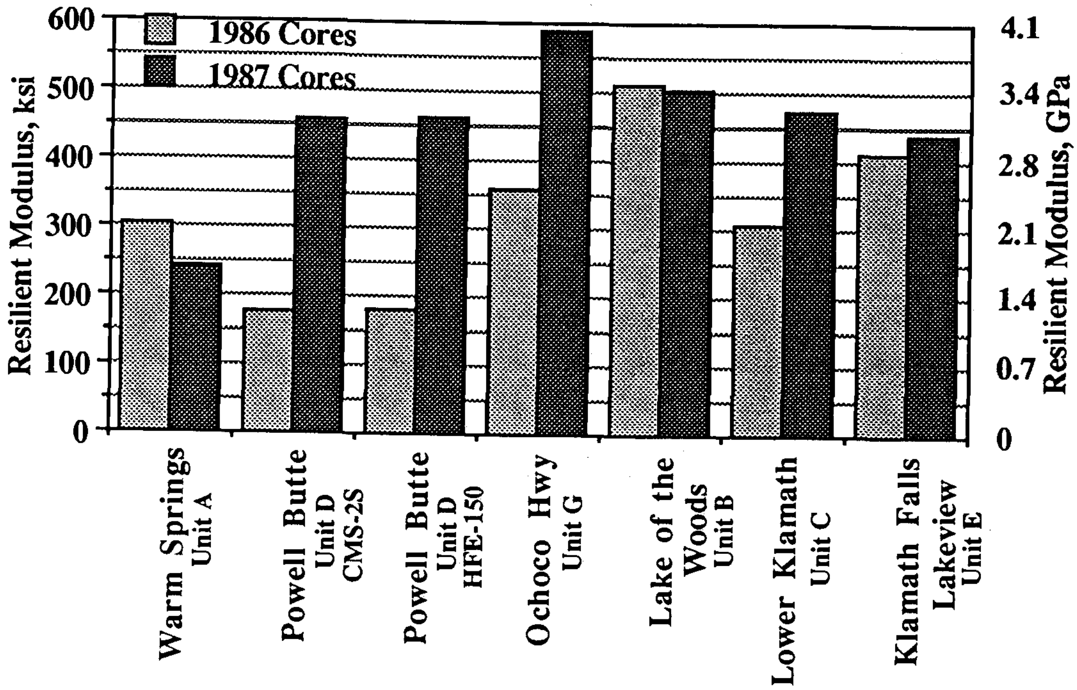
3.4.1 Modulus and Fatigue

The results of the resilient modulus and fatigue life tests are summarized in Table 3.9 while Figure 3.5 summarizes these results graphically. All results represent the average of the three cores from each project. As indicated, the modulus values ranged from 1.2 to 3.5 GPa (175 to 513 ksi) in 1986 and from 1.7 to 4.0 GPa (242 to 587 ksi) in 1987. Fatigue lives ranged from about 5,900 to 33,000 in 1986 and from about 32,000 to 72,000 in 1987. The moduli increased appreciably

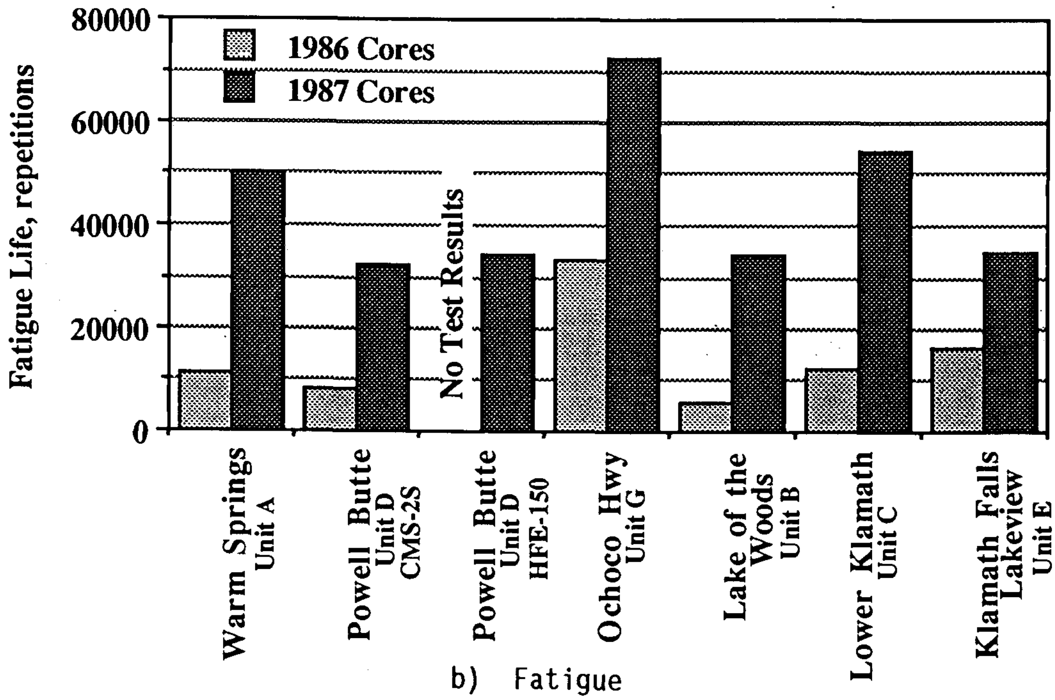
Table 3.9. Summary of Modulus and Fatigue Test Results (Selected 1986 Projects).

Project	Highway	Avg. Resilient Modulus, (GPa)/(ksi)		Avg. Fatigue Life	
		Fall 1986	Fall 1987	Fall 1986	Fall 1987
MP 79.2-Wasco Co. (Unit A)	Warm Springs	2.1/305	1.7/242	11030	50010
Powell Butte-Prineville, CMS-2S (Unit D)	Ochoco	1.2/175	3.2/458	8110	32325
Powell Butte-Prineville, HFE-150 (Unit D)	Ochoco	1.2/180	3.2/461	N/A†	34281
MP 89.6-Jct. OR 19 (Unit G)	Ochoco	2.5/357	4.0/587	33250	72472
Lakeshore Dr.-Greensprings Jct. (Unit B)	Lake of the Woods	3.5/513	3.5/504	5860	34261
US 97-OR 39 (Unit C)	Lower Klamath	2.1/309	3.2/472	12500	54326
Sprague River Rd (Unit E)	Klamath Falls-Lakeview	2.8/411	3.0/439	16240	34825

†N/A = Not Available



a) Modulus



b) Fatigue

Figure 3.5. Resilient Modulus and Fatigue Test Results for Selected 1986 Projects.

or remained about the same over time while, in all cases, the fatigue lives increased significantly (Figure 3.5).

It should be noted that for the Ochoco Highway project (MP 89.6-Jct. OR 19), data were included to indicate the extremes, to date, in the expected ranges. Excluding these data would result in the following expected ranges for the remaining six of the seven projects:

1) Modulus - 1.4 to 3.4 GPa (200 to 500 ksi) at 3 months after construction and 1.7 to 3.4 GPa (250 to 500 ksi) at 15 months.
2) Fatigue - 6,000 to 16,000 load repetitions at 3 months after construction and 32,000 to 54,000 at 15 months.

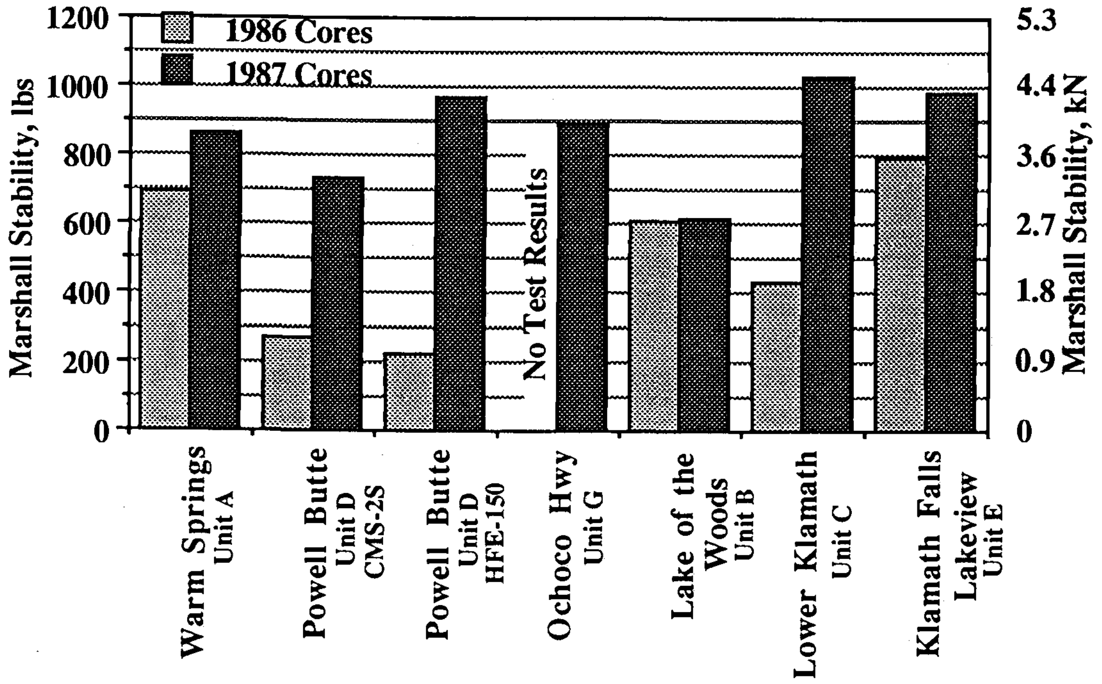
3.4.2 Stability and Flow

The Marshall stability test results are summarized in Table 3.10 while Figure 3.6 displays these results graphically. All results represent the average of the three cores from each project. The results indicate that the stability values range from 1.0 to 3.5 kN (226 to 793 lbs) in 1986 and from 2.7 to 4.6 kN (614 to 1032 lbs) in 1987. Flow values ranged from 0.4 to 1.5 mm (17 to 59 mils) in 1986 and from 0.4 to 0.5 mm (16 to 20 mils) in 1987. Figure 3.6 shows that, as expected, the stabilities generally increased over time while the flow values generally decreased over time. Note that the stability test results support the modulus test results. That is, when Figure 3.5a is compared with Figure 3.6a, the same trends (strength vs. time) are apparent.

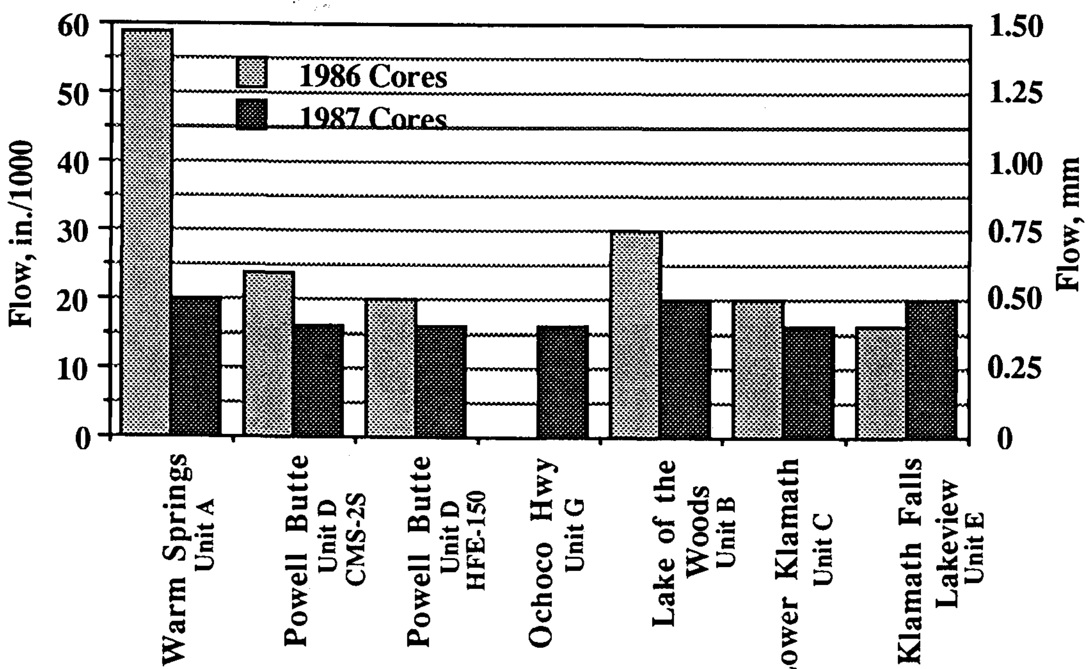
Table 3.10. Summary of Marshall Stability Test Results (Selected 1986 Projects)

Project	Highway	Avg. Stability (kN)/(lbs)		Avg. Flow (mm)/(in.)	
		Fall 1986	Fall 1987	Fall 1986	Fall 1987
MP 79.2-Wasco Co. (Unit A)	Warm Springs	3.1/694	3.8/861	1.5/0.059	0.5/0.020
Powell Butte-Prineville, CMS-2S (Unit D)	Ochoco	1.2/269	3.2/731	0.6/0.025	0.4/0.018
Powell Butte-Prineville, HFE-150 (Unit D)	Ochoco	1.0/226	4.3/968	0.5/0.020	0.4/0.017
MP 89.6-Jct. OR 19 (Unit G)	Ochoco	N/A†	4.0/890	N/A†	0.4/0.017
Lakeshore Dr.-Greensprings Jct. (Unit B)	Lake of the Woods	2.7/605	2.7/614	0.7/0.029	0.5/0.020
US 97-OR 39 (Unit C)	Lower Klamath	1.9/427	4.6/1032	0.5/0.017	0.4/0.016
Sprague River Rd (Unit E)	Klamath Falls-Lakeview	3.5/793	4.4/982	0.4/0.016	0.5/0.020

†N/A = Not Available



a) Stability



b) Flow

Figure 3.6. Marshall Stability Test Results for Selected 1986 Projects.

3.5 Conclusions and Recommendations

3.5.1 Conclusions

The following conclusions appear to be warranted as a result of the contents of this paper:

1) Class I or Class II cold in-place recycling using either the recycling train or the single-unit machine can produce a treatment suitable for a wearing or base course (the surface is usually sealed if the treatment is used as a wearing course).
2) Cold recycled mixtures are quite variable necessitating a simple procedure for estimating emulsion and water content.
3) The improved mix design procedure (1987-88) accurately estimates (within 0.2%) the design emulsion content for starting CIR projects.
4) The modified CTB test can be used to estimate the total fluids content in design and during construction.
5) Current mix design criteria (1988) need to be verified for confirming the estimated design emulsion content.
6) During the period 1984-88, 52 projects were completed. Pavement ratings in 1988 showed that 48 (92%) were performing very well and 4 (8%) had some unstable/rutted areas that required repairs.
7) Based on the data to date, the structural contribution of properly designed and constructed CIR mixtures is nearly

equivalent to that of conventional asphalt concrete mixtures.

3.5.2 Recommendations

The following recommendations for further study of cold in-place recycling in Oregon include:

1) Confirm the mix design criteria (1988) by preparing and testing laboratory samples as prescribed by the mix design procedure.
2) Continue to monitor the performance of the CIR projects constructed to date to obtain and document long-term field performance of the cold in-place recycled pavements.
3) Continue to test field cores from the CIR projects to track mix properties of cold in-place recycled pavements over time.

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4.0 REPEATABILITY OF TESTING PROCEDURES FOR RESILIENT MODULUS AND FATIGUE

by

Todd Scholz[1], R. Gary Hicks[2], and Lewis Scholl[3]

ABSTRACT

Extensive use of diametral resilient modulus and fatigue testing is made by the Oregon State Highway Division to evaluate asphaltic concrete materials. Test results on similar materials (e.g., adjacent field cores), however, often indicated a poor level of repeatability.

This report examines the repeatability of the diametral resilient modulus test at two laboratories — Oregon State Highway Division (OSHD) and Oregon State University (OSU) — and the repeatability of the fatigue test at a single laboratory (OSU). Modulus tests at the OSHD laboratory were conducted at one temperature and three strain levels while modulus tests at the OSU laboratory were conducted at two temperatures and the same three strain levels. Fatigue testing at the OSU laboratory was conducted at two temperatures and at one initial tensile strain level.

As a result of the test program involved in this study, the significant findings include:

1. Modulus tests are highly repeatable within each laboratory.

[1]Research Assistant, Department of Civil Engineering, Oregon State University, Corvallis, OR 97331

[2]Professor of Civil Engineering, Oregon State University, Corvallis, OR 97331

[3]Technical Studies Coordinator, Oregon State Highway Division, Salem, OR 97310

2. Modulus tests under similar test conditions could not satisfactorily reproduced (at the 5% significance level) between laboratories.
3. Depending on test conditions, fatigue tests had a very low to very high level of repeatability.

4.1 Introduction

4.1.1 Problem Statement

The Oregon State Highway Division (OSHD) currently makes extensive use of diametral resilient modulus and fatigue testing to evaluate the relative expected performance of asphalt concrete materials used for the wearing course on Oregon's state highways. Two recent studies, one on cold in-place recycling (CIR) and the other an on-going study of pavements constructed with various asphalt additives, have utilized the results of these tests as an integral part of the studies (1,2).

Resilient modulus and fatigue tests for the CIR study were primarily conducted at Oregon State University (OSU). In this study testing was performed on both field cores and laboratory prepared samples. Testing for the asphalt additives study was conducted at both the OSU and OSHD laboratories. In this latter study, these tests were performed exclusively on field cores. Test results, however, often indicated a poor level of repeatability between laboratory samples prepared from the same mix design (CIR study) and between adjacent field cores (both studies). This was particularly true of the additives study where significant differences between the two laboratories were often observed.

Another current study involving the investigation of polymer-modified asphalt concrete mixes will make use of the resilient modulus and fatigue tests to aid in establishing test procedures and specifications for these mixes (3). Because of the central role these mix property tests will play in this study, it is important to establish the repeatability (precision) of the diametral resilient modulus and fatigue tests.

4.1.2 Purpose

OSHD initiated this study to determine the repeatability of diametral resilient modulus and fatigue tests. More specifically, this study will provide answers for the following questions:

1. How repeatable (precise) within a single laboratory, measured in coefficient of variation, are tests for resilient modulus and fatigue?
2. Can the diametral resilient modulus results of the OSHD laboratory be reproduced by the OSU laboratory at the 5% significance level?

This study involved the testing of laboratory-fabricated samples to determine appropriate statistical parameters (e.g., standard deviation, coefficient of variation, etc.) of the test results when as much variability as possible was removed from the sample preparation and test procedures. Figure 4.1 shows the overall study approach.

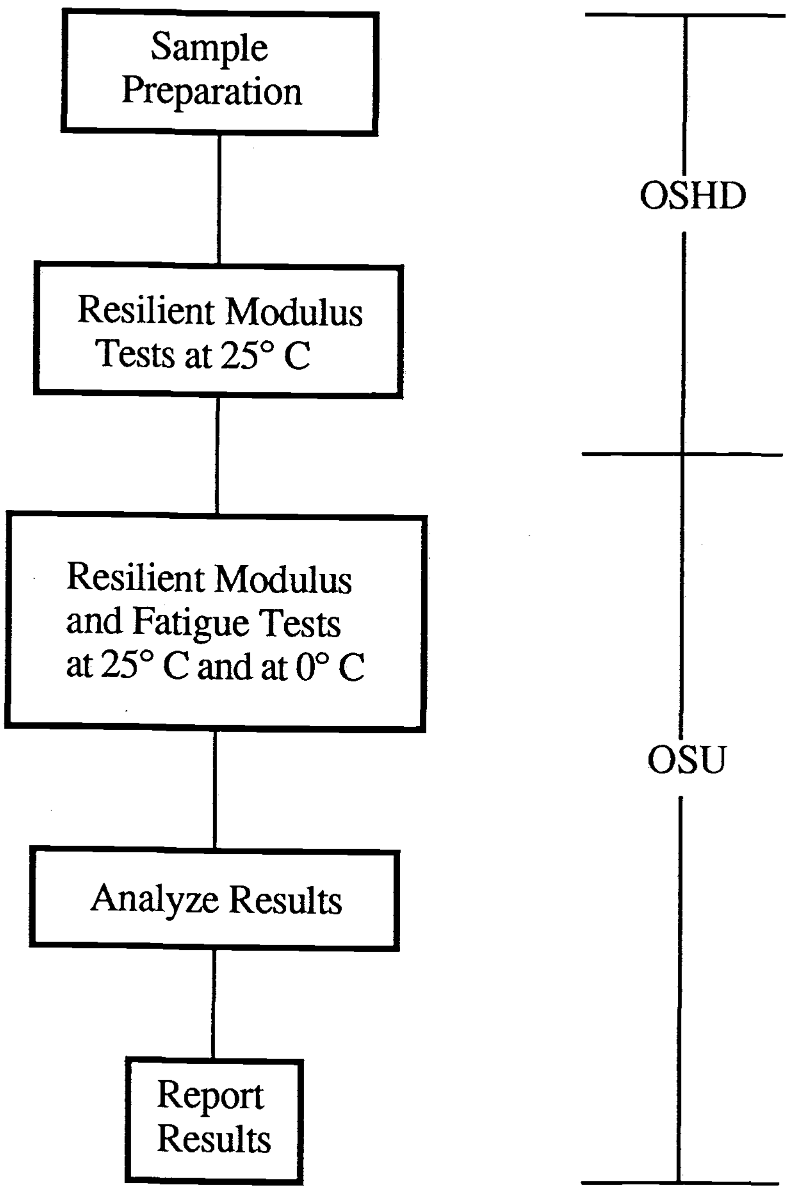


Figure 4.1 - Flowchart Showing the Scope of This Study.

4.2 Laboratory Study

4.2.1 Description of Laboratory Study

The laboratory study involved the fabrication and testing of two sets of samples prepared with the same mix design but having different asphalt types. The original test schedule for the laboratory study is shown in Table 4.1. A total of 48 samples were prepared by the OSHD Materials laboratory, 24 with an AC20 grade of asphalt cement and 24 with CA(P)-1 (a polymer-modified asphalt cement). Both mixes contained 1% lime. The specimens were compacted in accordance with ASTM D1561 (4) and having the job-mix formula shown in Table 4.2.

After fabrication, OSHD tested 24 of the samples (12 of each asphalt type) for resilient modulus at 25°C and at three strain levels (50, 100, and 150 microstrain). All 48 samples were submitted to OSU for resilient modulus and fatigue testing. OSU conducted modulus tests using ASTM D4123 (4) and fatigue tests (Appendix B) at 25°C on the 24 samples tested by OSHD. Analysis of the modulus results for the CA(P)-1 (polymer modified) material, however, indicated a significant difference at the 5% significance level between the OSHD and OSU laboratories. The results for the AC20 samples also showed some difference between the laboratories but the difference was not found to be significant at the 5% level.

It was reasoned that the significant difference between laboratories in the modulus results for the CA(P)-1 samples may have been due to the effect of aging. That is, OSHD conducted the warm temperature (25°C) modulus tests soon after the samples were fabricated while

Table 4.1 – Original Test Schedule for the Laboratory Study.

Laboratory	Mix Type	Modulus[1]		Fatigue[2]	
		@ 25°C	@ 0°C	@ 25°C	@ 0°C
OSHD	AC20	12	–	–	–
	CA(P)-1	12	–	–	–
OSU	AC20	12	12	12	12
	CA(P)-1	12	12	12	12

[1]Tests performed at three strain levels.

[2]Tests performed at 200 microstrain (initial).

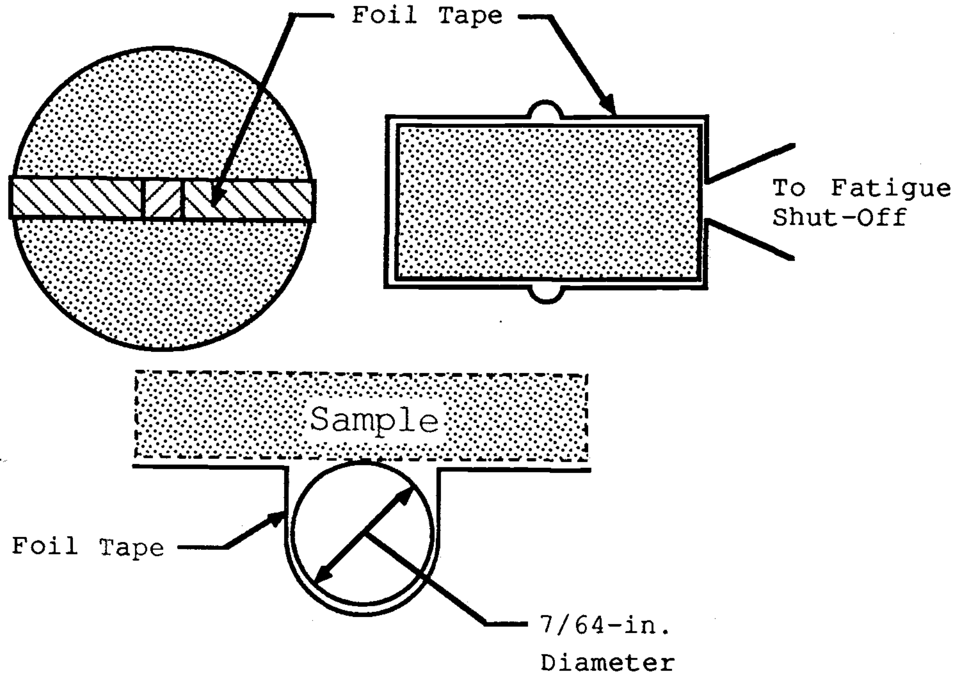
Table 4.2 – Job-Mix Formula for the Laboratory-Fabricated Test Specimens.

Sieve Size	Percent Passing	Oil Content (%)	Lime Content (%)
3/4	100.0	5	1
1/2	86.3		
3/8	73.5	(24 specimens with AC20 and 24 specimens with CA(P)-1)	(all 48 specimens)
1/4	59.0		
# 4	49.3		
#10	30.0		
#40	12.2		
#200	3.5		

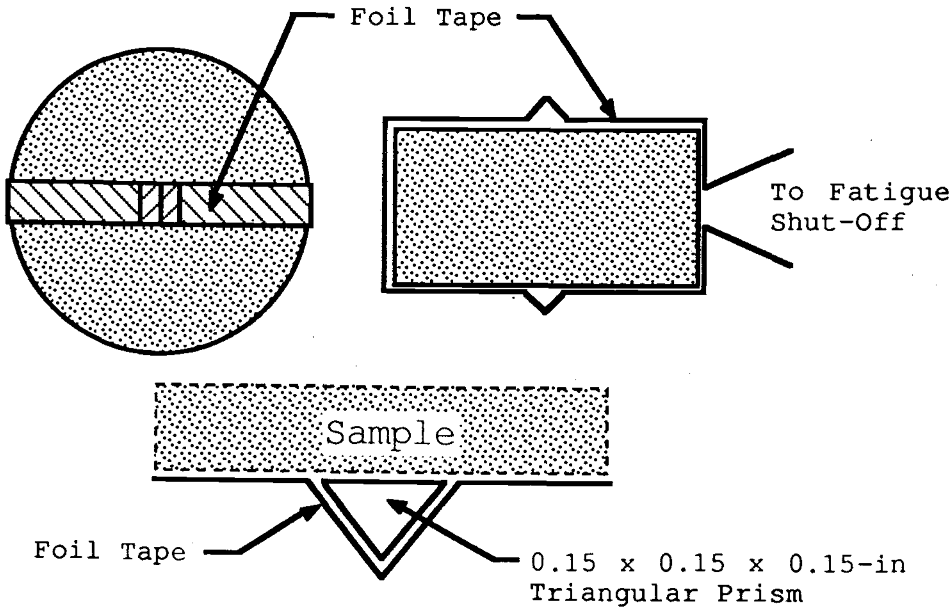
these tests were performed at OSU several weeks later. To further investigate the possibility of aging, the 12 CA(P)-1 samples scheduled for the low temperature (0°C) tests were returned to OSHD for modulus testing at 25°C and at the three strain levels of 50, 100, and 150 microstrain. Upon completion of these tests the samples were resubmitted to OSU to duplicate the tests at 25°C. To minimize any effect of aging, the modulus testing at OSU was performed the day following the modulus testing at OSHD.

Analysis of the fatigue results of tests conducted at 25°C indicated an acceptable coefficient of variation ($CV \leq 15\%$) for the AC20 mix but an unacceptable CV for the CA(P)-1 mix ($CV \approx 41\%$). This poor repeatability prompted an effort to improve the test by modifying the failure criterion. Thus, six of the cold temperature samples of each mix type were tested at the 25°C temperature using a modified failure criterion as illustrated in Figure 4.2. Permanent vertical deformation was also measured during the fatigue test. This was done to allow an alternative definition of failure as shown in Figure 4.3. The remaining six samples of each mix type were tested at the 0°C temperature using the modified failure criterion with measurement of permanent vertical deformation. Table 4.3 summarizes the revised test schedule as a result of the above changes to the original test schedule.

It should be noted that low temperature (0°C) modulus tests and fatigue tests (at either temperature) were not conducted by the OSHD laboratory since, at present, OSHD does not have low temperature (below ambient) or fatigue testing capabilities.



a) Original Failure Criterion



b) Modified Failure Criterion

Figure 4.2 – Original and Modified Failure Criteria for Fatigue.

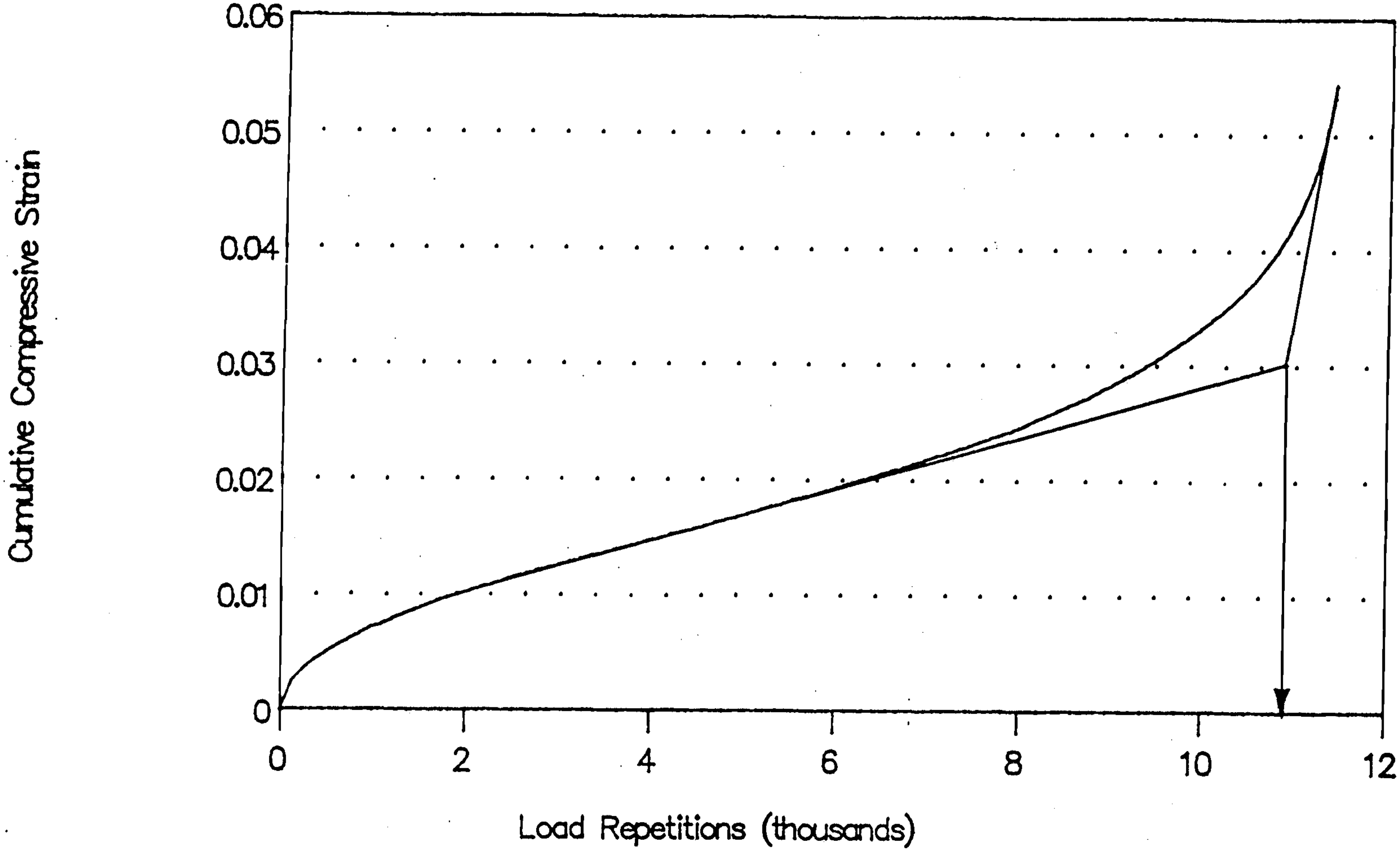


Figure 4.3 – Failure Criterion as Determined by Measurement of Permanent Deformation.

Table 4.3 – Revised Test Schedule for the Laboratory Study.

Laboratory	Mix Type	Modulus[1]		Fatigue Only[2]		Fatigue and Permanent Deformation[2]	
		@ 25°C	@ 0°C	@ 25°C	@ 0°C	@ 25°C	@ 0°C
OSHD	AC20	12	–	–	–	–	–
	CA(P)-1	24	–	–	–	–	–
OSU	AC20	18	6	12	–	6	6
	CA(P)-1	24	6	12	–	6	6

[1]Tests performed at three strain levels.

[2]Tests performed at 200 microstrain (initial).

4.2.2 Test Equipment and Procedures

Since one of the purposes of this study was to determine the repeatability of modulus tests between laboratories, it is worthy to note the following differences in modulus test equipment and procedures between the OSHD and OSU laboratories.

Test Equipment Differences (OSU vs. OSHD). The significant differences in test equipment between the OSHD and OSU laboratories include the following:

1. The OSHD test system records changes in load and horizontal deformation via analog signal and a microprocessor while the OSU test system records these changes via analog signal and a strip chart recorder.
2. The OSHD test system applies a load pulse of variable magnitude for a duration of 0.25 second while the OSU test system applies a load pulse which closely approximates a square wave having a 0.1 second duration.
3. While both test systems can test 4-in. diameter samples, the OSHD test system utilizes load strip widths of 3/4 and 1-in. while the OSU test system uses two 1/2-in. load strips. ASTM D4123 specifies the use of a 1/2-in. load strip when testing 4-in. diameter samples (4).

Test Procedure Differences (OSU vs. OSHD). The significant differences in resilient modulus test procedures between the OSHD and OSU laboratories include the following:

1. The OSU test procedure accounts for early plastic flow (through conditioning) while this plastic flow is not accounted for in the OSHD test procedure (no conditioning).
2. The test specimen is removed from a constant temperature environment and tested in an environment which may have a different temperature in the OSHD test procedure while the test specimen remains in the same environment in the OSU test procedure.

These differences are mentioned since variability in test results between laboratories may be influenced by differences in test equipment and procedures. Due to this possibility, further discussion of each of these differences is warranted as follows.

Recording System. The OSU test system records load and deformation via analog signal and a strip chart recorder which allows direct monitoring of the response of test specimens under loading. The OSHD test system records load and deformation via analog signal and a microprocessor which does not allow direct monitoring of the test specimen response (e.g., the shape of the load pulse and deformation trace cannot be monitored). This becomes very important when the test specimen is not responding appropriately which can significantly influence its modulus value. For example, slight ringing or shimmying of the test equipment can produce an inappropriate deformation waveform resulting in an incorrect modulus value. Such ringing cannot be directly monitored with the OSHD test system. Having a strip chart recorder, therefore, allows a manual check of the results.

Test Systems. Three test systems were employed in this study to apply repeated loads to the test specimens during modulus and fatigue testing. The OSHD test system (used exclusively for modulus testing) is an electro-pneumatic device that delivers a load pulse of variable magnitude for a duration of 0.25 seconds. That is, the load is continuously increased until the desired amount of horizontal deformation is induced in the test specimen at which time the load remains constant until the 0.25 second load duration has elapsed. The resultant load and deformation waveforms have the appearance similar to that of a saw tooth. A rest period of 3 seconds occurs between load pulses.

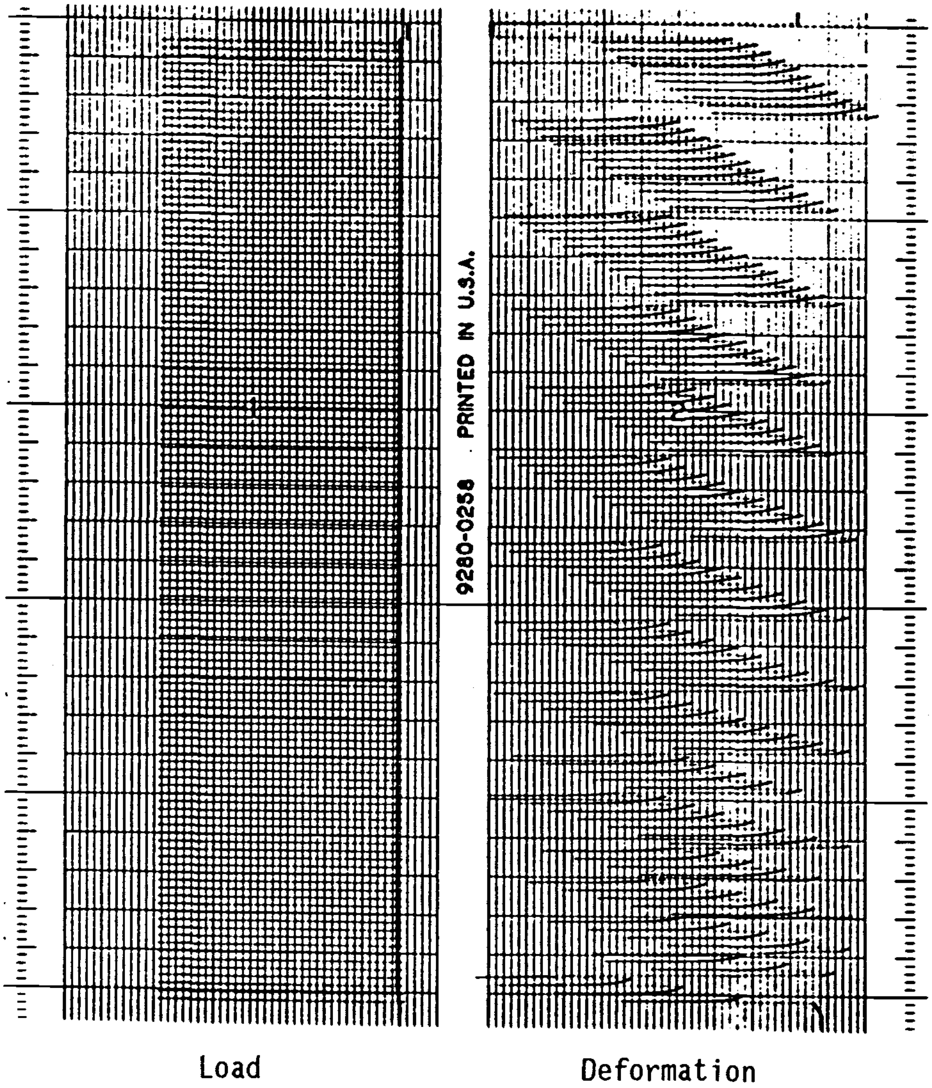
OSU, on the other hand, employed two test systems both of which are different from the OSHD test system. For the warm temperature (25°C) tests, OSU employed an electro-pneumatic device that delivers a load pulse which closely approximates a square wave having a duration of 0.1 seconds. That is, the magnitude of the load pulse remains essentially constant for the 0.1 second duration of time. The time between load pulses is adjustable to accommodate a variety of rest periods. The second test system employed by OSU was an electro-hydraulic device. This test system was employed for the cold temperature (0°C) modulus and fatigue tests due to the high loads (>2500 lbs.) required to induce the desired amount of horizontal deformation. The load waveform produced by this device was that of a haversine having a frequency of 1 Hertz. By using the haversine load pulse, the test specimen continuously experienced a compressive load of variable magnitude without a rest period.

Load Strip Widths. The OSHD test system uses two load strips having different widths (3/4 and 1-in.) while the OSU test system uses two load strips having the same width (1/2-in.). The load strip width has a direct effect on the specimen modulus since different widths produce different stress profiles in the test specimen. This is particularly true of the regions in the specimen very near the load strips.

Test Specimen Conditioning. Observations of the load and deformation waveforms of specimens being tested in repeated load indirect tension (resilient modulus) indicate that the modulus increases with increasing load repetition up to between 50 and 100 repetitions. The modulus then becomes essentially constant until the specimen nears failure at which time the modulus decreases appreciably. This initial increase in modulus can be evidenced when a specimen is tested under a repeated load of constant magnitude as shown in Figure 4.4. Note that the tensile strain decreases with increasing number of repetitions resulting in the indicated increase in modulus.

This initial increase in modulus (due to early plastic flow) is not accounted for in the OSHD test procedure. That is, in the OSHD test procedure the average of the first ten load repetitions and corresponding deformations is used to determine the test specimen modulus. In the OSU test procedure, however, the test specimen is conditioned under repeated load for 50-100 repetitions before the modulus is measured, thus accounting for initial plastic flow.

Test Specimen Temperature. Due to the temperature susceptibility of asphalt cement and, thus, asphalt concrete mixes, it is important to



Load Application Number	Load (lb)	Horizontal Deformation (μ-in.)	Resilient Modulus (ksi)
1	323	482	166
10	323	465	172
20	323	449	178
30	323	437	183
40	323	441	181
50	323	437	183
60	323	433	184
70	323	433	184
80	323	429	186
90	323	429	186
100	323	425	188

Figure 4.4 - Typical Strip Chart for the Diametral Resilient Modulus of an Asphalt Concrete Specimen.

maintain constant temperature environments while testing these materials. Under the OSHD test procedure, the test specimen is removed from a constant temperature airbath and tested in the ambient temperature of the laboratory (which can be several degrees Centigrade different from the airbath temperature). Under the OSU test procedure, the test specimen remains in the constant temperature airbath (±0.5 °C) throughout the test. This is not entirely true of the OSU low temperature (0°C) tests since the environmental cabinet varied in temperature between -4 and 1°C.

While there is clearly a need to improve the temperature control at the OSHD laboratory, the lack of an environmental cabinet does not explain the constant difference between laboratories in resilient modulus results (OSU consistently obtained higher modulus values). It would be expected that OSHD would have consistently higher values since the samples are removed from an air bath and tested while the sample is cooling down.

4.2.3 Test Results

The results of resilient modulus tests conducted at 25°C[**] by OSHD are summarized in Tables 4.4 and 4.5. Table 4.4 summarizes the results of tests performed on the 24 samples originally scheduled to be tested at the 25°C temperature. Table 4.5 summarizes the results of tests on the cold temperature CA(P)-1 samples which were tested at the 25°C temperature.

[**]The actual test temperature varied between 20 and 25°C.

Table 4.4 – Summary of OSHD Modulus Test Results for the Warm Temperature (25°C) Samples.

Mix Type	Sample ID	Average Height (in.)	Resilient Modulus* (ksi) @ 25°C and at 50 $\mu\epsilon$	100 $\mu\epsilon$	150 $\mu\epsilon$
AC20	1	2.477	333.79	293.24	240.42
	2	2.496	376.85	263.23	266.69
Voids:	3	2.522	386.22	274.49	266.22
2.7 –	4	2.461	282.22	262.84	271.79
3.5 %	5	2.509	357.08	275.01	301.50
	6	2.513	346.28	341.34	324.12
	7	2.496	281.15	290.79	259.17
	8	2.494	272.24	249.97	248.37
	9	2.505	294.87	253.68	314.24
	10	2.473	318.99	284.57	264.90
	11	2.514	301.45	274.28	256.90
	12	2.480	389.69	333.41	311.12
		Mean:	328.40	283.07	277.12
		Std. Dev.:	42.72	28.72	28.04
		CV (%):	13.00	10.14	10.12
CA(P)-1	21	2.483	276.30	258.53	235.90
	22	2.486	269.61	235.20	207.88
Voids:	23	2.476	302.48	267.87	295.84
2.1 –	24	2.495	289.75	244.89	216.50
3.6 %	25	2.481	205.82	200.32	200.36
	26	2.471	221.07	221.57	229.39
	27	2.467	194.47	203.93	216.63
	28	2.485	207.94	204.32	239.62
	29	2.460	244.26	257.70	210.03
	30	2.519	245.61	226.15	213.40
	31	2.477	247.04	212.54	219.17
	32	2.477	246.74	217.96	228.46
		Mean:	245.92	229.25	226.10
		Std. Dev.:	34.28	23.38	24.87
		CV (%):	13.94	10.20	11.00

* Tests conducted at a dynamic loading frequnecy of 1/3 Hertz, at a dynamic load duration of 0.25 sec, and at the indicated strain level.

Table 4.5 – Summary of OSHD Modulus Test Results for the Cold Temperature CA(P)-1 Samples Which Were Tested at 25°C.

Sample ID	Average Height (in.)	Resilient Modulus* (ksi) @ 25°C and at		
		50 $\mu\epsilon$	100 $\mu\epsilon$	150 $\mu\epsilon$
13	2.46	267.34	239.33	233.28
14	2.49	257.26	243.62	233.46
15	2.47	299.51	250.48	241.65
16	2.48	275.24	263.14	206.10
33	2.46	273.14	276.44	284.44
34	2.49	256.47	243.40	212.51
35	2.46	242.90	230.50	237.55
36	2.49	259.68	253.23	223.89
37	2.47	244.86	238.48	189.76
38	2.47	237.78	222.02	214.95
39	2.47	273.68	245.90	246.84
40	2.47	271.28	254.05	236.20
	Mean:	263.26	244.46	230.05
	Std. Dev.:	17.17	13.68	23.86
	CV (%):	6.52	5.60	10.37

* Tests conducted at a dynamic loading frequency of 1/3 Hertz, at a dynamic load duration of 0.25 sec, and at the indicated tensile strain.

The results of resilient modulus and fatigue tests conducted at 25°C by OSU are summarized in Tables 4.6 through 4.8. Table 4.6 summarizes the results of tests performed on the 24 samples originally scheduled to be tested at 25°C. Table 4.7 summarizes the results of tests on the cold temperature CA(P)-1 samples tested at the 25°C temperature. Note that only six of these samples were tested for fatigue and permanent deformation (using the modified failure criterion). The remaining six samples were tested at the 0°C temperature.

The results of tests conducted on six of the cold temperature AC20 samples tested at the 25°C temperature are summarized in Table 4.8. Note that these samples were tested for modulus at a dynamic loading frequency of 1 Hertz, a dynamic load duration of 0.1 sec, and at strain levels of 50, 100, and 150 microstrain. In addition, these samples were tested for fatigue using the modified failure criterion with measurement of permanent vertical deformation during the fatigue test.

The results of tests conducted at 0°C for both mix types are summarized in Table 4.9. The modulus tests were performed at a dynamic loading frequency of 1 Hertz using a haversine and at strain levels of 50, 100, and 150 microstrain. Fatigue tests were performed using the modified failure criteria with measurement of permanent vertical deformation during the fatigue test. It should be noted that failure as defined by measurement of permanent deformation was not possible for these samples. This was because of the highly erratic deformation curves.

Table 4.6 - Summary of OSU Modulus and Fatigue Tests for the Warm Temperature (25°C) Samples.

Mix Type	Sample ID	Average Height (in.)	Resilient Modulus* (ksi) at 25°C and @			Fatigue Life** at 25°C & 200 $\mu\epsilon$
			50 $\mu\epsilon$	100 $\mu\epsilon$	150 $\mu\epsilon$	
AC20	1	2.477	304.13	279.32	254.36	***
	2	2.496	312.13	310.17	268.10	2128
Voids:	3	2.522	311.91	285.82	268.72	2867
2.7 -	4	2.461	343.64	285.13	258.36	2875
3.5 %	5	2.509	314.80	297.21	267.66	2718
	6	2.513	304.32	286.08	254.91	2386
	7	2.496	281.84	243.82	231.00	2775
	8	2.494	281.62	268.49	240.83	2506
	9	2.505	279.56	283.01	246.02	2442
	10	2.473	321.11	294.50	275.30	2392
	11	2.514	303.43	282.56	249.77	1717
	12	2.480	370.32	337.57	292.10	2314
		Mean:	310.74	287.81	267.26	2465
		Std. Dev.:	26.20	22.47	35.59	346
		CV (%):	8.43	7.81	13.30	14.04
CA(P)-1	21	2.483	317.16	268.53	268.37	9009
	22	2.486	311.73	273.74	273.73	3521
Voids:	23	2.476	334.59	264.62	266.47	19801
2.1 -	24	2.495	298.55	267.26	257.67	8572
3.6 %	25	2.481	266.68	232.65	231.79	10036
	26	2.471	291.94	248.78	241.68	8618
	27	2.467	272.30	230.76	225.85	11268
	28	2.485	281.26	237.34	230.97	7097
	29	2.460	295.84	263.60	247.09	10521
	30	2.519	310.61	269.24	259.24	6300
	31	2.477	300.74	261.15	256.98	9152
	32	2.477	311.13	275.90	261.86	9400
		Mean:	299.38	257.80	251.81	9441
		Std. Dev.:	19.48	16.16	16.01	3865
		CV (%):	6.51	6.27	6.36	40.94

* Tests conducted at a dynamic loading frequency of 1/3 Hertz, at a dynamic load duration of 0.1 sec, and at the indicated strain level.

** Tests conducted at a dynamic loading frequency of 1 Hertz, at a dynamic load duration of 0.1 sec, and at the indicated initial strain level.

*** No test results.

Table 4.7 - Summary of OSU Modulus and Fatigue Tests Results for the Cold Temperature CA(P)-1 Samples Which Were Tested at 25°C.

Mix Type	Sample ID	Average Height (in.)	Resilient Modulus*(ksi) at 25°C and at			Fatigue Life** at 25°C and 200 $\mu\epsilon$	
			50 $\mu\epsilon$	100 $\mu\epsilon$	150 $\mu\epsilon$	Foil Tape	Perm Def
CA(P)-1	13	2.46	375.81	302.40	275.84	***	***
Voids: 2.1 - 3.6 %	14	2.49	312.51	285.30	258.86	***	***
	15	2.47	354.02	335.46	292.79	5549	5062
	16	2.48	320.39	276.76	254.05	5109	4836
	33	2.46	354.17	322.99	292.24	***	***
	34	2.49	343.76	290.80	261.46	4202	3753
	36	2.49	357.60	309.12	281.16	3158	2830
	37	2.47	296.37	286.48	264.52	***	***
	38	2.47	272.43	261.87	247.47	***	***
	39	2.47	326.58	307.74	283.73	4921	4627
	40	2.47	339.42	299.15	292.35	4281	4013
	Mean:		331.38	297.14	273.03	4537	4187
	Std. Dev.		28.88	20.16	15.79	846	830
	CV (%)		8.71	6.78	5.78	18.6	19.8

* Tests conducted at a dynamic loading frequency of 1/3 Hertz, at a dynamic load duration of 0.1 sec, and at the indicated strain level.

** Tested using the modified failure criteria at a dynamic loading frequency of 1 Hertz, at a dynamic load duration of 0.1, and at the indicated initial strain level.

*** Tested at 0°C.

Table 4.8 – Summary of OSU Modulus and Fatigue Tests Results for the Cold Temperature AC20 Samples Which Were Tested at 25°C.

Mix Type	Sample ID	Average Height (in.)	Resilient Modulus*(ksi) at 25°C and at			Fatigue Life** at 25°C and 200 $\mu\epsilon$	
			50 $\mu\epsilon$	100 $\mu\epsilon$	150 $\mu\epsilon$	Foil Tape	Perm Def
AC20	1x	2.479	362.00	317.58	283.96	3307	2965
Voids: 2.7 – 3.5 %	2x	2.466	364.86	322.75	322.75	3011	2773
	3x	2.470	353.02	319.54	286.79	2062	1772
	6x	2.480	352.74	306.26	285.92	2008	1796
	8x	2.500	391.50	356.38	332.59	2603	2186
	9x	2.481	388.04	339.28	329.28	2979	2786
	Mean:		368.69	326.96	303.22	2662	2380
	Std. Dev.		17.05	17.92	22.31	535	531
	CV (%)		4.62	5.48	7.36	20.09	22.31

* Tests conducted at a dynamic loading frequency of 1 Hertz, at a dynamic load duration of 0.1 sec, and at the indicated strain level.

** Tested using the modified failure criteria and at a dynamic loading frequency of 1 Hertz, at a dynamic load duration of 0.1 sec, and at the indicated initial strain level.

Table 4.9 - Summary of OSU Test Results for Both Mix Types at 0°C.

Mix Type	Sample ID	Average Height (in.)	Resilient Modulus*(ksi) at 0°C			Fatigue** at 0°C and 200 $\mu\epsilon$
			50 $\mu\epsilon$	100 $\mu\epsilon$	150 $\mu\epsilon$	Foil Tape
AC20 Voids: 2.7 - 3.5 %	4	2.48	2283.45	2234.28	2112.60	810
	5	2.48	2118.53	1880.18	1855.37	1770
	7	2.48	2408.18	2304.60	2196.20	3211
	10	2.49	1614.17	1621.60	1639.66	1581
	11	2.46	1934.11	1875.31	1866.97	***
	12	2.49	1707.77	1789.86	1697.26	8320
	Mean:		2011.04	1950.97	1894.68	3138.40
	Std. Dev.		315.87	264.76	221.24	3023.81
	CV (%)		15.7	13.6	11.7	96.4
CA(P)-1 Voids: 2.1 - 3.6 %	13	2.46	2122.56	2147.99	2055.28	19534
	14	2.49	1689.95	1684.95	1697.26	9222
	33	2.46	2097.44	2019.99	1900.16	3888
	35	2.46	1680.88	1677.92	1752.66	7411
	37	2.47	1800.47	1918.22	1893.44	***
	38	2.47	1711.48	1866.20	1882.65	1254
	Mean:		1850.46	1885.88	1863.58	8261.80
	Std. Dev.:		205.61	185.33	125.94	7016.90
	CV (%):		11.1	9.8	6.8	84.9

* Tests conducted at a dynamic loading frequency of 1 Hertz, at a dynamic load duration of 1 sec, and at the indicated strain level.

** Tested using the modified failure criteria and at a dynamic loading frequency of 1 Hertz, at a dynamic load duration of 1 sec (variable magnitude), and at the indicated initial strain level.

*** No test results.

4.2.4 Analysis of Results

The test results were statistically analyzed to determine the repeatability of modulus and fatigue tests within a single laboratory and to determine whether or not modulus tests are repeatable between two different laboratories. To determine repeatability of modulus test results between laboratories, a pairwise Student's t-test was performed at a 5% significance level. That is, the statistical analysis was performed on modulus test results of the same samples tested at both laboratories under approximately the same test conditions.

The results of the paired Student's t-test are summarized in Table 4.10. As indicated, there does not exist a significant difference at the 5% significance level between laboratories for only two test conditions, the AC20 mix type at the strain levels of 50 and 100 microstrain. The test results for the AC20 mix at the 150 microstrain level are just barely significantly different between laboratories. That is, the results would not be considered significantly different at the 4% level (P-value ≈ 0.04). All results are significantly different between laboratories for the CA(P)-1 mix type under all test conditions. In fact, all results are significantly different at the 1% level (all P-values are less than 0.01) for the CA(P)-1 mix type.

To determine the precision (or repeatability) of modulus test results within a single laboratory, the coefficient of variation*** was determined for each set of test results. These results are summarized

---

***CV = coefficient of variation

= (Std. Dev./Mean) × 100

Table 4.10 - Summary of Statistical Analysis (Student's t-test) of Modulus Test Results at 25°C Between Laboratories.

Mix Type	Strain Level ($\mu\epsilon$)	Mean*	Standard Deviation	t crit. (5%)	t calc.	Sig. Diff. ?	P-value
AC20	50	17.6675	36.7675	±2.20	1.665	No	0.2 > P > 0.1
Voids:	100	-4.7358	29.9903	±2.20	-0.547	No	P < 0.2
2.7 - 3.5 %	150	18.1925	27.5872	±2.20	2.284	Yes	P ~ 0.05
CA(P)-1	50	-53.4533	19.8634	±2.20	-9.322	Yes	P < 0.001
Voids:	100	-28.5492	17.8961	±2.20	-5.526	Yes	P < 0.001
2.1 - 3.6 %	150	-25.7100	25.2222	±2.20	-3.449	Yes	0.01 > P > 0.002
CA(P)-1**	50	-68.1233	22.7912	±2.20	-10.354	Yes	P < 0.001
Voids:	100	-52.6725	12.9436	±2.20	-14.097	Yes	P < 0.001
2.1 - 3.6 %	150	-42.9775	17.2747	±2.20	-8.618	Yes	P < 0.001

* Mean represents the average difference between OSHD and OSU modulus test results (i.e., $M_{R(OSHD)} - M_{R(OSU)}$).

** Cold temperature samples tested at the 25°C temperature.

in Table 4.11. As indicated, the coefficients of variation range between 4.6 and 13.3% for the OSU test results at 25°C while those for OSHD range between 5.6 and 13.9% at the same temperature. The coefficients of variation range between 6.8 and 15.7% for the cold temperature test results.

The repeatability of fatigue test results were also measured in terms of the coefficient of variation. These results are summarized in Table 4.12. As indicated, the coefficients of variation are reasonable for both mix types tested at 25°C when either the modified or the permanent deformation failure criteria are used. At the same temperature, using the initial failure criterion method, the coefficient of variation for the AC20 mix is reasonable but that for the CA(P)-1 mix is poor. At the 0°C temperature, the coefficient of variation is very poor for both mix types which were tested using the modified failure criterion.

4.2.5 Discussion of Results

Resilient Modulus Testing. The results of modulus tests on the same specimens between laboratories were determined to be significantly different for the CA(P)-1 (polymer-modified) mix while those for the AC20 mix were, for the most part, not significantly different. That is, it was shown that the OSHD laboratory modulus test results could not be satisfactorily reproduced at the OSU laboratory. Results of modulus tests within each laboratory, however, were determined to be quite repeatable as indicated by the low coefficients of variation which ranged between about 5 and 14% in both laboratories.

Table 4.11 - Statistical Summary of Resilient Modulus Test Results Within Each Laboratory.

Laboratory	Mix Type	Test Temperature (°C)	Strain Level ($\mu\epsilon$)	Mean (ksi)	Standard Deviation	CV (%)
OSHD	AC20	25	50 100 150	328.40 283.07 277.12	42.72 28.72 28.04	13.00 10.14 10.12
	CA(P)-1	25	50 100 150	245.92 229.25 226.10	34.28 23.38 24.87	13.94 10.20 11.00
	CA(P)-1*	25	50 100 150	263.26 244.46 230.05	17.17 13.68 23.86	6.52 5.60 10.37
OSU	AC20	25	50 100 150	310.74 287.81 267.26	26.20 22.47 35.59	8.43 7.81 13.30
	AC20*	25	50 100 150	368.69 326.96 303.22	17.05 17.92 22.31	4.62 5.48 7.36
	AC20	0	50 100 150	2011.04 1950.97 1894.68	315.87 264.76 221.24	15.70 13.60 11.70
	CA(P)-1	25	50 100 150	299.38 257.80 251.81	19.48 16.16 16.01	6.51 6.27 6.36
	CA(P)-1*	25	50 100 150	331.38 297.14 273.03	28.88 20.16 15.79	8.71 6.78 5.78
	CA(P)-1	0	50 100 150	1850.46 1885.88 1863.58	205.61 185.33 125.94	11.10 9.80 6.80

* Originally scheduled to be tested at 0°C.

Table 4.12 – Statistical Summary of Fatigue Test Results.

Mix Type	Test Temperature (°C)	Failure Criteria	Mean	Standard Deviation	CV (%)	Number of Samples
AC20 Voids: 2.7 – 3.5 %	25	Initial Modified Perm Def	2465 2662 2380	346 535 531	14.0 20.1 22.3	12 6 6
	0	Modified	3138	3024	96.4	5
CA(P)–1 Voids: 2.1 – 3.6 %	25	Initial Modified Perm Def	9441 4537 4187	3865 846 830	40.9 18.6 19.8	12 6 6
	0	Modified	8262	7017	84.9	5

The significant differences between laboratories that are not apparent within laboratories would suggest the differences lie in the manner in which the moduli are determined in each laboratory. That is, the two laboratories use different test conditions and procedures. Although the exact effects of these differences are not well understood, it is clear that the equipment and procedures need to be better standardized.

Fatigue Testing. For fatigue testing no comparison between laboratories is possible since these tests were conducted exclusively at OSU. Thus, only the repeatability within the OSU laboratory is addressed. The most notable result is that the modified failure criterion significantly improved the repeatability of fatigue results for the CA(P)-1 mix at the 25°C temperature. At this temperature the repeatability is satisfactory for both mix types when using the modified failure criterion. Tests conducted at the 0°C temperature, however, result in unsatisfactory repeatability for both mix types. It should be noted that all of the specimens tested for fatigue at the low (0°C) temperature experienced brittle failure. In all cases, there was strong visual evidence that the failure was an adhesive one between the asphalt cement and the aggregate. This was also evidenced in the specimens tested at the 25°C temperature but not nearly to the extent evidenced in the specimens tested at the 0°C temperature. This may have contributed to the high variability in the fatigue test results at the low (0°C) temperature.

It should also be noted that approximately half of the samples failed on one side (the side which corresponds to the bottom of the specimen during compaction). This was observed during fatigue testing only at the 25°C temperature. This would suggest there may have existed "bridging" and segregation in the specimens during compaction. This, in turn, may have contributed to some variability in the fatigue results.

One unexpected result from the fatigue testing at the two different temperatures contradicts the commonly accepted theory that asphaltic concrete fails more rapidly at lower temperatures. Results in Table 4.12 show that fatigue life actually appears to be greater at the lower temperature. This trend is shown for both types of material. For the AC20 mix, the five tests at 0°C averaged 3138 repetitions while the six tests at 25°C averaged 2662 repetitions. For the CA(P)-1 mix, the five tests at 0°C averaged 8262 repetitions while the six tests at 25°C averaged 4537 repetitions.

Two different factors could partially explain this unexpected result: (1) the variability of cold temperature fatigue testing is so great that it is not possible to place full confidence in the results; and (2) different pieces of equipment were used for the two test temperatures (i.e., the 25°C samples were tested with a pneumatically actuated device while the 0°C samples were tested with a hydraulically actuated device). The major difference that would be expected to affect the results is that the waveforms of the two devices are different. That is, the cold temperature samples were tested using a haversine while the warm temperature samples were tested using a square wave. Further study would be required to determine how the results are affected by this difference in load waveforms.

4.3 Conclusions and Recommendations

4.3.1 Conclusions

The following conclusions appear to be warranted as a result of the contents of this paper:

1. Modulus test results are highly repeatable within the OSHD or the OSU laboratory.
2. Modulus test results are, for the most part, not reproducible between the OSHD and OSU laboratories. The cause of the differences appear to lie in the test conditions, procedures, and apparatus used to determine resilient modulus of asphalt concrete specimens.
3. Fatigue test results at 25°C can be highly to moderately repeatable providing that the improved criteria are used to establish failure.
4. Repeatability of fatigue tests at 0°C were found to be very poor. This is not too surprising since others (5,6,7,8,9) have reported similar results at low temperatures.

4.3.2 Recommendations

The following recommendations appear to be warranted as a result of the contents of this paper:

1. Comparison of modulus test results between the OSHD and OSU laboratories are not warranted. This is primarily due to the significant differences in test conditions, procedures, and test apparatus used to measure resilient modulus. These differences need to be resolved as soon as possible.

2. Sample preparation (i.e., compaction) should be carried out such that segregation and "bridging" does not occur.
3. Additional fatigue testing should be carried out with improved temperature control to further investigate the poor repeatability of fatigue results at the 0°C temperature.

4.4 References

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5.0 CONCLUSIONS

As a result of this thesis the following conclusions appear to be warranted:

1. Partial depth cold in-place recycling of asphalt concrete pavements in Oregon has provided a successful alternative to other rehabilitation techniques.
2. Of the 52 CIR projects constructed between 1984 and 1988, 92% (48) are performing very well. Four of the projects required repair.
3. CIR mixes can provide modulus and fatigue properties comparable to those of conventional hot mixes. Marshall stability values, however, are slightly lower while flow values remain slightly higher relative to hot mixes.
4. The current method of determining the emulsion and water contents for CIR mixes accurately estimates (within 0.2%) the quantities actually used in the field. Design criteria based on mix property tests, however, need to be further developed such that oil and water contents can be determined from the results.
5. Although specific guidelines have not yet been developed, the construction guidelines presented have been successfully used to construct CIR pavements. These pavements have provided an improved wearing course at significant savings in capitol costs.
6. The structural contribution of a properly designed and

constructed CIR mixture can be comparable to that of a conventional asphalt concrete mixture.

7. Modulus test results which have, in part, been used to evaluate the performance of CIR mixes can be highly repeatable within a single laboratory. However, it was shown that modulus test results could not be satisfactorily reproduced (within a 95% confidence level) between two laboratories. The significant differences appear to be associated with differences in test conditions, test procedures, and test apparatus used in the two laboratories to determine the resilient modulus.

8. Fatigue test results can be highly repeatable within a single laboratory depending on the test temperature and criterion used to determine failure.

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APPENDICES

APPENDIX A

DETAILED MIX DESIGN PROCEDURE FOR CIR

1.0 Determine the following from the sample obtained with the 16-in mill:

1.1 Asphalt content, in percent, and extracted gradation

1.2 Penetration at 77°F, in dmm.

1.3 Absolute Viscosity at 140°F, in poises.

1.4 RAP Gradation (1/2", 1/4", and No. 10 sieves)

2.0 Determine the estimated emulsion content from Figure A.1 and the following equation:

$$EC_{EST} = 1.2 + A_G + A_{A/C} + A_{P/V}$$

where:

EC_{EST}= estimated emulsion content, in percent

A_G= adjustment for gradation, in percent

$A_{A/C}$= adjustment for asphalt content, in percent

$A_{P/V}$= adjustment for penetration or viscosity, in percent

NOTE: For borderline cases in Figure A.1, use the adjustment resulting in a lower estimated emulsion content. For discrepancy between penetration and viscosity adjustments (Figure A.1), use the adjustment resulting in a lower estimated emulsion content (See Example in Figure A.1).

3.0 Prepare test briquets using the following procedure:

3.1 Split the millings into approximately 15,000 gram batches.

3.2 Screen the sample on the 1-in sieve. Reduce all material retained on the 1-in sieve such that 100% of the sample passes the 1-in sieve using a hammer or chisel.

3.3 Batch five 1100 gram ± samples of the millings at the adjusted gradation. The adjusted gradation is determined from Figure A.2. Determine the asphalt content.

3.4 Using the remaining material determine the optimum total liquids content using OSHD TM-126 with the modification that the optimum total liquids content occurs at at liquid loss of 1-4 ml (1-4 grams).

3.5 Calculate water contents (based on dry weight of millings) to be added to the samples for each emulsion content using the following equation:

%water = opt. total liquids content - %emulsion content

Briquets are to be prepared with emulsion contents at the estimated emulsion content (EC_{EST}), at EC_{EST} - 0.3%, at EC_{EST} + 0.3%, at EC_{EST} + 0.6%, and at EC_{EST} + 0.9%.

3.6 Heat the five 1100 gram samples to 140°F ± for 1 hour.

3.7 Add the water calculated above to the five samples and thoroughly mix by hand.

3.8 Add the emulsion contents to the premoistened millings. The emulsion is to be preheated to 140°F ± for 1 hour and mixed thoroughly into the batch by hand.

3.9 Dump the material into a 12-in x 17-in baking pan and allow to cure for 1 hour at 140°F ± to simulate the average time elapsed between the paver laydown and the initial compaction during actual construction.

3.10 Mold the samples using standard Hveem procedures to produce 2.5-in ± briquets as described below:

1) Preheat molds to 140°F ±.

2) Compact the samples using 150 blows at 300 psi.

3) Cure the briquets overnight at 140°F ± and recompact 150 blows at 300 psi.

4) Lay the molds on their side and cure the briquets for 24 hours at 140°F ± prior to extrusion.

5) Extrude the briquets using a compression testing machine.

6) Lay the briquets on their side to maximize surface exposure and cure for 72 ± 24 hours at room temperature prior to testing.

7) Determine bulk gravity.

4.0 Test the specimens for mix properties as follows:

4.1 Test for Hveem stability. Plot stability versus emulsion content and draw a smooth curve through the data points.

4.2 Test for resilient modulus at 77°F. Plot modulus versus emulsion content and draw a smooth curve throught the data points.

4.3 Check voids after second compaction.

5.0 Analyze mix properties:

5.1 Record the emulsion content corresponding to the peak of the

Hveem stability versus emulsion content curve. (This is EC_{DES}.)

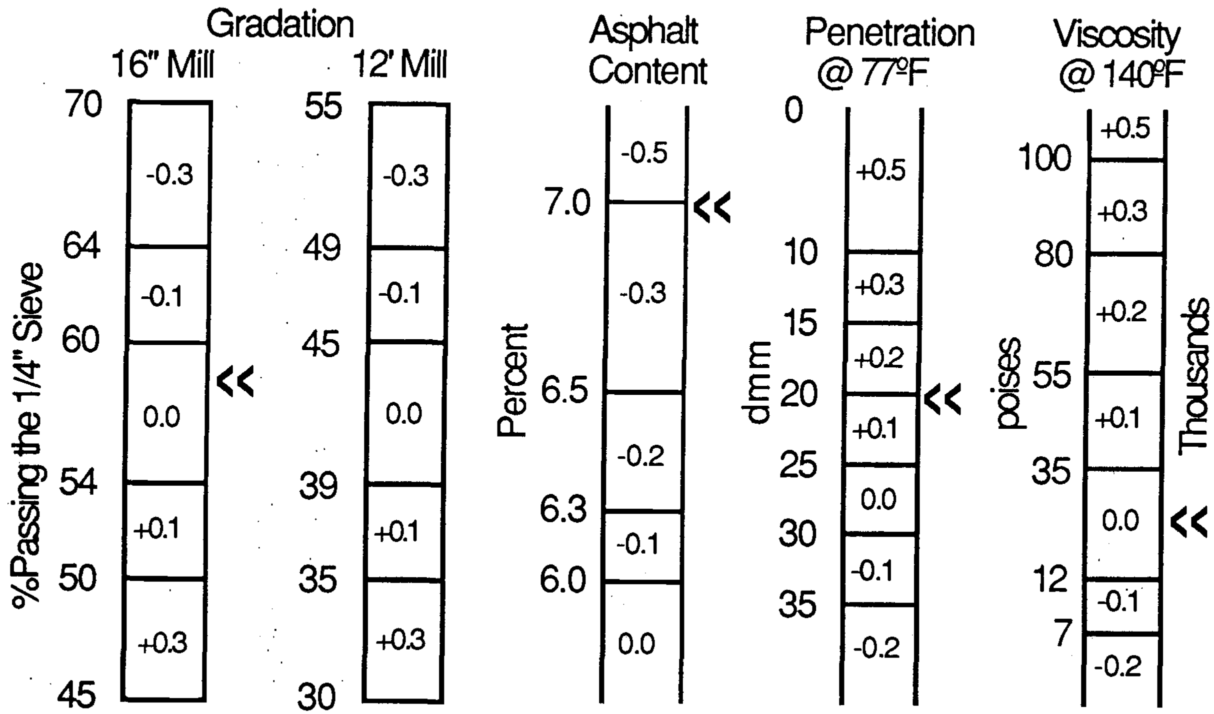
5.2 Record the emulsion content corresponding to the peak of the modulus versus emulsion content curve.

5.3 Check to ensure that EC_{DES} is approximately equal to EC_{EST}. Record EC_{DES} and EC_{EST}.

6.0 Recommended emulsion and W/C:

6.1 Use EC_{EST}.

6.2 Use WC_{CTB}



Example:

Given:

58% passing the 1/4" screen on the 16" mill, 7% residual asphalt, a penetration of 20 dmm, and a viscosity of 19,000 poises.

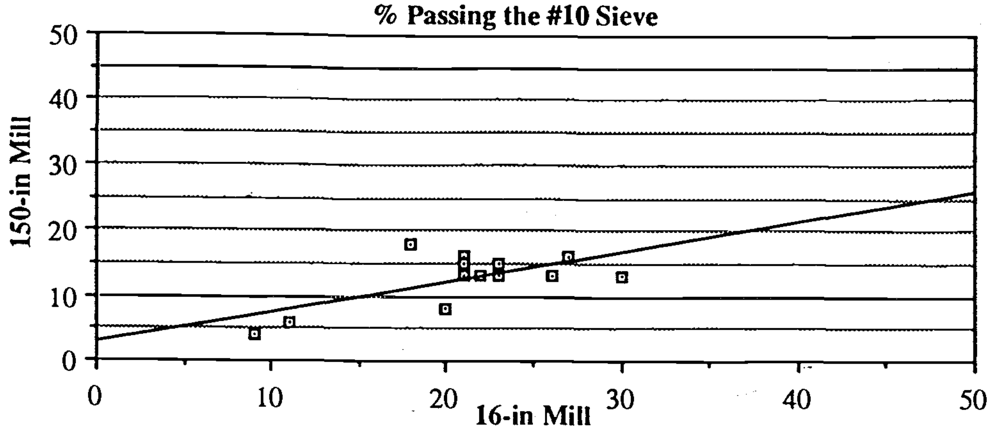
Adjustments (for borderline cases, use adjustment producing lower emulsion content):

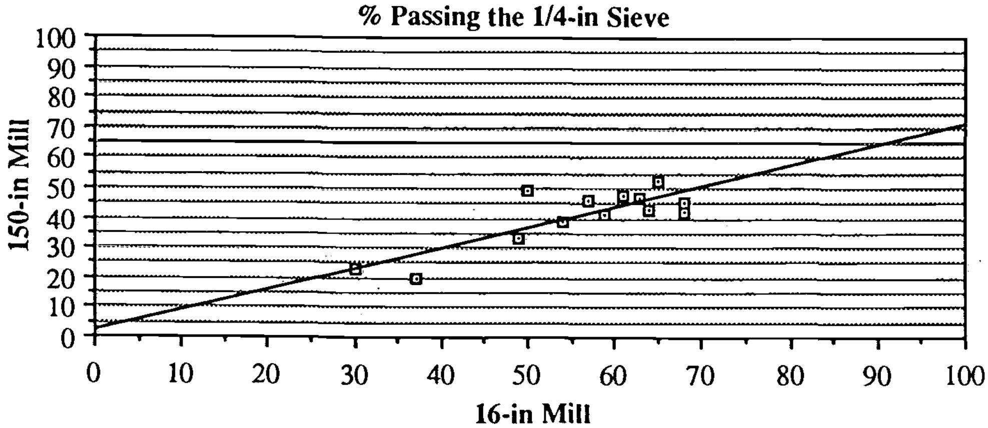
0.0% for gradation, -0.5% for asphalt content, and 0.0% for penetration/viscosity

Estimated Emulsion Content:

1.2 % + 0.0% - 0.5% + 0.0% = <u>0.7%</u>

Figure A.1 – Emulsion Content Adjustments for Gradation, Asphalt Content, and Asphalt Softness.





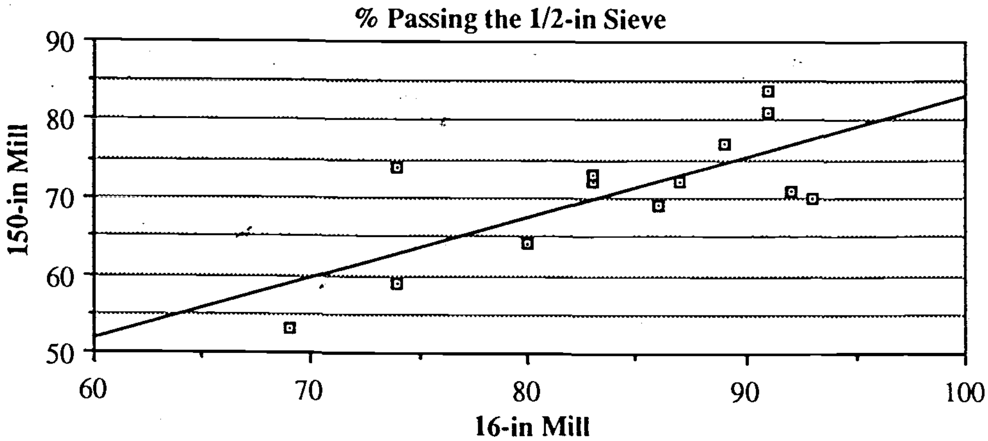


Figure A.2 – Determination of the Adjusted Gradation (for the 150-in Mill) from the Gradation on the 16-in Mill.

APPENDIX B

FATIGUE TEST PROCEDURE

1.0 INTRODUCTION

1.1 Scope

This appendix describes the procedure used to determine the fatigue life of laboratory-fabricated and field-recovered bituminous mixtures. The described procedure is essentially an extension of the Standard Method of Indirect Tension Test for Resilient Modulus of Bituminous Mixtures[1] (ASTM D4123-82). Other applicable documents include ASTM standards:

D1559 -- Test Method for Resistance to Plastic Flow of Bituminous Mixture Using Marshall Apparatus.

D1561 -- Method of Preparation of Bituminous Mixture Test Specimens by Means of California Kneading Compactor.

D3387 -- Test for Compaction and Shear Properties of Bituminous Mixtures by Means of the U.S. Corps of Engineers Gyratory Testing Machine (GTM).

D3496 -- Method for Preparation of Bituminous Mixture Specimens for Dynamic Modulus Testing.

D3515 -- Specifications for Hot-Mixed, Hot-Laid Bituminous Paving Mixtures.

1.2 Summary of Procedure

The repeated-load indirect tensile test for determining the resilient modulus and fatigue life of bituminous mixtures is conducted by application of compressive loads in the form of a pulse, haversine,

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[1] Annual Book of ASTM Standards, Vol. 04.03, 1988.

square, or other suitable waveform. The load is applied in the vertical diametral plane of a cylindrical asphalt concrete specimen as shown in Figure B.1. The magnitude of the repeated-load is the load that results in a specified recoverable horizontal deformation or tensile strain as determined via ASTM D4123.

The number of load applications that results in a specified amount of permanent horizontal deformation is the fatigue life of the specimen. Typical amounts of permanent horizontal deformation range between 0.28 and 0.36-in.

1.3 Significance and Use

Fatigue values (i.e., fatigue life) can be used to evaluate the relative performance of asphaltic concrete materials as well as be used as input for pavement thickness design or pavement evaluation and analysis. The test can also be used to study the effects of temperature, repeated-load magnitude, loading frequency and duration, etc. However, since the test is destructive, tests cannot be repeated on the same specimen as can be done for resilient modulus.

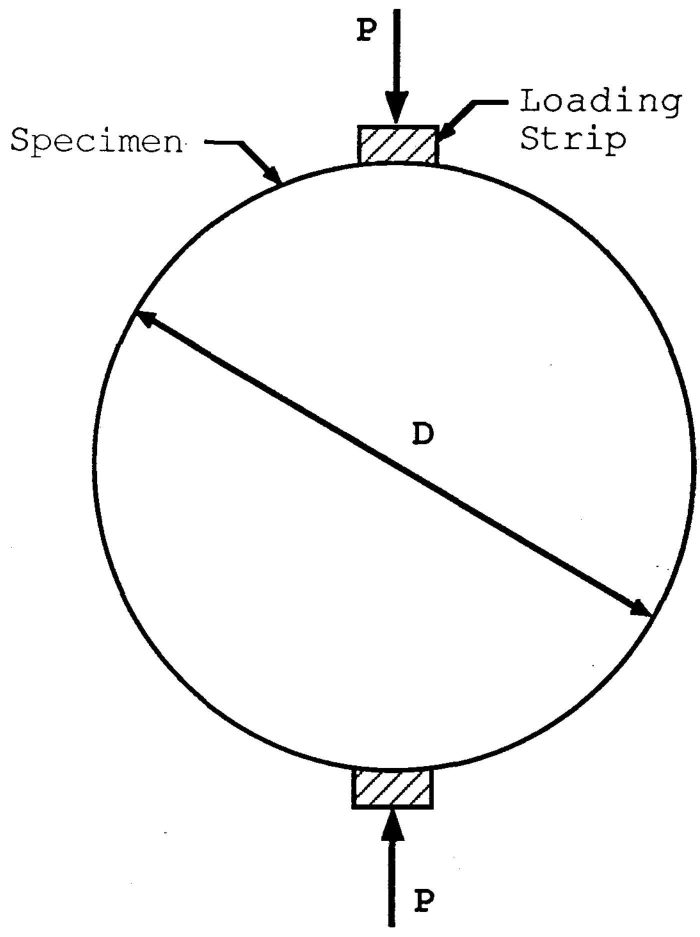


Figure B.1 – Loading of a Cylindrical Asphalt Concrete Specimen in the Repeated Load Indirect Tensile Test.

2.0 TEST PROCEDURE

2.1 Test Apparatus

The apparatus to perform the indirect tensile fatigue test should conform to that specified in ASTM D4123 with the added ability to count and record the number of load applications. In addition, the apparatus should have the ability to automatically discontinue load applications when the specified amount of permanent horizontal deformation has occurred.

2.2 Specimen Preparation

2.2.1 Laboratory-Fabricated Specimens. Prepare laboratory-fabricated specimens in accordance with ASTM Methods D1559, D1561, D3387, or D3496. The resulting specimens should have a height of at least 2-in. and a diameter of 4-in. for aggregate having a maximum size of 1-in. For aggregate having a maximum size of 1.5-in., the height should be at least 3-in. and the diameter should be 6-in.

2.2.1 Core Specimens. Core specimens should have relatively smooth and parallel surfaces. Height and diameter requirements specified for the laboratory-fabricated specimens are applicable to cores specimens.

2.3 Failure Criteria

As previously mentioned, the fatigue life is the number of load applications required to induce a specified amount of permanent horizontal deformation. Failure criteria typically range between 0.28 and 0.36-in. which roughly corresponds to the size of the crack developed in the specimen during fatigue. Experience has shown, however, that failure criteria greater than about 0.35-in. may result in the test specimen failing dramatically ("exploding") when the induced tensile strain is on the order of 100 microstrain or greater. That is, when the specimen is near failure, the sample may explode due to the failure criterion (permanent horizontal deformation) being too large.

Once the failure criterion is selected, it can be determined during the fatigue test by means of an electronic circuit which is closed with lead-based foil tape (burglar alarm tape for glass windows). When the foil tape breaks, the circuit is opened and load applications discontinue. The amount of permanent horizontal deformation that occurs before the foil tape breaks is set by the length of loop placed in the foil tape on both sides of the specimen (see Figure B.2).

2.4 Fatigue Test

The indirect tensile fatigue test is conducted as follows:

1. Determine loading conditions (i.e., load frequency and duration), test temperature, initial recoverable tensile strain, and amount of permanent horizontal deformation to be used in the determination of the fatigue life.
2. Determine the load magnitude required to induce the specified recoverable tensile strain via ASTM D4123.
3. Place lead-based foil tape around the diametral axis perpendicular to the loading axis such that the foil tape has two loops of length corresponding to the specified amount of permanent horizontal deformation (see Figure B.3a). The foil tape must not connect end-to-end since this would cause a short circuit.

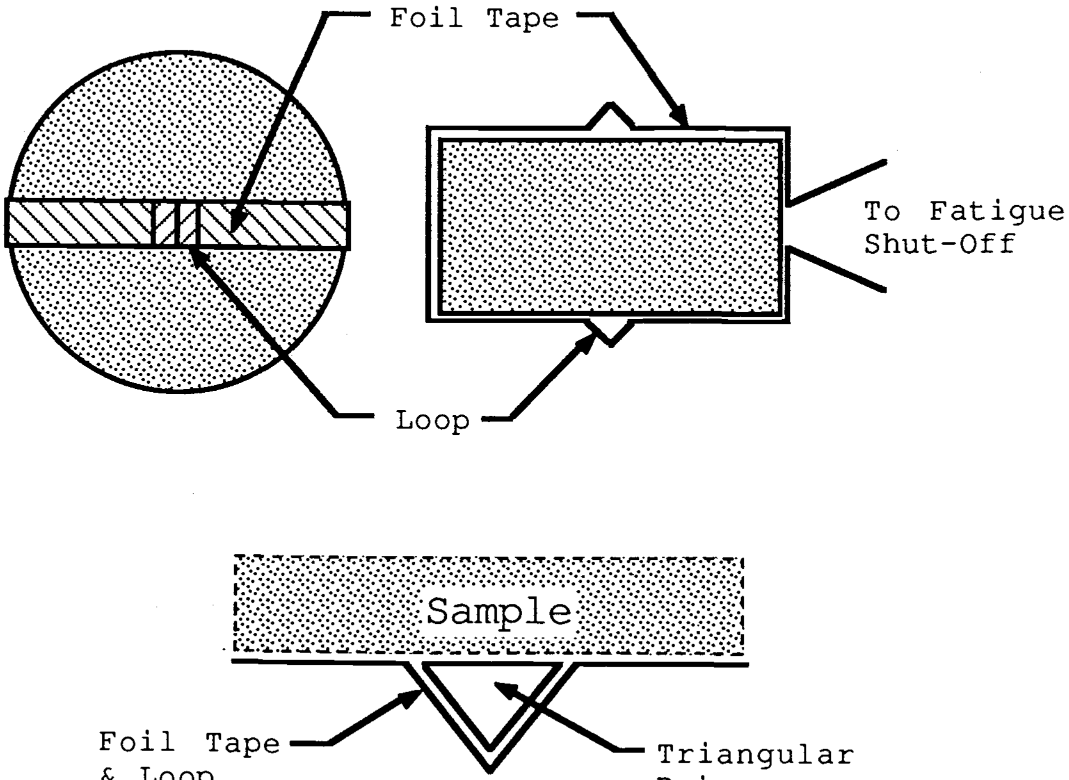
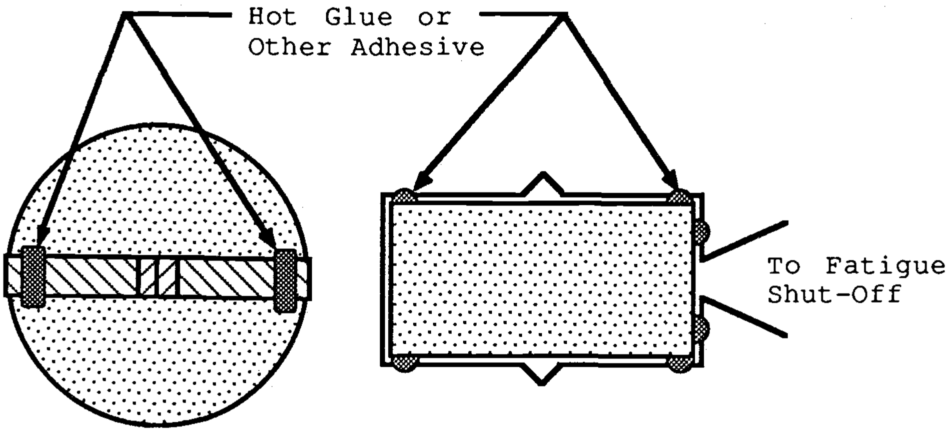
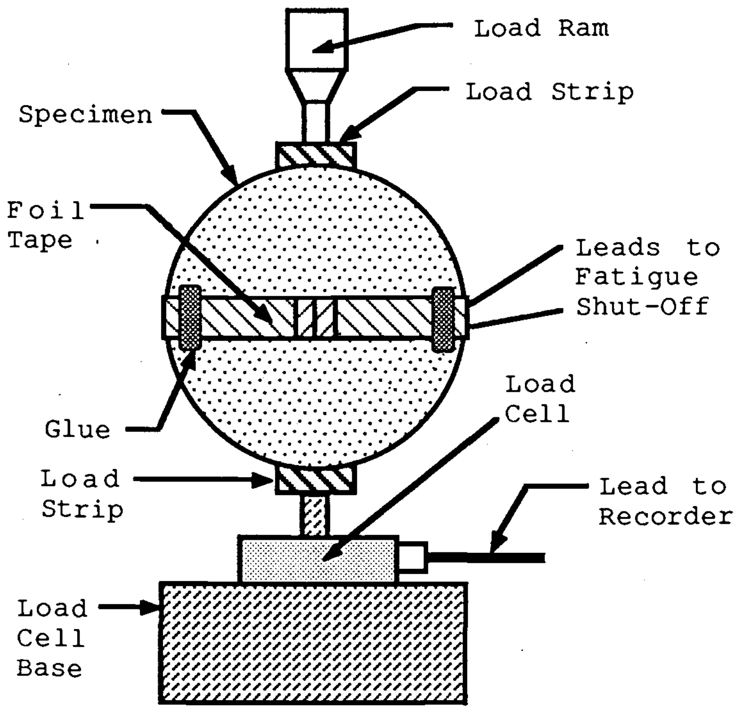


Figure B.2 – Asphalt Concrete Specimen with Foil Tape Having Loops on Both Sides of the Specimen.



a) Location of Glue



b) Orientation of Fatigue Specimen

Figure B.3 - Fatigue Specimen Set Up

4. Secure the foil tape by means of hot glue or other appropriate adhesive as shown in Figure B.3a.
5. Solder leads to each end of the foil tape and connect the leads to a circuit that continues load applications while closed and discontinues loading when open.
6. Place the test specimen in the test apparatus such that the line of the foil tape is perpendicular to the line of loading as shown in Figure B.3b.
7. Apply the static load that was applied when determining the load magnitude to induce the specified recoverable tensile strain.
8. Apply a repeated-load such that the magnitude of the load corresponds to that which induced the specified amount of recoverable tensile strain.
9. Count and record the number of load applications required to break the foil tape.