Evaluation of Cold In-Place Recycling for Rehabilitation of Transverse Cracking on US 412

Final Report

by

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SI (METRIC) CONVERSION FACTORS

Symbol	When you know	Multiply by	To Find	Symbol	Symbol	When you know	Multiply by	To Find	Symbol
Approximate Conversions to SI Units					*Approximate Conversions from SI Units*				
		LENGTH					**LENGTH**		
in	inches	25.40	millimeters	mm	mm	millimeters	0.0394	inches	in
ft	feet	0.3048	meters	m	m	meters	3.281	feet	ft
yd	yards	0.9144	meters	m	m	meters	1.094	yards	yds
mi	miles	1.609	kilometers	km	km	kilometers	0.6214	miles	mi
		AREA					**AREA**		
in^2	square inches	645.2	square millimeters	mm^2	mm^2	square millimeters	0.00155	square inches	in^2
ft^2	square feet	0.0929	square meters	m^2	m^2	square meters	10.764	square feet	ft^2
yd^2	square yards	0.8361	square meters	m^2	m^2	square meters	1.196	square yards	yd^2
ac	acres	0.4047	hectacres	ha	ha	hectacres	2.471	acres	ac
mi^2	square miles	2.590	square kilometers	km^2	km^2	square kilometers	0.3861	square miles	mi^2
		VOLUME					**VOLUME**		
fl oz	fluid ounces	29.57	milliliters	mL	mL	milliliters	0.0338	fluid ounces	fl oz
gal	gallon	3.785	liters	L	L	liters	0.2642	gallon	gal
ft^3	cubic feet	0.0283	cubic meters	m^3	m^3	cubic meters	35.315	cubic feet	ft^3
yd^3	cubic yards	0.7645	cubic meters	m^3	m^3	cubic meters	1.308	cubic yards	yd^3
		MASS					**MASS**		
oz	ounces	28.35	grams	g	g	grams	0.0353	ounces	oz
lb	pounds	0.4536	kilograms	kg	kg	kilograms	2.205	pounds	lb
T	short tons (2000 lb)	0.907	megagrams	Mg	Mg	megagrams	1.1023	short tons (2000 lb)	T
		TEMPERATURE (exact)					**TEMPERATURE (exact)**		
°F	degrees Fahrenheit	(°F-32)/1.8	degrees Celsius	°C	°C	degrees Fahrenheit	9/5(°C)+32	degrees Celsius	°F
		FORCE and PRESSURE or STRESS					**FORCE and PRESSURE or STRESS**		
lbf	poundforce	4.448	Newtons	N	N	Newtons	0.2248	poundforce	lbf
lbf/in^2	poundforce per square inch	6.895	kilopascals	kPa	kPa	kilopascals	0.1450	poundforce per square inch	lbf/in^2

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15. Abstract

Successful rehabilitation of transverse cracked hot mix asphalt (HMA) pavements has been a challenge for state DOTs. Conventional thin HMA overlays allow the rapid return of the existing transverse cracks and thicker HMA overlays are cost prohibitive. Cold In-Place Recycling (CIR) has been shown to be a cost-effective procedure for rehabilitation of transverse cracked HMA pavements. The use of fly ash Portland cement slurry to pretreat large transverse cracks has made CIR applicable to HMA pavements with severe (wide) transverse cracks. The new *Mechanistic-Empirical Pavement Design Guide* (M-EPDG) uses dynamic modulus as a material characterization parameter for asphalt mixtures. In order to ensure CIR's place in pavement maintenance and rehabilitation activities, guidelines on dynamic modulus (E*) parameters need to be developed.

The objectives of this study were to evaluate the effectiveness of CIR with slurry crack injection to rehabilitate transverse cracked HMA pavements on two rehabilitation projects on US 412 in Beaver and Harper counties, to investigate the dynamic modulus properties of CIR mixtures, and to evaluate the appropriateness of the M-EPDG predictive equation for dynamic modulus for CIR mixtures.

The CIR treatments have reduced the occurrence of transverse cracking but longitudinal wheel path cracking is occurring in the CIR test sections. The longitudinal cracking might be attributed to the thin, stiff HMA layer placed over the softer CIR layer. AASHTO TP 62 can be used to determine dynamic modulus of CIR mixtures with slight modification. The predictive dynamic modulus equations give good agreement with measured values at the three higher test temperatures if the aggregate properties are based on the RAP gradation and binder properties on the base binder in the asphalt emulsion.

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TABLE OF CONTENTS

page

LIST OF FIGURES viii
LIST OF TABLES ix

CHAPTER 1 INTRODUCTION 1
- PROBLEM STATEMENT 1
- OBJECTIVES 2
- WORK PLAN AND SCHEDULE 2
  - Phase 1: Field Evaluation 2
  - Phase 2. Laboratory Evaluation of CIR on US 412 3
- BENEFITS 4

CHAPTER 2 LITERATURE REVIEW 5
- RECYCLING 5
  - Cold Recycling (CR) 5
    - Cold Central Plant Recycling (CCPR) 5
    - Cold In-Place Recycling (CIR) 6
- CIR EQUIPMENT 6
  - Single-Unit Trains 6
  - Two-Unit CIR Train 7
  - Multi-Unit CIR Trains 8
- ADDITIVES 9
  - Asphalt Emulsions 9
  - Rejuvenators 10
  - Chemical Additives 10
  - Cutback Asphalts 10
- MIX DESIGN 10
  - AASHTO-AGC Task Force 38 11
  - Modified Superpave Mix Design Methods 11
  - SemMaterials ReFlex® CIR Design Method 11
- WHY RECYCLING? 11
- MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE (M-EPDG) 12
  - Need for the Design Guide 12
  - General Input Requirements 12
  - Layers 13
    - Asphalt Mix Screen 13
    - Asphalt Binder Screen 13
    - Asphalt General Screen 13

CHAPTER 3 FIELD OBSERVATIONS 15
- SITE DESCRIPTIONS 15
  - Beaver County Sites 15
    - *Test Sections* 19

page

Harper County Site 19
Test Section 20

CHAPTER 4 LABORATORY TEST PROCEDURES 25
INTRODUCTION 25
MATERIALS..25
Reclaimed Asphalt Pavement (RAP)..25
Asphalt Emulsions 27
Lime 27
Conventional HMA Mix 28
TEST PLAN..28
Preparation of Dynamic Modulus Test Specimen 29
Sample Requirements..29
Batching 29
Emulsion Content..30
Mixing 31
Compaction 31
Curing 31
Coring & Sawing 32
Testing...34
Dynamic Modulus 34
Aging Effects 34

CHAPTER 5 TEST RESULTS 37
FIELD MEASUREMENTS 37
Summary 37
DYNAMIC MODULUS TEST RESULTS...39
CIR Samples 39
Conventional Testing 39
Oven Aged Dynamic Modulus 40
HMA Samples...43
VOLUMETRIC PROPERTIES...46

CHAPTER 6 ANALYSIS OF TEST RESULTS...49
LABORATORY DYNAMIC MODULUS 49
Initial CIR Test Data...49
Comparisons with HMA 51
Aging Effects 53
MASTER CURVES 54
Extrapolated results for E* at 54.4°C (130°F)...56
E* PREDICTIVE EQUATION 59
Comparison of Experimental and Predicted Master Curves......................63
M-EPDG 66
Project Information 66

page

Structure 67
Material Properties 68
Design Trials and Results 68

CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS 71
CONCLUSIONS 71
Field Test Sites 71
Dynamic Modulus Testing 71
RECOMMENDATIONS 72

REFERENCES 75

LIST OF FIGURES

page

FIGURE 1 Traditional single-unit CIR train (8)....7
FIGURE 2 Super single-unit CIR train (8)....7
FIGURE 3 Two-unit CIR train....8
FIGURE 4 Schematic of a multi-unit CIR train (8)....9
FIGURE 5 Multi-unit CIR train (8)....9
FIGURE 6 US 412 prior to CIR rehabilitation....15
FIGURE 7 Fly ash slurry crack injection on US 412....16
FIGURE 8 Slurry injection equipment....16
FIGURE 9 Pressure injecting transverse cracks with slurry....17
FIGURE 10 Treated transverse crack....17
FIGURE 11 Cold recycling US 412 with multi-unit CIR train....18
FIGURE 12 Laydown of CIR mixture....18
FIGURE 13 Placement of HMA overlay on US 412 Beaver, County....19
FIGURE 14 Typical distress on US 412 Harper, County....20
FIGURE 15 US 412 Harper, County after slurry crack injection....21
FIGURE 16 Installation of precoated fabric membrane strip....21
FIGURE 17 Seating precoated fabric membrane strip with pneumatic roller....22
FIGURE 18 Seated precoated fabric membrane strip....22
FIGURE 19 Installation of precoated fabric membrane strips....23
FIGURE 20 Grain-size distribution curves for RAP and extracted aggregate....26
FIGURE 21 Bucket mixer used for mixing CIR samples....32
FIGURE 22 Sample being cored to required test diameter....33
FIGURE 23 Sample is sawed to obtain parallel faces....33
FIGURE 24 Test specimens for dynamic modulus testing....34
FIGURE 25 Test procedure for dynamic modulus of CIR samples....35
FIGURE 26 Comparison of E* with temperature at 5 Hz between ReFlex® and HMA....52
FIGURE 27 Comparison of E* with temperature at 5 Hz between CSS-1h and HMA....52
FIGURE 28 Comparison of E* with temperature at 5 Hz between HFE 150P and HMA..53
FIGURE 29 Master curve for US 412 and ReFlex® emulsion....57
FIGURE 30 Master curve for US 412 RAP and CSS-1h emulsion....57
FIGURE 31 Master curve for US 412 RAP and CSS-1h+Lime emulsion....58
FIGURE 32 Master curve for US-412 RAP and HFE-150P emulsion....58
FIGURE 33 Experimental and predicted dynamic modulus at 5 Hz for ReFlex® emulsion....64
FIGURE 34 Experimental and predicted dynamic modulus at 5 Hz for CSS-1h and CSS-1h + Lime emulsion....65
FIGURE 35 Experimental and predicted dynamic modulus at 5 Hz for HFE-150P emulsion....65
FIGURE 36 Typical performance plot....67
FIGURE 37 M-EPDG trial section....68

LIST OF TABLES

page

TABLE 1. RAP and Recovered Aggregate Graduation 26
TABLE 2. Residual Asphalt Cement Content of Emulsions...27
TABLE 3. S-4 Mix Design 28
TABLE 4. Criteria for Acceptance of Dynamic Modulus Test Specimen (20)...................30
TABLE 5. Percentages of Water and Emulsions Based on Dry Weight of RAP................31
TABLE 6. Test Parameters for Dynamic Modulus Test (20)...36
TABLE 7. Summary of Distress Surveys...38
TABLE 8. Dynamic Modulus for Samples Prepared with ReFlex® Emulsion 40
TABLE 9. Dynamic Modulus for Samples Prepared with CSS – 1h 41
TABLE 10. Dynamic Modulus for Samples Prepared with CSS – 1h + Lime 42
TABLE 11. Dynamic Modulus for Samples prepared with HFE 150P..............................43
TABLE 12. Dynamic Modulus for HMA Samples Prepared with PG 76-28 44
TABLE 13. Dynamic Modulus for HMA Samples Prepared with PG 70-28 45
TABLE 14. Dynamic Modulus for HMA Samples Prepared with PG 64-22 46
TABLE 15. Volumetric and Mix Design Properties 47
TABLE 16. ANOVA for CIR Dynamic Modulus..49
TABLE 17. Results of Duncan's Multiple Range Test for CIR Dynamic Modulus at 4.4°C (40°F) Test Temperature 50
TABLE 18. Results of Duncan's Multiple Range Test for CIR Dynamic Modulus at 21.1°C (70°F) Test Temperature 50
TABLE 19. Results of Duncan's Multiple Range Test for CIR Dynamic Modulus at 37.8°C (100°F) Test Temperature 50
TABLE 20. Effect of LTOA on E* 54
TABLE 21. Average Dynamic Modulus with Extrapolated Values..................................56
TABLE 22. Predicted Dynamic Modulus for US 412 RAP and ReFlex® Emulsion 60
TABLE 23. Predicted Dynamic Modulus for US 412 and CSS-1h Emulsion 61
TABLE 24. Predicted Dynamic Modulus for US 412 and CSS-1h+Lime..........................62
TABLE 25. Predicted Dynamic Modulus for US 412 and HFE 150P 63
TABLE 26. Percent Increase in Predicted E* Compared to Measured E*..........................64
TABLE 27. Default Performance Criteria (18) 67
TABLE 28. Trial M-EPDG Runs to Arrive at Minimum HMA Overlay Thickness 69
TABLE 29. Minimum HMA Overlay Thickness 70
TABLE 30. Preliminary Recommended E* Values for CIR..73

CHAPTER 1

INTRODUCTION

PROBLEM STATEMENT

Successful rehabilitation of transverse cracked hot mix asphalt (HMA) pavements has been a challenge for state DOTs. Conventional thin HMA overlays generally allow the return of the existing transverse cracks in a short period of time. Thicker HMA overlays have shown a reduced occurrence of reflective cracking, but the procedure becomes cost prohibitive.

Cold in-place recycling (CIR) has been shown to be a cost-effective procedure for rehabilitation of transverse cracked HMA pavements. Recent advances in emulsion formulations and additives have made marked improvement in the performance of CIR pavements. The use of a fly ash Portland cement slurry to pretreat large transverse cracks has made CIR applicable to HMA pavements with severe (wide) transverse cracks.

Some of the reported drawbacks to the use of CIR are unfamiliarity with the process and limited guidelines on the structural support to assign to the CIR layer in overlay thickness design. Oklahoma is fortunate in that the surrounding states (Kansas, New Mexico, Texas, Nebraska, Arizona, and Iowa) have significant experience in CIR procedures and experienced contractors are available.

With the development of the *2002 Design Guide for New and Rehabilitated Pavement Structures,* or the Mechanistic-Empirical Pavement Design Guide (M-EPDG) as it is now called, there is a new emphasis in mechanistic-empirical thickness design procedures. Materials input parameters for these procedures are typically either resilient modulus or dynamic modulus, and Poisson's ratio.

The M-EPDG uses dynamic modulus and Poisson's ratio as the material characterization parameters for asphalt mixtures. The procedure is contained in AASHTO TP 62. The test is performed at different temperatures, stress levels and loading frequencies and a master curve is developed that describes the relationship between mix stiffness, mix temperature and time rate of loading. This master curve is combined with a binder aging model and is used as the basis for selecting mixture modulus values over the service life of the pavement.

The M-EPDG uses a hierarchical approach with three levels of materials characterization. The first level provides the highest design reliability and each succeeding level is a drop in design reliability. The first or highest level uses measured dynamic modulus and Poisson's ratio for each asphalt stabilized mixture used in the design. The second and third levels of material characterization entail the use of default master curves from predictive equations developed by the NCHRP 1-37A research team.

In order to ensure CIR's place in pavement maintenance and rehabilitation activities, guidelines on dynamic modulus (E*) parameters need to be developed. There is evidence that the E* predictive equations provided for hot mix asphalt (HMA) are not sufficiently accurate, especially with modified asphalts. It is doubtful that the E* predictive equations are accurate for CIR mixtures either. A thorough laboratory evaluation of the dynamic modulus properties of CIR mixtures is needed to evaluate the input parameters required by the M-EPDG.

Two proposed rehabilitation projects on US 412 in Beaver and Harper counties would provide an excellent opportunity to investigate the suitability of CIR with slurry crack injection as a rehabilitation technique for transverse cracked pavements in Oklahoma. The western portion of US 412 in Beaver County is scheduled for CIR with slurry crack injection to retard reflection cracking and the Harper County portion will receive a more conventional treatment of a fabric interlayer and HMA overlay. Both sections could be monitored to evaluate the effectiveness of retarding reflective cracking. The proposed CIR project on US 412 would provide an excellent opportunity to investigate the dynamic modulus properties of CIR mixtures and evaluate the appropriateness of the default dynamic modulus values.

OBJECTIVES

The objectives of this research project would be to evaluate the effectiveness of CIR with slurry crack injection to rehabilitate transverse cracked HMA pavements, to investigate the dynamic modulus properties of CIR mixtures and to evaluate the appropriateness of the predictive dynamic modulus values. If CIR is shown to be an effective rehabilitation procedure for transverse cracked HMA pavements, then a cost effective procedure for rehabilitation would be available to the Oklahoma department of transportation (ODOT). The improved performance of CIR over conventional fabric interlayer and thin HMA overlays would result in significant cost savings to ODOT.

WORK PLAN AND SCHEDULE

To accomplish the objectives of this study, the following two phase study is proposed. The first phase of the study would entail setting up test sections on both projects on US 412 and performing condition surveys of the test sections. The second phase would entail a laboratory evaluation of the dynamic modulus properties of CIR mixtures made from RAP obtained from the phase 1 testing. To meet the objectives of this study the following tasks are proposed.

Phase 1: Field Evaluation

Task 1: Establishment of Test Sections: Three test sections would be established on US 412 for detailed testing of the CIR and conventional overlay procedures. Test sections would be established by the Principal Investigator with input from ODOT personnel.

It is recommended that each test section be a minimum of 500 feet in length and contain a minimum of six transverse cracks. Two test sections would be established on the western portion of US 412 to evaluate CIR. One of the CIR test sections would have the cracks slurry injected prior to CIR and the second section would omit the crack slurry injection. A third test section would be established on the eastern section of US 412 that will receive the conventional fabric interlayer and HMA overlay.

Task 2: Performance Evaluation: A crack map of each test section would be prepared prior to construction. The location and severity of each crack would be evaluated using the protocols of the *SHRP Distress Identification Manual*. The three test sections would be monitored yearly for three years and again at five years to evaluate the effectiveness of the three procedures in reducing the occurrence of reflective cracking.

Task 3: Construction Monitoring and Mixture Sampling: During construction of the CIR on the western portion of US 412, samples of RAP would be obtained from each test section prior to the addition of any recycling agent. Samples of the recycling agent used, and any other additive used, would be obtained. Sufficient materials would be obtained to allow completion of phase 2 of this proposed project. The in-place density and moisture content of the CIR in each test section would be evaluated by ODOT.

Phase 2. Laboratory Evaluation of CIR on US 412

Task 1: Literature Review: The available literature would be reviewed to gain insight on current work regarding evaluation of dynamic modulus of HMA mixtures. Development of the test procedure is extensively covered in the draft final report of the M-EPDG and would not be the emphasis of the literature review. The emphasis of the literature review would be on recent work to gain insight as to the most efficient way to perform dynamic modulus testing.

Task 2: Equipment Purchase and Setup: A universal testing machine, test head fixtures, LVDTs and an environmental chamber are required for performing dynamic modulus. The purchase, set-up and familiarization of the test equipment and test procedures would be carried out under task 2.

Dynamic modulus testing requires three additional pieces of equipment, a Superpave Gyratory Compactor (SGC), a core drill and saw that can prepare the 100 mm diameter by 150 mm high test samples from the 150 mm diameter by 175 mm high SGC compacted test specimens. Oklahoma State University (OSU) has a core drill and saw that can trim the SGC compacted test specimens to the required test sample size, reducing equipment costs. OSU has a Troxler SGC which can not compact a sample to the required 175 mm height for dynamic modulus testing. Therefore, it is proposed that OSU swap its Troxler SGC for the ODOT Central Materials Laboratory Pine SGC for the duration of the proposed study. At the completion of the study the SGC compactors would be returned to each agency. OSU would be responsible for transporting the SGC compactors.

Task 3: CIR Mixture Evaluation: Once the equipment is purchased and set up, CIR mixture evaluation would commence. The objective of this task would be to evaluate the dynamic modulus properties of the CIR mix obtained from task 3 of phase 1. Three asphalt emulsions would be selected for evaluation with the RAP from US 412; the emulsion used on the project and two other commonly used emulsions. Hydrated lime is commonly used with CIR and the effect of lime on dynamic modulus would be evaluated on at least one emulsion as well. The dynamic modulus would be determined in accordance with AASHTO TP 62. The results would be used to develop dynamic modulus master curves for the CIR mixtures. The resulting master curves would be compared to the dynamic modulus values determined from the predictive equations for HMA found in the M-EPDG.

Task 4: Aging Effects: The M-EPDG has an environmental effects model where the aging of typical HMA mixtures is accounted for in the design procedure. It is doubtful that CIR mixtures undergo the same change in stiffness (aging) as conventional HMA mixtures. To evaluate aging effects, the CIR mixtures from task 3 of phase 2 would be subjected to the long-term oven-aging (LTOA) protocols developed from the SHRP program and the dynamic modulus testing would be repeated on the LTOA samples. The results for would be compared to evaluate the effects of aging on both HMA and CIR mixtures.

Task 5: Final Report: A final report would be prepared summarizing the significant findings from phase 1 and 2 of the study. The effectiveness of CIR with slurry crack injection would be evaluated. Recommendations for default dynamic modulus values for ODOT CIR mixtures for use in the M-EPDG would be provided.

BENEFITS

The benefits of implementation of the mechanistic-empirical procedures of the 2002 Design Guide are numerous and are adequately spelled out on the web page of the 2002 Design Guide at www.2002designguide.com. The specific benefits of completing the proposed research program are as follows:

1. The effectiveness of CIR with slurry crack injection for rehabilitation of transverse cracked pavements would be evaluated. A cost effective rehabilitation procedure for these pavements could be available.
2. Test equipment, test procedures and trained personnel would be available to ODOT for determination of dynamic modulus of paving mixtures.
3. Default dynamic modulus values for CIR mixtures in Oklahoma would be available.

CHAPTER 2

LITERATURE REVIEW

RECYCLING

Recycling of hot mix asphalt (HMA) has increased in popularity since the late 1970s. Many different reasons led to the increased demand and awareness of recycling. Probably the largest single factor was the oil embargo of the early 1970s and the subsequent increase in the price of asphalt cement. The increase in the price of asphalt made any reclaimed asphalt pavement (RAP) material a valuable asset. Before the oil embargo, the price of asphalt cement was so low that the cost of removing, stockpiling, and recycling old pavements was more than that for purchasing, mixing, and placing new material (1).

A second item that has had a great impact on recycling is the development of the milling machine. Prior to the development of the milling machine, old asphalt pavement had to be ripped from the roadway and then crushed prior to using. This process often required that the roadway be shut down for extended periods of time.

The milling machine can remove any amount of material desired and does not produce appreciable pollution since heat is not required. The material removed with a milling machine does not have to be crushed since it is fine enough immediately after being removed to recycle. A milled surface can also be opened to traffic temporarily until the overlay has been completed (1).

Cold in-place recycling is a subcategory of cold recycling, one of five broadly classified categories of recycling defined by the Asphalt Recycling and Reclaiming Association (ARRA) (2). These categories are 1) Cold Planning (CP), 2) Hot Recycling, 3) Hot In-Place Recycling (HIR), 4) Full Depth Reclamation (FDR) and 5) Cold Recycling (CR).

Cold Recycling (CR)

Cold Recycling consists of recycling HMA pavement without the application of heat during the recycling process. CR is classified into two sub-categories based on the process used. These processes are Cold Central Plant Recycling (CCPR) and Cold In-Place Recycling (CIR).

Cold Central Plant Recycling (CCPR)

Cold Central Plant Recycling is the process in which the asphalt recycling takes place in a central location using a stationary cold mix plant. The stationary plant can be either a specifically designed plant or a CIR train, minus the milling machine, set up in a stationary configuration. The CCPR mix can be used immediately or it can be stockpiled for later use in such applications as maintenance blade patching or pothole repair (2).

Cold In-Place Recycling (CIR)
ARRA defines Cold In-Place Recycling as a partial depth recycling process that rehabilitates the upper portion of an existing pavement, normally between 2 and 4 inches. The RAP material is obtained by milling, planning, or crushing the existing pavement. Virgin aggregate or recycling agent or both are added to the RAP material which is then laid and compacted (2,3).

In the CIR process the old pavement is milled then crushed and screened to size and ultimately mixed with a liquid recycling agent. New aggregate and other additives can be added, if needed. A paver following the CIR train lays the cold mix and the mix is then compacted as soon as the emulsion or recycling agent breaks and sets up, releasing the moisture from the mix (usually 15 to 90 minutes after placement) (2,4,5,6). The process is completed with the laying of a wearing surface over the CIR mix. A minimum HMA thickness of 1.5 inches is recommended, though low traffic volume roads sometimes are treated with chip seals (2,6).

CIR EQUIPMENT

CIR equipment can differ in size and sophistication. The equipment differs by the way the RAP is removed and sized, how the additives are added and mixed and how the mix is placed. Generally, CIR is carried out by single or multiple units called trains. The different kinds of CIR trains, which vary depending on their operation and size, are single, two unit and multiple unit trains (2,4,7).

Single-Unit Trains
The traditional single-unit train consists of a machine that mills the pavement to the specified depth and cross slope and blends the recycling additive. One of the unique features of the single-unit recycler is the use of a down-cutting milling head (most mills cut on the up cycle). The down-cutting milling head lets the operator control RAP size by adjusting the forward speed of the machine (2,4,6).

Figure 1 shows a setup of a single unit train. In a single pass the unit is capable of milling, adding recycling agent and paving the surface. Some of the single-unit trains, called super single-unit trains, are capable of milling the surface to be treated, screening the RAP, crushing any oversized material, blending the RAP with the recycling agent in the pugmill and placing the mixture onto the surface using a screed. Figure 2 shows a sectional view of a super single-unit CIR train.

The advantages of the single-unit train are simplicity of operation and high production capacity. The single-unit train may be preferred over a multi-unit train in urban areas and on roads with short turning radius, due to its shorter length. Single-unit CIR trains are less than 70 feet in length, compared to the multiple-unit CIR trains which can measure over 150 feet in length. (2).

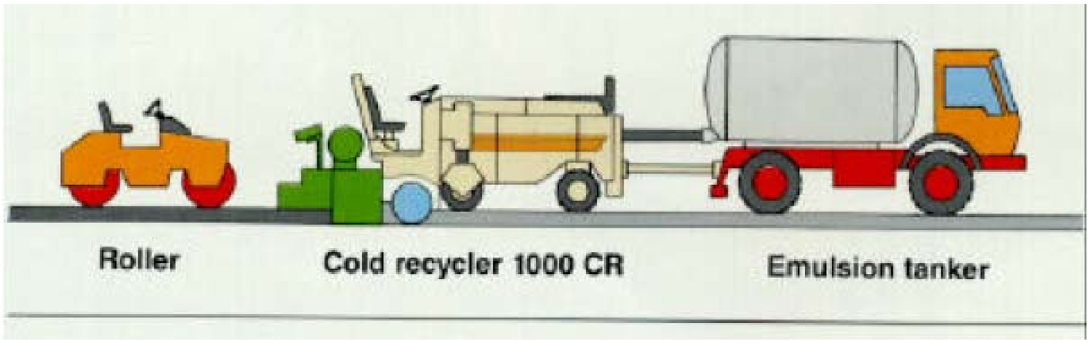

FIGURE 1 Traditional single-unit CIR train (8).

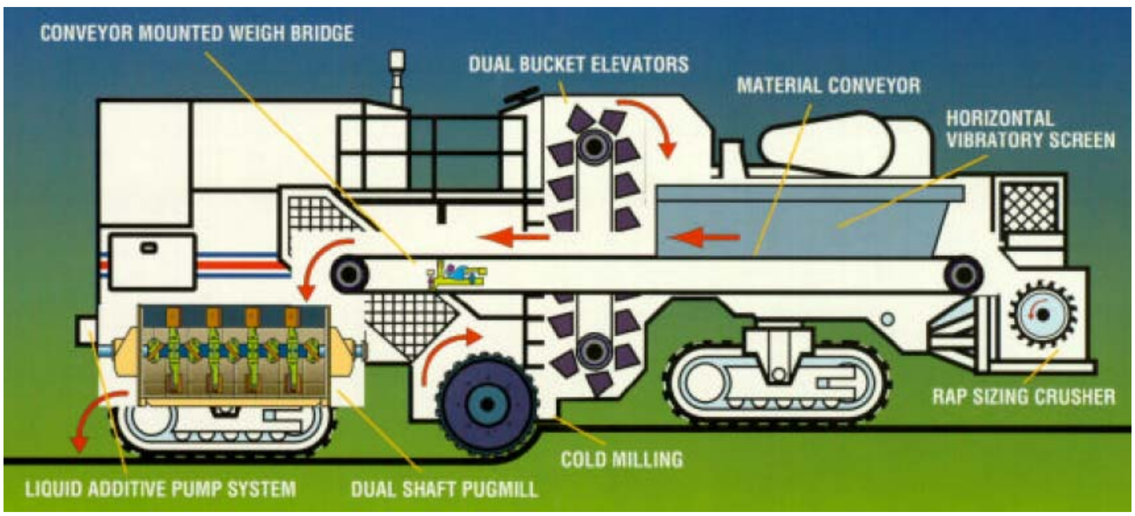

FIGURE 2 Super single-unit CIR train (8).

Two-Unit CIR Train

A two-unit CIR train usually consists of a large, full lane-width milling machine and a mix paver. The milling machine removes a portion of the old pavement and deposits the RAP into a mix-paver. A scalping screen can be used to remove oversize RAP. The RAP and the additives are mixed in a mix-paver to form a uniform mixture. The mix-paver has an in-feed belt with a belt scale and processing computer to accurately control the amount of recycling additive and modifier being added. The mixture is placed and pre-compacted by a screed which is automatically controlled. The two-unit train provides an intermediate to high degree of process control, with the liquid recycling additive being added based on the weight of the RAP, independent of the treatment volume and forward speed of the train (2). Figure 3 shows a two-unit CIR train.

FIGURE 3 Two-unit CIR train.

Multi-Unit CIR Trains

Multiple-unit CIR trains consists of a large milling machine which is capable of milling the full lane width, a screening and crushing unit which is mounted on a separate trailer unit and a trailer mounted pugmill mixer. The milling machine removes the RAP and has conveyer belts which move the millings to the crushing and screening unit. The sizing and crushing of the oversized RAP is performed in a separate screening/crushing unit. The screening unit has a set of vibrating screens which are stacked with larger sieve on the top. The crushed RAP from the milling machine is passed through the set of screens. The oversized pieces of the RAP are retained and put into a crushing machine for further crushing to the desired size.

The graded RAP is mixed with recycling additives in a separate pugmill unit. RAP in the pugmill is mixed uniformly. The amount of additive is calculated by computer, based on the mass of the RAP, and then added to the pugmill. The mixture is then laid in a windrow and placed with conventional HMA pavers equipped with a windrow pickup attachment. CIR mixes are compacted as the mixture begins to "break", turning from brown to black. Compaction is usually achieved with heavy pneumatic tired rollers followed by vibrating steel drum rollers in static mode to remove roller marks (2).

The multi-unit train provides the highest level of process control. The main advantages of the multi-unit train are high productivity and high process control. Most highway and interstate work is performed by multi-unit CIR trains. The major disadvantage of the multi-unit train is its length, which can make traffic control difficult in urban locations. In figure 4, a schematic of a multi-unit train is shown and figure 5 shows a multi-unit CIR train.

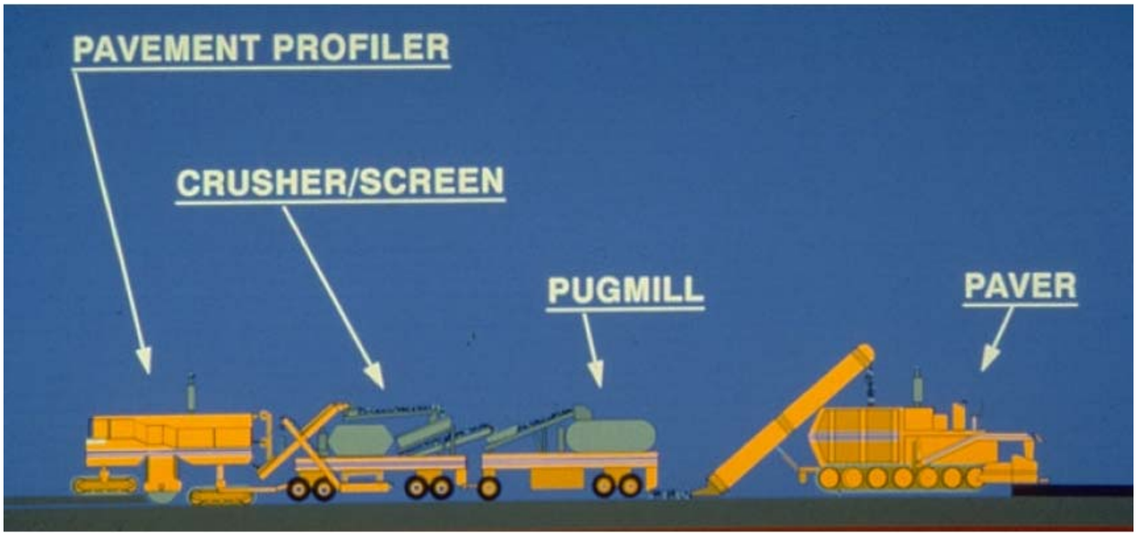

FIGURE 4 Schematic of a multi-unit CIR train (8).

FIGURE 5 Multi-unit CIR train (8).

ADDITIVES

The proper selection of additives to the reclaimed asphalt pavement plays a vital role in the proper performance of the CIR project.

Asphalt Emulsions

The most common additives for CIR are asphalt emulsions. Asphalt emulsions are classified according to the charge on the droplets and their setting or breaking time. Cationic emulsions have droplets which carry a positive charge and have a “C” in front of the setting time. Anionic emulsions have negatively charged droplets and do not have a prefix of “C”. Emulsion classification for setting times are rapid setting (RS), medium setting (MS), and slow setting (SS) (9).

Rapid and medium setting emulsions are rarely used with the CIR process because they do not allow for adequate mixing before breaking. High float (HFE) and high float medium setting (HFMS) asphalt emulsions are a special class of anionic medium setting asphalt emulsions. They have a gel structure in the asphalt residue which allows for thicker films on aggregate particles, giving better coating in some instances, such as under high temperatures. Slow setting (SS) asphalt emulsions work well with dense graded aggregates or aggregates with high fines content. Polymer modified versions of asphalt emulsions are used to improve early strength, resist rutting, and reduce thermal cracking. The more common asphalt emulsions for CIR are CSS-1, HFE and HFMS grades (9).

Rejuvenators

Rejuvenators are occasionally used as recycling additives in CIR. According to the ARRA (2), rejuvenators are products designed to restore original properties to aged asphalt cement by restoring the original ratio of asphaltenes to maltenes. Many rejuvenators are proprietary, making it difficult to offer a good generic description. However, many rejuvenators contain maltenes because their quantity is reduced by oxidation. Rejuvenators will retard the loss of surface fines and reduce the formation of additional cracks; however, they can also reduce pavement skid resistance for prolonged periods of time. Because of this, their use has generally been restricted to low volume, low speed roads and parking lots (2).

Chemical Additives

Type C fly ash, lime and Portland cement have been successfully utilized as recycling additives in CIR. These chemical additives can be used to improve early strength gain, increase rutting resistance, and improve moisture resistance of CIR mixtures containing rounded coarse aggregates or high percentages of natural sand. Typical hydrated lime and Portland cement contents used are 1 to 2 percent by weight of RAP. Fly ash contents in the range of 8 to 12 percent have been reported. Type C fly ash is applied dry by spreading in front of the recycling train. Portland cement and hydrated lime can be added dry or as slurry (10-13).

Cutback Asphalts

Cutback asphalt cements have been successfully utilized in the past as a recycling agent for CIR but are not currently recommended due to environmental and safety concerns. The flash point of some cutbacks can be at or below the CIR application temperatures (2).

MIX DESIGN

There is no nationally accepted method for the design of CIR and most of the agencies that use CIR have their own procedures (4). Several agencies have developed general mix design methods for CIR based on modifications to HMA procedures.

AASHTO-AGC Task Force 38

Task Force 38 reviewed several mix design procedures and recommended that the Marshall mix design method for HMA, with minor modifications, be adopted for CIR mixtures. Recommended modifications include compacting samples at lower temperatures and determining Marshall stability at reduced temperatures as well. Optimum emulsion content is the emulsified asphalt content (EAC) that meets or exceeds minimum stability requirements (5). Most other CIR mix design procedures have adopted the temperature recommendations of Task Force 38.

Modified Superpave Mix Design Methods

Lee (14) adopted the Task Force 38 procedures to Superpave volumetric procedures using the Superpave Gyratory Compactor (SGC). Lee showed that the optimum emulsion content (OEC) can be identified using Superpave procedures if changes in compaction temperatures are made.

Two additional studies (15,16) were performed to determine the CIR mix design compactive effort using the SGC. The studies determined the mix design compactive effort (N_{design}) that is needed to match the field density of CIR mixtures. The N_{design} compactive effort is generally believed to fall between 30 and 35 gyrations.

SemMaterials ReFlex® CIR Design Method

The ReFlex® system developed by SemMaterials (17) is a new mix design protocol and specially formulated emulsion for cold recycling. The system uses an engineered design approach with performance tests for raveling, rutting resistance (strength), thermal cracking resistance, and moisture susceptibility (stripping). Three different RAP gradations: coarse, medium, and fine are used in the design procedure and optimum asphalt emulsion content is determined by establishing the asphalt emulsion content that passes the four performance tests. The procedure also has a method to adjust asphalt emulsion contents in the field based on changes in the RAP gradation (17). This mix design procedure, or slight variations, has become the standard CIR mix design procedure for many agencies (17).

WHY RECYCLING?

Cold recycling is a proven method of pavement rehabilitation that has been successfully used on low volume roads to interstate highways. Distresses treated by CIR include raveling, bleeding, edge cracking, block cracking, slippage, longitudinal cracking, transverse or thermal cracking and poor ride quality caused by corrugations, swells or depressions. Environmental advantages to CIR include the elimination of material disposal, minimal energy consumption and greatly reduced material transportation costs. Limitations to CIR include pavement failures caused by wet, unstable bases or subgrade materials, heaving or swelling of underlying soils, deformations caused by high asphalt contents or fine graded aggregates, and pavements that exhibit stripping of the asphalt from the aggregate (2).

MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE (M-EPDG)

Need for the Design Guide

The various editions of the *AASHTO Guide for Design of Pavement Structures* have served well for several decades; nevertheless, many serious limitations exist for their continued use as the nation's primary pavement design procedures. Listed below are some of the major deficiencies of the existing design guide (18):

- Traffic loading deficiencies
- Rehabilitation deficiencies
- Climatic effects deficiencies
- Subgrade deficiencies
- Surface materials deficiencies
- Base course deficiencies
- Truck characterization deficiencies
- Construction and drainage deficiencies
- Design life deficiencies
- Performance deficiencies
- Reliability deficiencies

General Input Requirements

The *Guide for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures* (referred to hereinafter as M-EPDG) was developed to provide the highway community with a state-of-the-practice tool for design of new and rehabilitated pavement structures. The M-EPDG is a result of a large study sponsored by AASHTO in cooperation with the Federal Highway Administration and was conducted through the National Cooperative Highway Research Program (NCHRP) [NCHRP-1-37A]. The final product is design software and a user guide. The M-EPDG is based on comprehensive pavement design procedures that use existing mechanistic-empirical technologies. M-EPDG software is temporarily available for trial use on the web. The software can be downloaded from www.trb.org/mepdg. The software is described as a user oriented computational software package and contains documentation based on M-EPDG procedures (18). The M-EPDG employs common design parameters for traffic, subgrade, environment, and reliability for all pavement types (18).

Input parameters for the M-EPDG are grouped into five areas: project information, design information, traffic loadings, climatic data and structural data. Structural data is separated into two sections, one on structural layers and one on thermal cracking. The focus of this study is on the input data required in the *Layers* section for CIR mixtures (18).

Layers

The input requirement for asphalt layers uses a hierarchical approach with three levels of materials characterization. The first level provides the highest design reliability and each succeeding level is a drop in design reliability. Within each level there are three input screens, *Asphalt Mix, Asphalt Binder* and *Asphalt General*. Any level of reliability may be used with any layer in the pavement system. However, the same level of reliability is required for each input screen within a pavement layer (18).

Asphalt Mix Screen
The *Asphalt Mix* screen allows three levels of reliability; however, the required inputs are the same for reliability levels 2 and 3. For level 1 reliability, dynamic modulus is required at a minimum of three temperatures and three frequencies. One of the temperatures must be greater than 51.7°C (125°F). For level 2 and 3 reliability, the dynamic modulus is calculated using a predictive equation based on mix properties. The required mix properties for the *Asphalt Mix* screen are aggregate percent retained on the 3/4 inch, 3/8 inch and No. 4 sieves and the percent passing the No. 200 sieve (18).

Asphalt Binder Screen
The *Asphalt Binder* screen allows three levels of reliability; however, the required inputs are the same for reliability levels 1 and 2. For level 1 or 2 reliability, the shear modulus (G*) and phase angle (delta) for the binder are required from the dynamic shear rheometer (DSR) test. The DSR parameters are required at a minimum of three temperatures. For level 3 reliability, the grading of the asphalt binder is all that is required. The M-EPDG allows the use of PG graded binders, viscosity (AC) graded binders or penetration graded binders (18).

Asphalt General Screen
The *Asphalt General* screen allows three levels of reliability; however, the required inputs are the same for all three reliability levels. The *Asphalt General* screen is separated into four sections: *General, Poisson's Ratio, As Built Volumetric Properties* and *Thermal Properties*. The *General* section requires the reference temperature for development of the dynamic modulus master curve. The default value is 70°F but other temperatures may be entered. The *Poisson's Ratio* section allows the user to select the default value of 0.35 for asphalt, enter a user defined value or allow the software to calculate Poisson's ratio using a predictive equation. *As Built Volumetric Properties* include volume of binder effective (Vbe), air voids and compacted unit weight. Default values are 11.0%, 8.5% and 148 pcf, respectively. Required *Thermal Properties* are thermal conductivity and heat capacity. Either user defined or deafault values may be entered. Default values are 0.67 BTU/hr-ft-°F for thermal conductivity and 0.23 BTU/lb-°F for heat capacity (18).

CHAPTER 3

FIELD OBSERVATIONS

One of the objectives of this research project was to evaluate the effectiveness of CIR with slurry crack injection to rehabilitate transverse cracked HMA pavements. Two proposed rehabilitation projects on US 412 in Beaver and Harper counties provided an excellent opportunity to investigate the suitability of CIR with slurry crack injection as a rehabilitation technique for transverse cracked pavements in Oklahoma. The western portion of US 412 in Beaver County received CIR with slurry crack injection to retard reflection cracking and the Harper County portion received a more conventional treatment of a fabric interlayer and HMA overlay.

SITE DESCRIPTIONS

Beaver County Sites

The CIR project was located on US 412 in Beaver County, Oklahoma. The project began at the junction of US 83 and US 412 and proceeded west approximately 5.5 miles. The ODOT project number was NHY-017N(170)3R. The 20-year design equivalent single axle loads (ESALs) were 2.6 million. The average daily traffic (ADT) was reported as 1700 vehicles. Figure 6 shows project prior to CIR.

FIGURE 6 US 412 prior to CIR rehabilitation.

The rehabilitation project consisted of sealing the existing cracks by pressure injecting them with fly ash slurry. Figures 7-10 show the fly ash slurry injection procedure. The fly ash slurry was allowed to cure and then the top four inches of the pavement was cold in-place recycled. Recycling was accomplished using a multi-unit recycling train (figures 11 and 12). The RAP was screened to produce 100% passing a 31.5mm (1.25-inch) screen. The emulsified asphalt cement (EAC) was a CSS-1 Special (ReFlex®) applied at a rate of 2.0 percent by weight of the RAP. Water was applied at a rate of 3.0 percent by weight of RAP. A two inch HMA overlay consisting of an S-4 mix with PG 76-28 asphalt was placed over the CIR mix. Figure 13 shows the placement of the S-4 wearing surface.

FIGURE 7 Fly ash slurry crack injection on US 412.

FIGURE 8 Slurry injection equipment.

FIGURE 9 Pressure injecting transverse cracks with slurry.

FIGURE 10 Treated transverse crack.

FIGURE 11 Cold recycling US 412 with multi-unit CIR train.

FIGURE 12 Laydown of CIR mixture.

FIGURE 13 Placement of HMA overlay on US 412 Beaver, County.

Test Sections
Two 500 foot long test sections were selected on US 412 for evaluation. The western test section was located between stations 83+00 and 88+00 and the eastern test section between stations 88+00 and 93+00. One of the objectives of this study was to evaluate the effectiveness of the fly ash slurry injection procedure to retard reflective cracking. The eastern test section had the fly ash slurry injection procedure. The western test section did not utilize the fly ash slurry injection procedure.

Harper County Site

Cold in-place recycling is not a standard rehabilitation procedure for ODOT as it is for several of the surrounding state DOTs including Kansas, New Mexico and Nebraska. Therefore, the CIR test sections in Beaver County were compared to a more conventional rehabilitation procedure used by ODOT. The conventional rehabilitation project was located on US 412 in Harper County, Oklahoma. The project began at the eastern Beaver County line and proceeded east approximately 7.0 miles. The ODOT project number was NHY-017N(160)3R. The 20-year design equivalent single axle loads (ESALs) were 2.6 million and the average daily traffic (ADT) was reported as 1700 vehicles. This is the same traffic as reported on the Beaver County test sections.

The Harper County project also consisted of sealing the existing cracks by pressure injection with fly ash slurry. The procedure was the same as that used on the Beaver

County project. The fly ash slurry was allowed to cure and the top three inches of the pavement was cold milled. A two-foot wide, precoated fabric membrane strip was placed over the transverse cracks and sealed to the pavement by rolling with a pneumatic roller. Figure 14 shows typical distress for the Harper County site and figure 15 shows the test site after slurry crack injection but prior to rehabilitation. Figures 16-19 show the placement of the precoated fabric membrane strip. A three inch HMA overlay consisting of an S-4 mix with PG 76-28 asphalt was placed over the milled pavement.

Test Section

A third 500-foot long test section was established on US 412 in Harper County. The test section began at the intersection of NS 177 Road and proceeded west for 500 feet along US 412.

FIGURE 14 Typical distress on US-412 Harper, County.

FIGURE 15 US 412 Harper, County after slurry crack injection.

FIGURE 16 Installation of precoated fabric membrane strip.

FIGURE 17 Seating precoated fabric membrane strip with pneumatic roller.

FIGURE 18 Seated precoated fabric membrane strip.

Figure 19 Installation of precoated fabric membrane strips.

CHAPTER 4

LABORATORY TEST PROCEDURES

INTRODUCTION

One of objectives of the study was to determine the dynamic modulus of laboratory prepared CIR mixes and compare the laboratory dynamic modulus values with the predicted dynamic modulus values in the M-EPDG. One RAP and three asphalt emulsions were used. The dynamic modulus values were compared based on emulsion type to determine if emulsion had a significant effect on dynamic modulus.

MATERIALS

Reclaimed Asphalt Pavement (RAP)

ODOT personnel obtained samples of RAP from the rehabilitation project on US 412 in Beaver County. RAP was obtained without the emulsified asphalt cement (EAC). The gradation of the RAP, as received, was determined in general accordance with AASHTO T 27. The entire amount of RAP was placed in large flat pans and heated in a forced draft oven at 40°C (104°F) for 24 hours to remove surface moisture. The material was then sieved over a 38.1-mm (1.5-inch) sieve through 2.36-mm (No. 8) sieve, inclusive, and the material separated into sizes for batching. To determine the gradation of the RAP through the No. 200 sieve (0.075-mm), two 1,000-g samples of the material retained in the pan (passing No. 8 (1.18 mm) were sieved over the No. 8 sieve through the No. 200 sieve, inclusive. The gradation of the entire RAP was then calculated.

Two 2,000-g samples of the RAP from each site were batched to the as received gradation and the physical properties determined. The theoretical maximum density (Gmm) was determined in accordance with AASHTO T 209. Next, the asphalt content was determined using the ignition furnace in accordance with AASHTO T 308. The gradation of the recovered aggregate was determined in accordance with AASHTO T 30. The crushed face count of the recovered coarse aggregate was determined in accordance with ASTM D 5821. The fine aggregate angularity of the recovered aggregate was determined in accordance with AASHTO T 304, Method A. The gradation of the RAP and extracted aggregate are shown in table 1 and presented graphically in figure 20.

To determine the theoretical maximum specific gravity (G_{mm}) of the CIR mixtures, two 2,000 gram samples of RAP were batched to the as received gradation. The samples were mixed with Reflex emulsion at the job mix formula emulsion content and mix water content reported for the US 412 project. The mixed samples were cured and tested for G_{mm} in accordance with AASHTO T 209. The average of the two results was used for further calculations. The average G_{mm} for the RAP was 2.406. The same Gmm was used for subsequent samples because all mixtures were made at the same residual asphalt content and the specific gravity of the asphalt cement in each EAC was unknown.

TABLE 1. RAP and Recovered Aggregate Gradations

Sieve Size	RAP	Aggregate
	Percent Passing	
1"	100	
3/4"	97	100
1/2"	80	98
3/8"	66	94
No. 4	39	76
No. 8	25	63
No. 16	17	52
No. 30	10	39
No. 50	5	27
No. 100	2	16
No. 200	0.7	10.4
Pb (%)	6.8	N/A

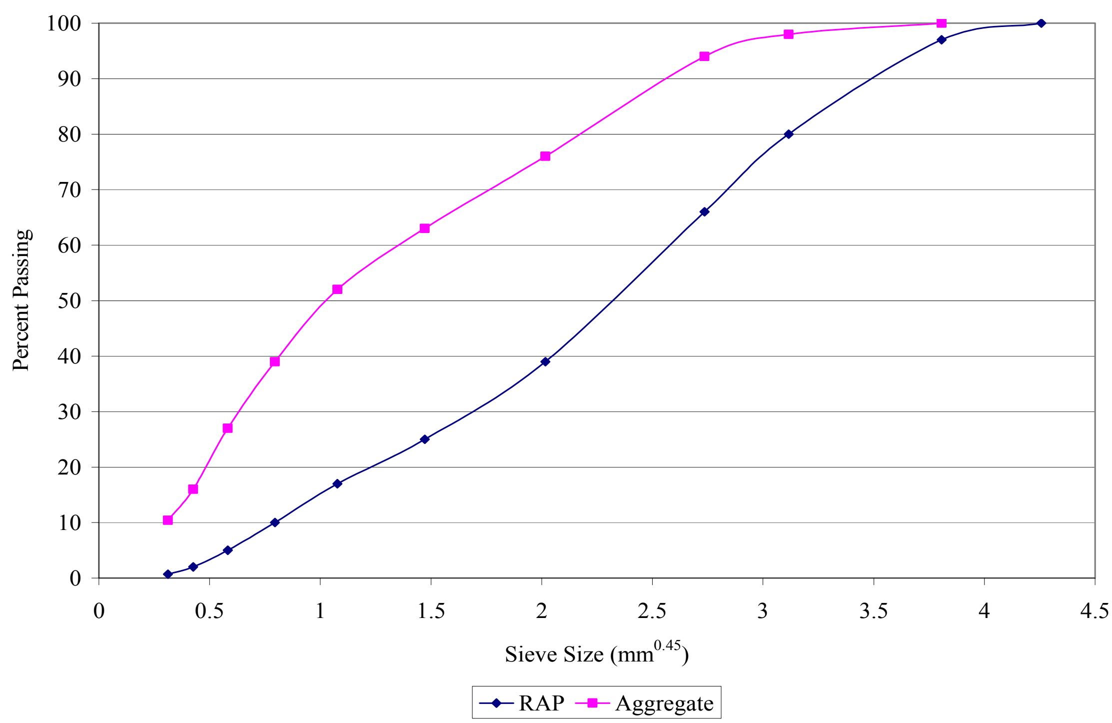

FIGURE 20 Grain-size distribution curves for RAP and extracted aggregate.

Asphalt Emulsions

The asphalt emulsion used on the Beaver County project was a specially formulated CSS-1 asphalt emulsion. The emulsion was supplied by SemMaterials and is generally referred to by its trade name, ReFlex®. Two other emulsions that are routinely used in the CIR process, SCC-1h and HFE 150P, were selected for comparison. CSS-1h is a popular CIR recycling agent in the region and the New Mexico DOT routinely uses an HFE 150P in their CIR work. The CSS-1h was supplied by Vance Brothers, Kansas City, Missouri. The HFE 150P was supplied by the New Mexico Department of Transportation.

The job-mix formula emulsion and total liquids contents on the Beaver County project were for ReFlex® emulsion. To make similar CIR mixtures with the other emulsions, the residual asphalt content of the CIR mixtures was held constant. Adjustments were made in the mix water added to keep the total liquids in the mix constant. The residual asphalt cement content of the ReFlex® and CSS-1h emulsions were available from the suppliers. The residual asphalt content of the HFE 150P was determined using the Colorado Department of Transportation's quick procedure (19). The residual asphalt cement content of the emulsions is shown in table 2.

TABLE 2. Residual Asphalt Cement Content of Emulsions

Emulsion	Residual AC (%)
Reflex	65
CSS-1h	55
HFE 150P	72

Lime

Lime, either hydrated or as slurry, is often used with CSS-1 asphalt emulsion in CIR. Typical solids (hydrated lime) contents are one percent, based on the dry mass of the RAP. Lime is added to increase initial stiffness and to increase resistance to moisture induced damage. Lime was used with the CSS-1h to produce a fourth recycling agent for evaluation.

Pebble quicklime, obtained from Brown and Brown Contractors, Inc., was used to make hydrated or slaked lime slurry. The quicklime was added to the CSS-1h mixtures in the form of slaked lime slurry with 30-35% solids. Three percent of this slurry, based on the dry mass of the RAP, was added to the CIR mixtures to provide the same amount of calcium as 1% hydrated lime.

Conventional HMA Mix

In order to compare CIR with conventional HMA, the aggregates used for the surface mix on the Beaver County project were sampled for dynamic modulus testing. The mixture was an S-4 mix made with PG 76-28 asphalt cement. The relevant mix design information is shown below (table 3).

TABLE 3. S-4 Mix Design

Pit	8104	8104	3504		
% in Blend	25	60	15	ODOT	
Material	#57 Chips	1/4" Chips	Man. Sand	S-4 Spec.	Combined Blend
Sieve Size	% Passing				
3/4"	100			100	100
1/2"	65			90-100	91
3/8"	35	100	100	< 90	84
No 4	8	94	99		73
No 8	5	64	86	34-58	53
No 16	4	46	60		38
No 30	4	34	29		26
No 50	3	25	10		17
No 100	3	16	3		11.0
No 200	1.5	9.0	2	2-10	6.1
Pb (%)				4.6 min.	5.4
F.A.A. %U				40 min.	45.0
Sand Equivalent				40 min.	81
L A Abrasion				40 max	25.5
Durability				40 min.	85.0
Insol. Residue				30 min	94.6
Fractured Faces				75/75 min	96/91

TEST PLAN

The main objective of this project was to evaluate the dynamic modulus properties of CIR mixtures made with different recycling agents. The dynamic modulus of CIR mixtures made with one RAP and four recycling agents was determined in accordance with AASHTO TP 62 (20). The results were used to develop dynamic modulus master curves for the CIR mixtures. The resulting master curves were compared to dynamic modulus values calculated from the predictive equations for HMA found in the M-EPDG.

The M-EPDG has an environmental effects model where the aging of typical HMA mixtures is accounted for in the design procedure. It is doubtful that CIR mixtures undergo the same change in stiffness (aging) as conventional HMA mixtures. To evaluate aging effects, the CIR mixtures and the HMA mixture used as the surface mix for the Beaver County project were subjected to the long-term oven-aging (LTOA) protocols developed from the SHRP research program. After LTOA, the dynamic modulus testing was repeated. The change in dynamic modulus due to LTOA for CIR mixtures was compared to the change in dynamic modulus due to LTOA for the HMA mixtures.

Preparation of Dynamic Modulus Test Specimen

Samples for dynamic modulus testing were prepared by mixing the RAP with four different recycling agents. The four recycling agents were a specially formulated CSS-1 asphalt emulsion (ReFlex®), HFE 150P, CSS-1h and CSS-1h with one percent hydrated lime added as slaked lime slurry. The samples were prepared in general accordance with the requirements of AASHTO TP 62 (20). Adjustments to AASHTO TP 62 were required to accommodate CIR mixture. The Kansas DOT procedure for making CIR samples was followed (21). The required modifications to AASHTO TP 62 and the mixing and compaction procedures for making CIR samples are discussed below.

Sample Requirements

The AASHTO TP 62 requirements for dynamic modulus test samples are provided in table 4. The requirements are for HMA samples. Dynamic modulus testing requires a 150 mm high by 100 mm diameter sample, of a target air void content, be cored from a 175 mm high by 150 mm diameter sample. There is no simple conversion factor for compaction of a 175 mm high, 150 mm diameter SGC compacted sample to a cored dynamic modulus sample with a given target air void content. The two samples will not have the same VTM due to the density gradient present in SGC compacted samples. A trial and error procedure is required to determine the density or void content of the larger sample required to produce a cored and sawed test sample of the intended void content.

Recommended target air void contents for HMA samples are 4-7%. CIR samples can not be treated like HMA samples because of the increased viscosity of the asphalt cement due to the lower compaction temperatures of CIR mixture. For this project, the CIR test samples were compacted to the field unit weight reported by ODOT. This worked out to be a void content of 12 ± 1 % VTM. After several trials, it was determined that a 175 mm high by 150 mm diameter sample compacted to 15 ± 1% VTM would yield a dynamic modulus test sample of the target 12 ± 1% void content.

Batching

A 6,113 gram batch of RAP, batched to the as received gradation, was required to produce a 175 mm high by 150 mm diameter test specimen with 15 ± 1% VTM. When the compacted sample was cored to 100 mm diameter and sawed to the required sample height of 150 mm, the target void content of 12 ± 1% VTM was obtained.

TABLE 4. Criteria for Acceptance of Dynamic Modulus Test Specimen (20)

Criterion Items	Requirements
Size	Average diameter between 100 mm and 104 mm. Average height between 147.5 mm and 152.5 mm.
Gyratory Specimens	Prepare 175 mm high specimens to required air void content (AASHTO T 312).
Coring	Core the nominal 100 mm diameter test specimens from the center of the gyratory specimen. Check the test specimen is cylindrical with sides that are smooth parallel and free from steps, ridges and grooves.
Diameter	The standard deviation should not be greater than 2.5 mm.
End Preparation	The specimen ends shall have a cut surface waviness height within a tolerance of ± 0.05 mm across diameter. The specimen end shall not depart from perpendicular to the axis of the specimen by more than 1 degree.
Air Void Content	The test specimen should be within ± 1.0 percent of the target air voids.
Replicates	For three LVDT's, two replicates with a estimated limit of accuracy of 13.1 percent.
Sample Storage	Wrap specimens in polyethylene and store in environmentally protected storage between 5 and 26.7° C (40 and 80° F) and be stored no more than two weeks prior to testing.

Emulsion Content

The ReFlex® emulsion and mix water content on US-412, as reported by ODOT, was 3.0% and 2.0%, respectively, for a total liquids content of 5.0%. In the lab, these mix water and emulsion contents were maintained for the ReFlex® samples. The emulsion contents for the CSS–1h and HFE 150P emulsions were determined by keeping the residual asphalt contents the same as the ReFlex® samples and adjusting the mix water content to maintain 5 % total liquids. The percentage of mix water and emulsion used for each emulsion is shown in table 5. For the CSS-1h + lime samples, slaked lime slurry was added at 3.3% by dry mass of the RAP to result in 1% hydrated lime being added to the mix. The lime slurry replaced all of the mix water.

TABLE 5. Percentages of Water and Emulsions Based on Dry Weight of RAP

	Reflex	CSS-1h	HFE 150P
Residual AC (%)	65	55	72
EAC (%)	3.00	3.55	2.71
Water (%)	2.00	1.45	2.29
Total Liquids (%)	5.00	5.00	5.00

Mixing

All samples were mixed in a bucket mixer (figure 21). The emulsion was stirred until thoroughly mixed and then heated to 54°C for 30 minutes before mixing. The RAP was not heated. For mixing, half of the mix water was added to the RAP and mixed for one minute. The remainder of the mix water and the asphalt emulsion was then added and the sample mixed for an additional 1.5 minutes. Lime samples were mixed in the same manner except that all of the lime slurry was added and mixed for one minute then the emulsion was added and mixed for 1.5 minutes.

Compaction

The samples were compacted in a 150 mm diameter mold to a height of 175 mm using a Pine Superpave gyratory compactor. To produce the required 175 mm high by 150 mm diameter sample with a void content of 15 ± 1 %, 6,113 grams of RAP were required. The required amount of RAP was more than would fit into the compaction mold in one lift. Therefore, it was necessary to lightly tamp down the material with a tamping rod to get the mixture into the mold. Thirty to 40 gyrations were typically required to reach a height of 175 mm.

Curing

After compaction, the samples were immediately extruded from the compaction mold, labeled and placed in a pan. The samples were placed in an oven at 60°C for a minimum of 16 hours. After 1 to 2 hours of curing, the paper discs on the top and bottom of the samples were removed. After 16 hours, the samples were checked every 2 hours until the mass loss was less than 0.05%, up to a maximum of 48 hours of oven curing (21).

FIGURE 21 Bucket mixer used for mixing CIR samples.

Coring & Sawing

After curing, the samples were allowed to cool to room temperature. When cooled, the samples were cored and sawed to obtain a 150 mm tall by 100 mm diameter test sample with 12 ± 1 % voids. The samples were cored using a diamond studded core barrel to obtain a diameter of 100 mm (figure 22). The cored samples were then sawed to obtain the 150 mm height (figure 23). The cored and sawed samples were washed to eliminate all loose debris. After cleaning, the samples were tested for bulk specific gravity in general accordance with AASHTO T 166. The dry mass was determined by placing the samples at room temperature under a fan over night and allowed to dry to constant mass. From the bulk specific gravity and the calculated Gmm for each recycling agent, the air void content was determined.

The CIR test samples were next checked for conformance to the sample requirements of AASHTO TP 62. The criterion for acceptance of the samples was listed in table 4. Samples which met all criteria were fixed with six steel studs to hold three LVDT's. The LVDT's had a gauge length of 4 inches. Care was taken to precisely position the studs 4 inches apart from each other and 2 inches from the center of the sample. Once the epoxy was dry and the studs were firmly attached to the sample they were ready for testing. Figure 24 shows a sample prepared for dynamic modulus testing.

FIGURE 22 Sample being cored to required test diameter.

FIGURE 23 Sample is sawed to obtain parallel faces.

FIGURE 24 Test specimens for dynamic modulus testing.

Testing

Dynamic Modulus

The dynamic modulus of the CIR samples was determined in accordance with AASHTO TP 62 (20). The procedure is briefly explained in figure 25. The test parameters are provided in table 6.

Conventional HMA samples were prepared from the aggregates obtained for the S-4 mix used as the overlay on US 412 in Beaver County. HMA samples were mixed to the job mix formula asphalt content of 5.4% and compacted to 6 ± 1.0 % air voids. Samples were made with PG 76-28, PG 70-28 and PG 64-22 asphalt cements. All HMA samples were made and tested for dynamic modulus in accordance with AASHTO TP 62.

Aging Effects

One of the objectives of this study was to compare the effects of oven aging on dynamic modulus of CIR to HMA samples. Additional dynamic modulus test samples were placed in a forced-draft oven for 120 ± 0.5 hours at 85 ± 3°C in accordance with AASHTO R 30-02, section 7.3.4 (22). The samples were allowed to cool and then tested for dynamic modulus in accordance with AASHTO TP 62. The results were compared to the conventional dynamic modulus results.

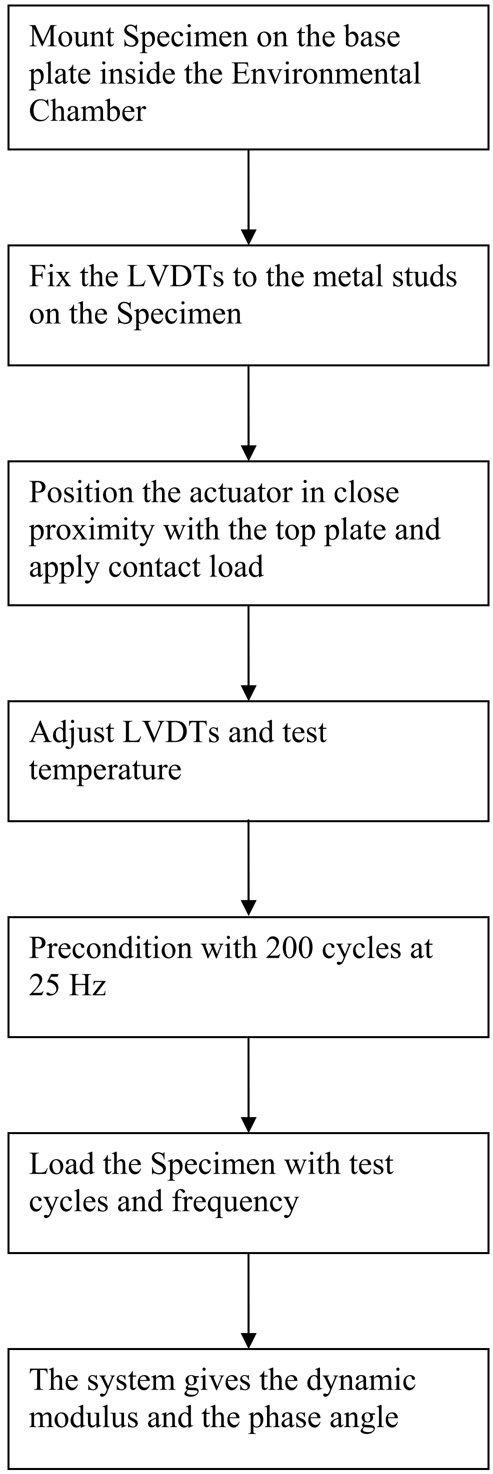

FIGURE 25 Test procedure for dynamic modulus of CIR samples.

TABLE 6. Test Parameters for Dynamic Modulus Test (20)

Test Parameters	Values		
Frequencies	25, 10, 5, 1, 0.5, 0.1 Hz		
Temperature	4.4°, 21.1°, 37.8° and 54.4°C (40°, 70°, 100° and 130° F)		
Equilibrium Times	Specimen Temperature,°C (°F)	Time from room temperature, hrs 25°C (77°F)	Time from previous test temperature, hrs
	4.4 (40)	Overnight	4 hrs or overnight
	21.1 (70)	1	3
	37.8 (100)	2	2
	54.4 (130)	3	1
Contact Load	5 percent of the test load		
Axial Strains	Between 50 to 150 microstrain		
Dynamic load range	Depends on the specimen stiffness and ranges between 2 and 400 psi		
Load at Test Frequency *	At 4.4° C (40° F): 100 to 200 psi		
	At 21.1° C (70° F): 50 to 100 psi		
	At 37.8° C (100° F): 20 to 50 psi		
	At 54.4° C (130° F): 5 to 10 psi		
Preconditioning	With 200 cycles at 25Hz		
Cycles	At 25Hz: 200 cycles		
	At 10Hz: 200 cycles		
	At 5Hz: 100 cycles		
	At 1Hz: 20 cycles		
	At 0.5Hz: 15 cycles		
	At 0.1Hz: 15 cycles		

* The load should be adjusted to obtain axial strains between 50 and 150 microstrains.

CHAPTER 5

TEST RESULTS

FIELD MEASUREMENTS

Prior to rehabilitation and yearly thereafter, distress maps were made for each 500 foot test section. Distresses measured were rutting; longitudinal cracking, both wheel path and non wheel path; transverse cracks and patching. The results of the distress measurements are shown in table 7.

As shown from the original distress surveys, the two test sections on the Beaver County project showed more distress than the Harper County test section. The two Beaver County test sections had similar levels of distress with the main distresses being longitudinal wheel path cracking and thermally induced transverse cracking. Longitudinal wheel path cracking is often considered to be top down fatigue cracking. The Harper County test section did not exhibit as much thermally induced transverse cracking and had little longitudinal wheel path cracking. The major distress at the Harper County site was longitudinal joint cracking that ran the entire length of the project. Rutting was minor at each test site, less than 1/4 inch.

A second survey was made in September 2005. The results are shown in table 7. A small patch, approximately 16 ft^2 was found in the eastern (slurry) test section in Beaver County. No distress was evident in the western (no slurry) test section. Approximately 100 feet of non wheel path longitudinal cracking was found in the Harper County test section. Rut depths were not measured at any of the test sections as rutting was minor, less than 1/4 inch.

A third survey was made in September 2006. The results are shown in table 7. A considerable amount of longitudinal wheel path cracking, over 365 feet, was found in both of the Beaver County test sections. The majority of the wheel path cracking was in the outer wheel path. One 1/4 pavement width transverse crack had developed in the eastern (slurry) test section. No transverse cracks were observed in the western (non slurry) test section. At the Harper County test section the longitudinal joint cracking had increased from 100 feet to 375 feet. One full pavement width and two 1/2 pavement width transverse cracks had appeared. Rut depths were not measured at any of the test sections as rutting was observed to be minor, less than 1/4 inch.

Summary

CIR is considered a procedure to rehabilitate transverse cracking. It is not generally considered a structural repair and additional structure, if necessary, must come from the thickness of the HMA overlay. Although two years is not sufficient time to evaluate the effectiveness of CIR to mitigate reflective cracking, there is less transverse cracking in

TABLE 7. Summary of Distress Surveys

Distress	Beaver Co. Eastern Slurry	Beaver Co. Western No Slurry	Harper Co. Slurry
Original Condition Spring 2004			
Longitudinal Cracking (ft)			
Wheelpath	226	231	82
Non-Wheelpath	87	20	500
Transverse Cracking (number)			
Full Width	16	12	5
3/4 Width	3	5	0
1/2 Width	9	4	8
1/4 Width	3	0	2
Patching (ft^2)	0	0	0
Rutting (in.)	< 1/4	<1/4	<1/4
September 2005			
Longitudinal Cracking (ft)			
Wheelpath	0	0	0
Non-Wheelpath	0	0	100
Transverse Cracking (number)			
Full Width	0	0	0
3/4 Width	0	0	0
1/2 Width	0	0	0
1/4 Width	0	0	0
Patching (ft^2)	16	0	0
Rutting (in)	<1/4	<1/4	<1/4
September 2006			
Longitudinal Cracking (ft)			
Wheelpath	367	377	0
Non-Wheelpath	0	0	375
Transverse Cracking (number)			
Full Width	0	0	1
3/4 Width	0	0	0
1/2 Width	0	0	2
1/4 Width	1	0	0
Patching (ft^2)	16	0	0
Rutting (in)	<1/4	<1/4	<1/4

the Beaver County test sections than the Harper County test section, even though the Beaver County sections originally had more transverse cracking. There is no practical difference in performance at this time between the eastern (slurry) and western (non slurry) test sections. The longitudinal joint cracking at the Harper County test section could be a result of poor workmanship in placing the HMA overlay and not be related to the crack repair treatment.

The wheel path cracking in the Beaver County test sections warrant further study to determine if the cracking is top down fatigue cracking or reflective cracking. Top down fatigue cracking is suspected. CIR can be a rather soft or flexible material. A two inch thick HMA overlay constructed with PG 76-28 asphalt can be considered a thin stiff layer over a soft flexible layer. Placing a thin stiff layer over a thick soft layer has been reported to lead to poor performance in the form of top down fatigue cracking (23).

DYNAMIC MODULUS TEST RESULTS

CIR Samples

Conventional Testing

The main objective of this project was to obtain typical dynamic modulus values for CIR mixture for use in the M-EPDG. RAP and emulsion were obtained from a CIR project on the western portion of US 412 in Beaver County. Two additional emulsions and lime were obtained. Test samples were prepared using the RAP from US 412 with four combinations of the emulsions and lime. The test samples were prepared and tested in accordance with AASHTO TP 62. The dynamic modulus of each sample was determined and the effect of emulsion on dynamic modulus evaluated. The measured dynamic modulus values were compared to the calculated dynamic modulus values wising the M-EPDG predictive equation. Finally, the effect of different emulsions on pavement thickness was evaluated using the M-EPDG.

The AASHTO TP 62 test protocol requires testing at -10° C (14°F). With OSU's test apparatus, samples could not be easily tested at -10° C (14°F) due to accumulation of frost in the test chamber. When changing from one test sample to another the environmental chamber door must be opened. When the door is opened, warm moist air mixes with the cold chamber air causing moisture to collect on metal surfaces of the test chamber and test specimen. At -10°C (14°F), significant frost build-up can result making it very difficult and time consuming to perform the test. The M-EPDG does not require dynamic modulus testing at -10°C (14°F) even though it is listed as a recommended test temperature in AASHTO TP 62. The M-EPDG only requires dynamic modulus values at three temperatures for Level 1 analysis, one less than 7°C (45°F), one between 7°C and 52°C (45°F - 125°F) and one greater than 52°C (125°F) (18). After only a few attempts, testing at -10°C was discontinued.

At the highest test temperature, 54.4°C (130°F), problems were encountered with repeatability of the strain measurements within each test frequency. Several test samples were damaged due to excessive strain. The problem was eventually traced to insufficient sensitivity of the 10-kip load cell at the low loads required at elevated test temperatures. It was later corrected by the purchase of a 2-kip load cell. Samples could not be retested due to the limited quantity of RAP available and results at 54.4°C were not available for some samples. Results from the conventional dynamic modulus testing are provided in tables 8-11.

Oven Aged Dynamic Modulus

Additional samples were prepared and tested for dynamic modulus after the long-term oven-aging procedure of AASHTO R 30-02 section 7.3.4 (22). The results are also shown in tables 8-11.

TABLE 8. Dynamic Modulus for Samples Prepared with ReFlex® Emulsion

Test Temp. (C)	Frequency (Hz)	Dynamic Modulus (psi)			
		Conventional		LTOA	
		Sample 1	Sample 2	Sample 1	Sample 2
4.4	25	863,500	907,981	967,664	1,476,076
	10	793,072	868,540	889,063	1,349,939
	5	719,848	790,864	803,663	1,223,660
	1	549,311	611,389	607,007	983,844
	0.5	494,693	550,399	534,066	882,598
	0.1	378,596	418,034	417,574	630,853
21.1	25	468,193	467,638	558,491	739,307
	10	404,601	392,505	502,527	697,229
	5	335,638	327,665	427,171	594,383
	1	222,871	212,830	280,654	405,652
	0.5	187,392	177,024	238,139	344,122
	0.1	129,229	124,899	164,990	236,541
37.8	25	209,876	240,229	311,063	380,457
	10	147,115	148,995	213,131	285,973
	5	116,352	111,739	176,083	239,502
	1	70,777	78,173	107,861	151,846
	0.5	45,131	59,926	88,291	124,678
	0.1	N/T	N/T	81,711	84,745
54.4	25	N/T	N/T	115,990	160,659
	10	N/T	N/T	75,033	106,456
	5	N/T	N/T	61,154	88,508
	1	N/T	N/T	38,034	54,986
	0.5	N/T	N/T	30,303	46,582
	0.1	N/T	N/T	19,199	30,553

TABLE 9. Dynamic Modulus for Samples Prepared with CSS – 1h

Test Temp. (C)	Frequency (Hz)	Dynamic Modulus (psi)			
		Conventional		LTOA	
		Sample 1	Sample 2	Sample 1	Sample 2
4.4	25	957,585	1,065,514	N/T	N/T
	10	877,498	1,023,436	N/T	N/T
	5	801,942	917,607	N/T	N/T
	1	627,014	715,094	N/T	N/T
	0.5	563,247	641,563	N/T	N/T
	0.1	436,626	494,033	N/T	N/T
21.1	25	547,109	766,988	N/T	N/T
	10	458,460	611,676	N/T	N/T
	5	386,915	496,709	N/T	N/T
	1	250,693	318,907	N/T	N/T
	0.5	203,647	253,565	N/T	N/T
	0.1	133,546	167,652	N/T	N/T
37.8	25	308,067	256,034	N/T	N/T
	10	185,093	168,303	N/T	N/T
	5	140,080	130,248	N/T	N/T
	1	84,859	81,665	N/T	N/T
	0.5	66,527	66,517	N/T	N/T
	0.1	N/T	67,928	N/T	N/T
54.4	25	N/T	N/T	N/T	N/T
	10	N/T	N/T	N/T	N/T
	5	N/T	N/T	N/T	N/T
	1	N/T	N/T	N/T	N/T
	0.5	N/T	N/T	N/T	N/T
	0.1	N/T	N/T	N/T	N/T

TABLE 10. Dynamic Modulus for Samples Prepared with CSS – 1h + Lime

Test Temp. (C)	Frequency (Hz)	Dynamic Modulus (psi)			
		Conventional		LTOA	
		Sample 1	Sample 2	Sample 1	Sample 2
4.4	25	1,341,448	1,462,943	1,574,992	1,531,665
	10	1,298,784	1,363,747	1,432,739	1,402,536
	5	1,216,269	1,278,824	1,316,481	1,283,163
	1	1,016,050	1,076,058	1,106,760	1,040,301
	0.5	940,586	1,002,879	1,023,809	932,176
	0.1	761,391	816,113	833,146	694,872
21.1	25	788,497	954,739	897,770	1,025,160
	10	742,074	871,825	897,538	970,695
	5	648,761	762,499	775,633	855,293
	1	457,786	554,572	558,323	619,326
	0.5	400,590	476,392	482,285	539,337
	0.1	296,027	332,223	343,985	381,676
37.8	25	657,462	473,953	457,445	613,245
	10	364,400	328,642	365,689	466,916
	5	274,119	276,368	307,912	384,021
	1	170,400	178,529	201,589	237,740
	0.5	143,043	146,876	171,173	202,701
	0.1	96,228	96,163	120,320	138,477
54.4	25	N/T	N/T	177,014	280,474
	10	N/T	N/T	135,487	212,408
	5	N/T	N/T	114,327	190,937
	1	N/T	N/T	69,633	111,430
	0.5	N/T	N/T	54,921	89,900
	0.1	N/T	N/T	43,445	64,825

TABLE 11. Dynamic Modulus for Samples Prepared with HFE 150P

Test Temp. (C)	Frequency (Hz)	Dynamic Modulus (psi) Conventional Sample 1	Conventional Sample 2	LTOA Sample 1	LTOA Sample 2
4.4	25	1,373,061	1,029,089	1,229,519	1,223,880
	10	1,298,416	965,219	1,150,789	1,152,988
	5	1,169,526	896,088	1,066,187	1,072,733
	1	919,560	717,806	857,441	879,761
	0.5	833,857	652,000	794,056	818,430
	0.1	648,841	513,803	646,393	681,263
21.1	25	595,883	557,936	659,674	740,185
	10	492,334	466,930	551,234	644,171
	5	424,817	390,146	484,069	567,853
	1	284,121	255,358	342,757	390,574
	0.5	243,267	218,099	290,265	333,990
	0.1	176,855	163,468	202,155	235,693
37.8	25	251,286	224,751	331,736	311,722
	10	206,213	188,768	276,762	271,054
	5	173,096	155,597	231,378	227,848
	1	117,213	99,409	144,876	139,922
	0.5	98,498	84,583	118,679	112,845
	0.1	66,157	60,055	81,100	74,709
54.4	25	110,558	84,378	167,480	157,437
	10	100,189	69,632	99,616	95,151
	5	73,385	56,095	95,958	83,680
	1	39,864	34,734	49,058	44,185
	0.5	36,090	30,443	41,843	36,800
	0.1	25,413	22,609	30,305	27,114

HMA Samples

The aggregates used for the surface mix on US 412 in Beaver County were sampled for dynamic modulus testing. Samples were prepared to the job mix formula gradation and optimum asphalt content using PG 76-28, PG 70-28 and PG 64-22 asphalt cements. Samples were tested for dynamic modulus in accordance with AASHTO TP 62 and after the long-term oven-aging procedures of AASHTO R 30-02. The results are shown in tables 12-14.

TABLE 12. Dynamic Modulus for HMA Samples Prepared with PG 76-28

Test Temp. (C)	Frequency (Hz)	Dynamic Modulus (psi)			
		Conventional		LTOA	
		Sample 1	Sample 2	Sample 1	Sample 2
4.4	25	4,172,370	2,497,402	N/T	N/T
	10	3,428,234	2,257,024	3,845,029	3,746,006
	5	2,960,076	2,035,835	3,521,536	3,470,450
	1	2,183,832	1,575,095	2,850,166	2,824,008
	0.5	1,947,394	1,408,532	2,583,173	2,585,764
	0.1	1,478,689	1,076,120	2,051,943	2,103,875
21.1	25	1,884,801	1,607,441	4,102,166	2,022,159
	10	1,420,543	1,189,333	2,287,394	1,681,368
	5	1,149,052	941,290	1,923,988	1,442,693
	1	710,007	556,524	1,311,937	998,292
	0.5	581,369	453,503	1,111,410	860,007
	0.1	379,783	299,047	770,967	605,065
37.8	25	860,408	553,745	723,566	603,762
	10	607,379	387,954	652,378	512,455
	5	471,709	299,960	567,376	451,380
	1	288,135	183,667	350,370	286,176
	0.5	238,025	157,492	286,035	233,737
	0.1	166,154	107,782	195,493	159,028

TABLE 13. Dynamic Modulus for HMA Samples Prepared with PG 70-28

Test Temp. (C)	Frequency (Hz)	Dynamic Modulus (psi)			
		Conventional		LTOA	
		Sample 1	Sample 2	Sample 1	Sample 2
4.4	25	2,867,479	2,867,479	3,206,059	2,502,094
	10	2,637,324	2,637,324	2,864,623	2,286,280
	5	2,351,472	2,351,472	2,591,026	2,064,963
	1	1,776,064	1,776,064	2,051,112	1,577,284
	0.5	1,560,407	1,560,407	1,850,765	1,413,729
	0.1	1,135,222	1,135,222	1,425,034	1,070,295
21.1	25	1,102,274	1,102,274	1,233,008	1,211,667
	10	900,657	900,657	1,014,456	964,713
	5	724,968	724,968	863,615	796,483
	1	445,207	445,207	563,056	506,515
	0.5	368,976	368,976	475,358	419,511
	0.1	251,361	251,361	316,381	276,985
37.8	25	296,493	296,493	504,623	396,641
	10	247,477	247,477	463,073	348,973
	5	203,946	203,946	376,095	279,637
	1	140,009	140,009	245,266	164,127
	0.5	121,002	121,002	196,079	130,076
	0.1	93,649	93,649	133,598	88,441

TABLE 14. Dynamic Modulus for HMA Samples Prepared with PG 64-22

Test Temp. (C)	Frequency (Hz)	Dynamic Modulus (psi)			
		Conventional		LTOA	
		Sample 1	Sample 2	Sample 1	Sample 2
4.4	25	3,403,052	3,621,988	3,483,033	4,164,913
	10	3,191,326	3,434,191	3,237,633	3,923,133
	5	2,907,241	3,099,791	3,026,511	3,684,297
	1	2,303,234	2,359,763	2,509,084	3,089,176
	0.5	2,077,749	2,113,143	2,319,287	2,869,420
	0.1	1,586,928	1,514,325	1,875,631	2,219,925
21.1	25	1,817,468	1,521,031	2,187,511	2,567,846
	10	1,481,488	1,186,494	1,879,842	2,034,690
	5	1,215,238	965,697	1,617,870	1,681,121
	1	745,204	585,334	1,086,189	1,055,386
	0.5	602,659	470,968	920,101	870,682
	0.1	365,573	284,110	591,579	550,000
37.8	25	510,907	562,913	770,273	633,471
	10	398,260	412,505	641,354	506,727
	5	310,660	308,838	515,419	407,507
	1	179,682	174,544	309,215	236,769
	0.5	145,643	138,650	226,610	186,880
	0.1	95,754	88,201	110,534	118,401

VOLUMETRIC PROPERTIES

The dynamic modulus of HMA can be estimated using a predictive equation found in the M-EPDG. The predictive equation is often referred to as Witczak's predictive equation (18). The volumetric properties required are gradation of the mix, void properties and effective volume of the binder (18). These mix properties are well defined and easily determined for HMA; however, for CIR mixes several assumptions must be made to determine these parameters. Even with conventional hot recycled HMA mixtures, the properties of the asphalt and aggregate are easily determined. At the elevated mixing temperatures used for hot recycling, the new asphalt is blended with the old asphalt from the RAP. With CIR, the aggregate is coated with old, aged asphalt. At the low mixing temperatures of CIR, there will be little blending of the old asphalt from the RAP with the new asphalt from the emulsion. The amount of blending is difficult to determine and this partial blending makes determination of conventional aggregate and asphalt properties problematic.

Most researchers model CIR mixtures for void calculations by assuming that no blending of the old and new asphalt occurs. This is often referred to as the "black rock" model where the RAP is treated as a black rock and the only asphalt is the residual asphalt from the emulsion. This was our approach; therefore, all gradation parameters were determined based on the gradation of the RAP and asphalt properties were of the new asphalt in the emulsion.

The Gmm was determined on samples of RAP mixed with the ReFlex® emulsion only. The residual asphalt content of the ReFlex® was reported as 65%. At 3.0% emulsion in the mix, this works out to 1.95% residual asphalt (Pb). The 1.95% residual asphalt content was held constant for the other asphalt emulsions. The specific gravity of the base binder (G_b) in the emulsions was not known and is difficult to obtain; therefore, a G_b of 1.020 was assumed. The effective specific gravity (G_{se}) of the black rock (RAP) was back calculated from the Gmm obtained in accordance with AASHTO T 209. When using the black rock model, the bulk specific gravity (G_{sb}) of the black rock (RAP) is obtained by assuming RAP has no absorption, so G_{sb} would be equal to G_{se}. The volumetric and mix properties used with the predictive equation are shown in table 15.

TABLE 15. Volumetric and Mix Design Properties

Emulsion	Reflex		CSS-1h		CSS-1h + Lime		HFE-150P	
Sample	1	2	1	2	1	2	1	2
Pb (%)	1.95	1.95	1.95	1.95	1.95	1.95	1.95	1.95
Gb	1.020	1.020	1.020	1.020	1.020	1.020	1.020	1.020
Ret. 3/4 (%)	3	3	3	3	3	3	3	3
Ret 3/8 (%)	34	34	34	34	34	34	34	34
Ret. # 4 (%)	61	61	61	61	61	61	61	61
Pass. # 200 (%)	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7
Gmm Mix	2.406	2.406	2.406	2.406	2.406	2.406	2.406	2.406
Gsb	2.473	2.473	2.473	2.473	2.473	2.473	2.473	2.473
Gmb	2.114	2.134	2.114	2.117	2.142	2.146	2.130	2.134
Vbeff (%)	4.05	4.09	4.05	4.05	4.1	4.11	4.08	4.09
VTM (%)	12.1	11.3	12.1	12.0	11.0	10.8	11.5	11.3

CHAPTER 6

ANALYSIS OF TEST RESULTS

This chapter provides the analysis of the experimental data. The analysis was performed to determine the effect of emulsion type on dynamic modulus of CIR mixtures. Laboratory measured dynamic modulus values were compared to values determined using Witczak's predictive equation in the M-EPDG. Lastly, the impact on the pavement thickness of any statistical difference found in dynamic modulus was evaluated using the M-EPDG.

LABORATORY DYNAMIC MODULUS

Initial CIR Test Data

AASHTO TP 62 requires dynamic modulus testing at different frequencies and test temperatures because temperature and frequency have a significant effect on dynamic modulus. A review of the test data indicated that frequency had a consistent effect on dynamic modulus, showing an increase in dynamic modulus with an increase in frequency. Therefore, in order to simplify the analysis, a two-way analysis of variance (ANOVA) was performed to determine if there is a statistical difference in dynamic modulus between asphalt emulsion and test temperature by using a single frequency. The middle frequency (5 Hz) was selected since all the frequencies showed similar trends.

The results of the ANOVA, shown in table 16, indicate that asphalt emulsion and test temperature had a significant effect on measured dynamic modulus. The interaction between asphalt emulsion and test temperature did not have a significant effect at a confidence limit of 95% ($\alpha = 0.05$) but was significant at 93% ($\alpha = 0.07$).

TABLE 16. ANOVA for CIR Dynamic Modulus

Source	Degrees Freedom	Sum Squares	Mean Square	F Ratio	Prob. > F
EAC	3	386324703952	128774901317	24.97	<.0001
Temp	2	2.6255345E12	1.3127673E12	254.52	<.0001
EAC * Temp	6	81527140720	13587856787	2.63	0.0723
Error	12	61894527698	5157877308.1		
Total	23	3.1552809E12			

Because there was a slight interaction effect, Duncan's multiple range test was performed by test temperature. Duncan's multiple range test indicates which means are significantly different at a confidence limit of 95% ($\alpha = 0.05$). The results of Duncan's multiple range test at 4.4° C (40°F), 21.1° C (70°F) and 37.8° C (100°F) are shown in tables 17-19, respectively. Means with the same letter not significantly different at a confidence limit of 95% (alpha = 0.05).

TABLE 17. Results of Duncan's Multiple Range Test for CIR Dynamic Modulus at 4.4°C (40°F) Test Temperature

Grouping *	Mean Dynamic Modulus (psi)	n	EAC
A	1,247,547	2	CSS-1h + Lime
A & B	1,032,807	2	HFE-150P
B	859,775	2	CSS-1h
B	755,356	2	ReFlex®

* Means with the same letter not significantly different

TABLE 18. Results of Duncan's Multiple Range Test for CIR Dynamic Modulus at 21.1°C (70°F) Test Temperature

Grouping *	Mean Dynamic Modulus (psi)	n	EAC
A	705,630	2	CSS-1h + Lime
B	441,812	2	HFE-150P
B	407,482	2	CSS-1h
B	331,652	2	ReFlex®

* Means with the same letter not significantly different

TABLE 19. Results of Duncan's Multiple Range Test for CIR Dynamic Modulus at 37.8°C (100°F) Test Temperature

Grouping *	Mean Dynamic Modulus (psi)	N	EAC
A	275,244	2	CSS-1h + Lime
B	164,347	2	HFE-150P
C	135,164	2	CSS-1h
D	114,046	2	ReFlex®

* Means with the same letter not significantly different

At the 4.4°C (40°F) test temperature, no statistical difference in dynamic modulus exists between CSS-1h + Lime and HFE 150P, or between the HFE 150P, CSS-1h and ReFlex® emulsions. A statistical difference in dynamic modulus exists between CSS-1h + Lime and both CSS-1h and ReFlex®. At the 21.1° C (70°F) test temperature the differences in stiffness become more apparent. There is no statistically significant difference between the CSS-1h, HFE 150P or the ReFlex® emulsions. However, the CSS-1h with lime is significantly stiffer than the other three emulsions. At the highest test temperature measured, 37.8° C (100°F), the effect of the lime and polymer modification are apparent. There is a significant difference in dynamic modulus between each emulsion evaluated. The CSS-1h with lime was the stiffest followed by the HFE 150P, the CSS-1h and the ReFlex® emulsion.

There was little information found in the available literature for typical dynamic modulus values of CIR mixtures except for CIR mixtures with ReFlex® emulsion (24). The dynamic modulus values obtained with the US 412 RAP and ReFlex® emulsion were similar to the values found in the literature. The ANOVA indicates the emulsion type has a significant effect on dynamic modulus. Therefore, when using the M-EPDG, different emulsion types could impact predicted pavement performance.

Comparisons with HMA

The dynamic modulus values of the CIR mixtures were compared to the dynamic modulus values of the S-4 mix used as the overlay on US 412, Beaver, County. The S-4 mix was made with PG 76-28, PG 70-28 and PG 64-22 asphalt cements. The comparisons can be made by master curve, which would show the effect of both temperature and frequency. However, frequency has a consistent effect on dynamic modulus and making the comparisons at one frequency simplifies the analysis. The comparisons between the CIR mixtures and HMA mixtures at a frequency of 5 Hz are shown in figures 26-28.

Figure 26 shows the comparisons between the ReFlex® CIR mix and the HMA mixtures. The ReFlex® was reported as being made with PG 58-28 base asphalt. The ReFlex® CIR mixture was not as stiff as the S-4 HMA mix at any of the three temperatures evaluated. The dynamic modulus values of the ReFlex® mix did compare well with values reported in the literature (24).

The CSS-1h mixtures are typically made with PG 64-22 base asphalt. Even so, the CSS-1h CIR mixture was not as stiff as the S-4 HMA mixtures at any of the temperatures evaluated (figure 27). Adding lime to the CSS-1h CIR mixture increased the measured dynamic modulus at all temperatures evaluated. The CSS-1h with lime samples had dynamic modulus values comparable to the S-4 mix at the higher two test temperatures.

HFE 150P mixtures are typically made with PG 64-28 base asphalt. The HFE 150P CIR mixture was not as stiff as the S-4 HMA mixtures at the two lower test temperatures (figure 28). At the highest test temperature, 37.8°C, the HFE 150P was similar to, but slightly less stiff, than the HMA mixtures.

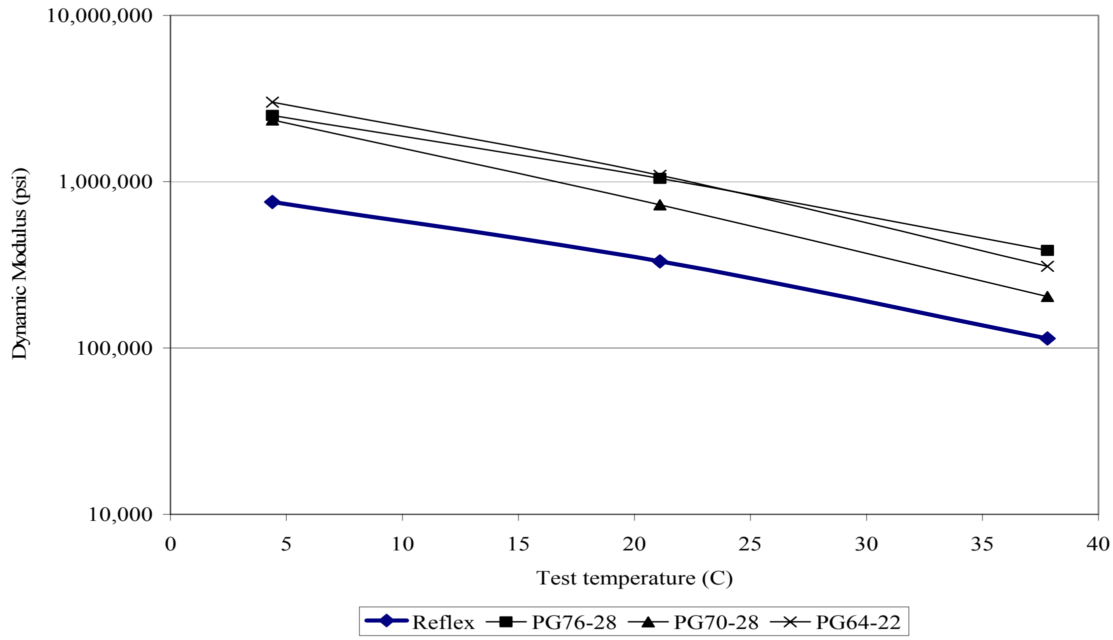

FIGURE 26 Comparison of E* with temperature at 5 Hz between ReFlex® and HMA.

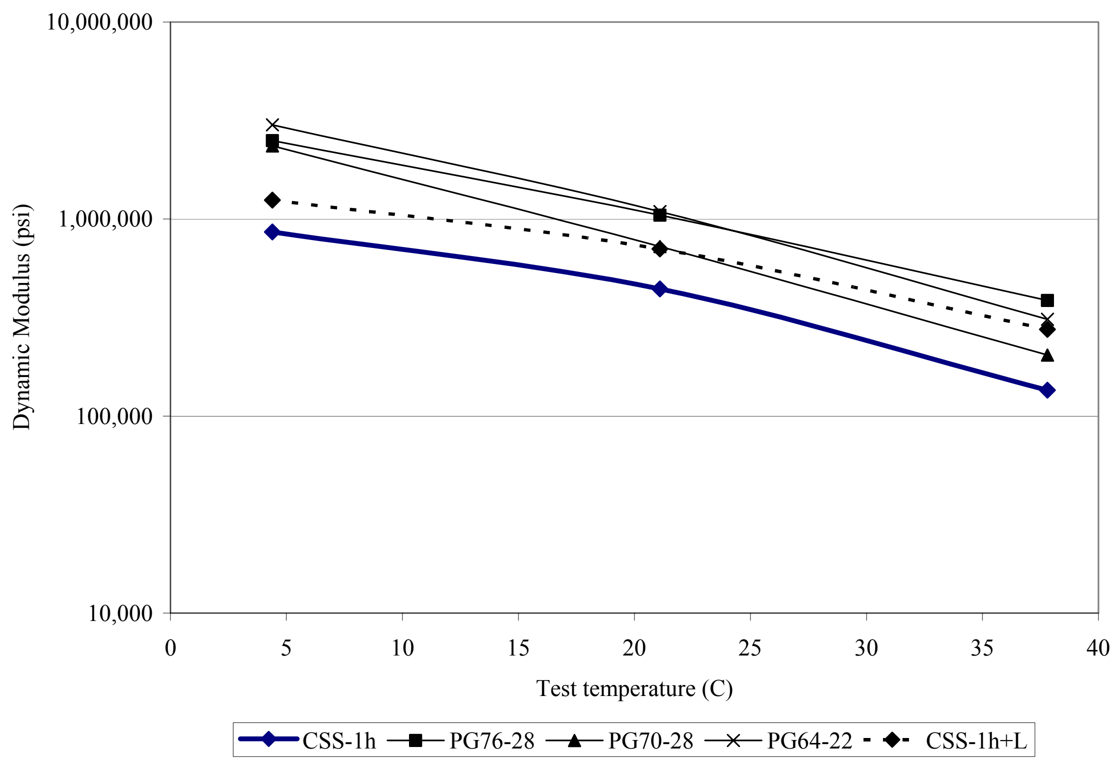

FIGURE 27 Comparison of E* with temperature at 5 Hz between CSS-1h and HMA.

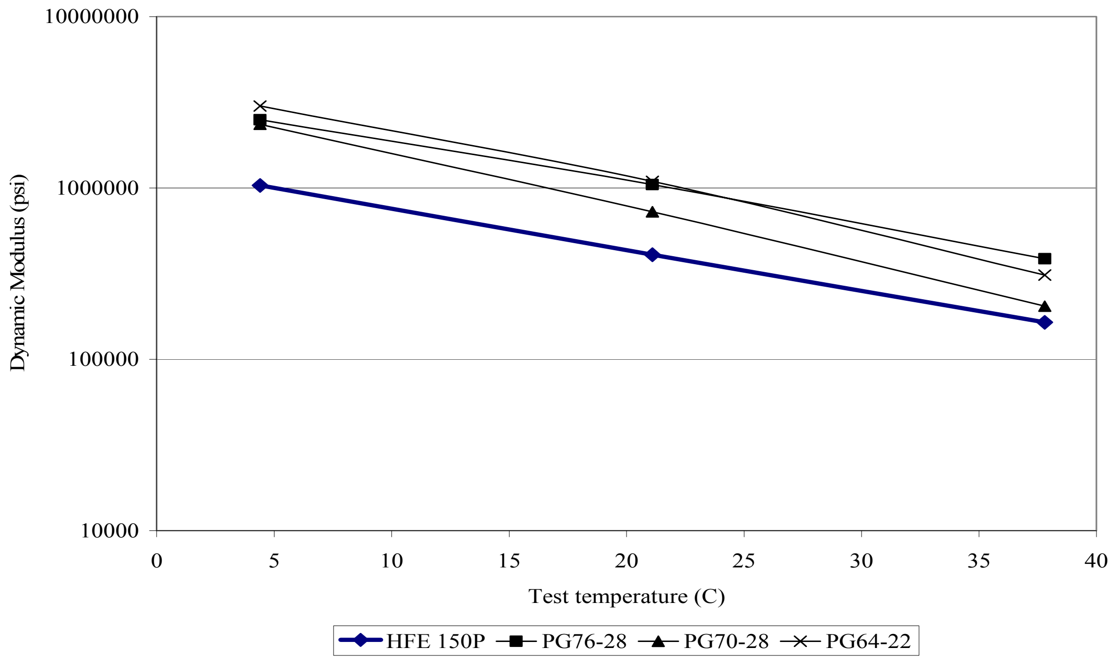

FIGURE 28 Comparison of E* with temperature at 5 Hz between HFE 150P and HMA.

The CIR mixtures evaluated were not as stiff as the HMA samples. It would appear that, at least initially, the lower stiffness of the CIR samples is a function of the increased air voids and softer base asphalts. The old residual asphalt in the RAP does not appear to affect the dynamic modulus of the CIR mixtures. CIR samples must fully cure to gain their ultimate strength and stiffness. The CIR samples were oven cured prior to testing, which should have eliminated the initial curing effect.

Aging Effects

It was originally thought that the old asphalt cement in the RAP would have an effect on the dynamic modulus values of CIR samples. However, because the old asphalt cement is oxidized, it was thought that aging effects would be less for CIR mixtures than conventional HMA samples. To evaluate the aging effects, additional samples were tested after the long-term oven-aging (LTOA) protocol of AASHTO R 30. Again, to simplify the analysis, the discussion is limited to samples tested at 5 Hz. The average dynamic modulus and percent increase with LTOA are shown in table 20.

The data indicates that LTOA had similar effects on dynamic modulus for CIR and HMA mixtures. The average percent increase in dynamic modulus at 4.4^{o}C was 14 percent for both the CIR and HMA mixtures. At 21.1^{o}C the percent change in dynamic modulus with LTOA was 33 percent for the CIR mixtures and 43 percent for the HMA mixtures. At 37.8^{o}C the percent change in dynamic modulus with LTOA was 49 percent for the CIR mixtures and 48 percent for the HMA mixtures. LTOA had a greater effect on dynamic modulus, as measured by percent increase in stiffness, with an increase in test temperature.

TABLE 20. Effect of LTOA on E*

Mix	Test Temp (C)	TP 62 E* (psi)	LTOA E* (psi)	Change in E* (%)
Reflex	4.4	755,356	1,013,662	34
	21.1	331,651	510,777	54
	37.8	114,046	207,792	82
CSS-1h + Lime	4.4	1,247,547	1,299,822	4
	21.1	705,630	815,463	16
	37.8	275,243	345,966	26
HFE 150P	4.4	1,032,807	1,069,460	4
	21.1	407,481	525,961	29
	37.8	164,346	229,613	40
PG 76-28	4.4	2,497,956	3,495,993	40
	21.1	1,045,171	1,683,341	61
	37.8	385,834	509,378	32
PG 70-28	4.4	2,578,959	2,327,995	-10
	21.1	714,540	830,049	16
	37.8	201,022	327,866	63
PG 64-22	4.4	3,003,516	3,355,404	12
	21.1	1,090,467	1,649,496	51
	37.8	309,749	461,463	49

MASTER CURVES

To perform a level 1 analysis using the M-EPDG, the dynamic modulus at 54.4°C (130°F) is required (18). The dynamic modulus of the CIR samples at the 54.4°C (130°F) test temperature could not be determined due to the sensitivity of the 10-kip load cell available at the time of testing. A smaller capacity (2-kip) load cell was purchased but only the HFE 150P samples were tested with the smaller load cell and there was not sufficient RAP to retest the other emulsions. To determine the dynamic modulus at 54.4° C (130°F) for use in the M-EPDG, master curves were developed at the three available test temperatures and the dynamic modulus at each frequency determined by extrapolating the values from the master curves.

According to the user manual for the M-E PDG (18), the stiffness of HMA at all levels of temperature and time rate of load is determined from a master curve constructed at a reference temperature (generally taken as 70°F). Master curves are constructed using the principle of time-temperature superposition. The data at various temperatures are shifted with respect to time until the curves merge into a single smooth function. The master curve of the dynamic modulus as a function of time formed in this manner describes the time dependency of the material. The amount of shifting at each temperature required to form the master curve describes the temperature dependency of the material. The greater the shift factor, the greater the temperature dependency (temperature susceptibility) of the mixture.

According to the M-EPDG (18), the master modulus curve can be mathematically modeled by a sigmoidal function described as:

$$\log\left(E^*\right) = \delta + \frac{\alpha}{1+e^{\beta+\gamma(\log t_r)}} \qquad [1]$$

Where,

t_r = reduced time of loading at reference temperature
δ = minimum value of E*
$\delta + \alpha$ = maximum value of E*
β, γ = parameters describing the shape of the sigmoidal function

The shift factor can be shown in the following form:

$$a(T) = t / t_r \qquad [2]$$

Where,
a(T) = shift factor as a function of temperature
t = time of loading at desired temperature
t_r = reduced time of loading at reference temperature
T = temperature of interest

For precision, a second order polynomial relationship between the logarithm of the shift factor i.e. log a (Ti) and the temperature in degrees Fahrenheit is used. The relationship can be expressed as follows:

$$Log\, a(Ti) = aTi^2 + bTi + c \qquad [3]$$

Where,
a(Ti) = shift factor as a function of temperature Ti
Ti = temperature of interest, °F
a, b and c = coefficients of the second order polynomial

Extrapolated Results for Dynamic Modulus at 54.4°C (130°F)

The data available at the three test temperatures were shifted with respect to time until the curves merged into a single sigmoidal function representing the master curve using a second order polynomial relationship between the logarithm of the shift factors, log $a(T_i)$ and the temperature. The time-temperature superposition was performed by simultaneously solving for the four coefficients of the sigmoidal function (δ, α, β, and γ) as described in equation [1] and the three coefficients of the second order polynomial (a, b, and c) as described in equation [3]. A Microsoft™ Excel program, developed by Tran (25), was used to conduct the nonlinear optimization for simultaneously solving these seven parameters for developing the master curves. Table 21 shows the extrapolated dynamic modulus values for 54.4°C (130°F) test temperature and figures 29–32 show the complete master curves.

TABLE 21. Average Dynamic Modulus with Extrapolated Values

Test Temp. (C)	Frequency (Hz)	Dynamic Modulus (psi) Reflex	CSS-1h	CSS-1h + Lime	HFE 150P
4.4	25	885,741	1,011,549	1,402,196	1,201,075
	10	830,806	950,467	1,331,266	1,131,817
	5	755,356	859,774	1,247,547	1,032,807
	1	580,350	671,054	1,046,054	818,683
	0.5	522,546	602,405	971,732	742,929
	0.1	398,315	465,329	788,752	581,322
21.1	25	467,915	657,049	871,618	576,910
	10	398,553	535,068	806,949	479,632
	5	331,651	441,812	705,630	407,481
	1	217,850	284,800	506,179	269,739
	0.5	182,208	228,606	438,491	230,683
	0.1	127,064	150,599	314,125	170,161
37.8	25	225,052	282,050	565,708	238,018
	10	148,055	176,698	346,521	197,490
	5	114,046	135,164	275,243	164,346
	1	74,475	83,262	174,464	108,311
	0.5	52,528	66,522	144,960	91,540
	0.1	27,456*	42,343*	96,195	63,106
54.4	25	56,499*	71,000*	157,897*	97,468
	10	38,492*	53,078*	123,876*	84,910
	5	30,845*	44,000*	97,865*	64,740
	1	15,344*	29,087*	60,864*	37,299
	0.5	10,343*	25,867*	54,765*	33,267
	0.1	5,783*	19,878*	35,876*	24,011

* Extrapolated test results

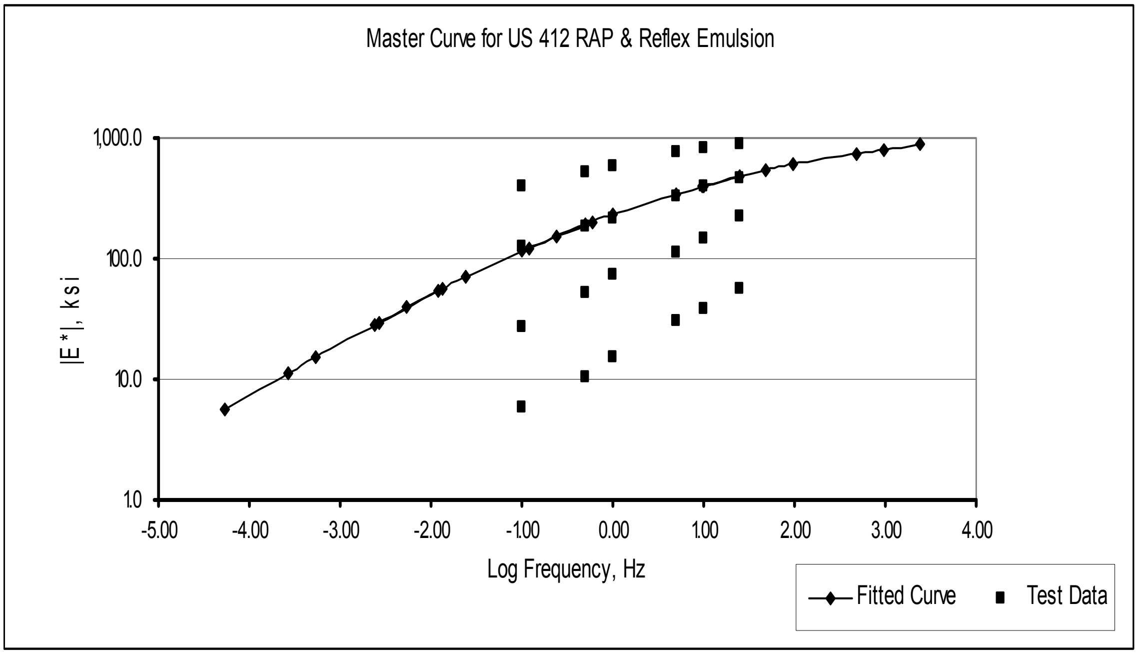

FIGURE 29 Master curve for US 412 and ReFlex® emulsion.

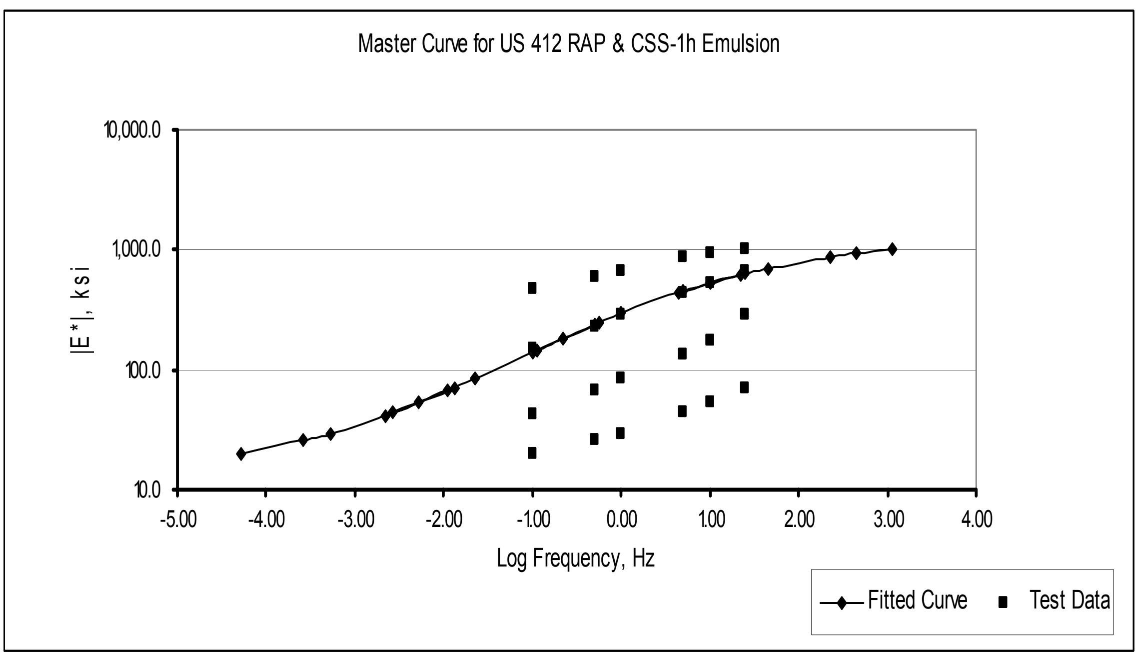

FIGURE 30 Master curve for US 412 RAP and CSS-1h emulsion.

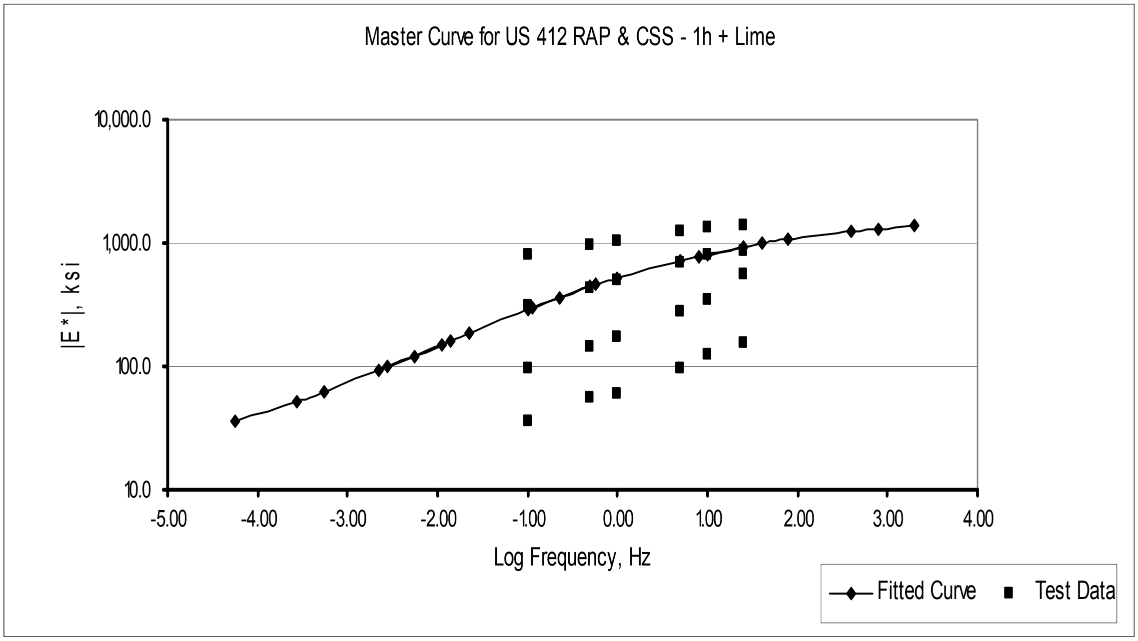

FIGURE 31 Master curve for US 412 RAP and CSS-1h+Lime emulsion.

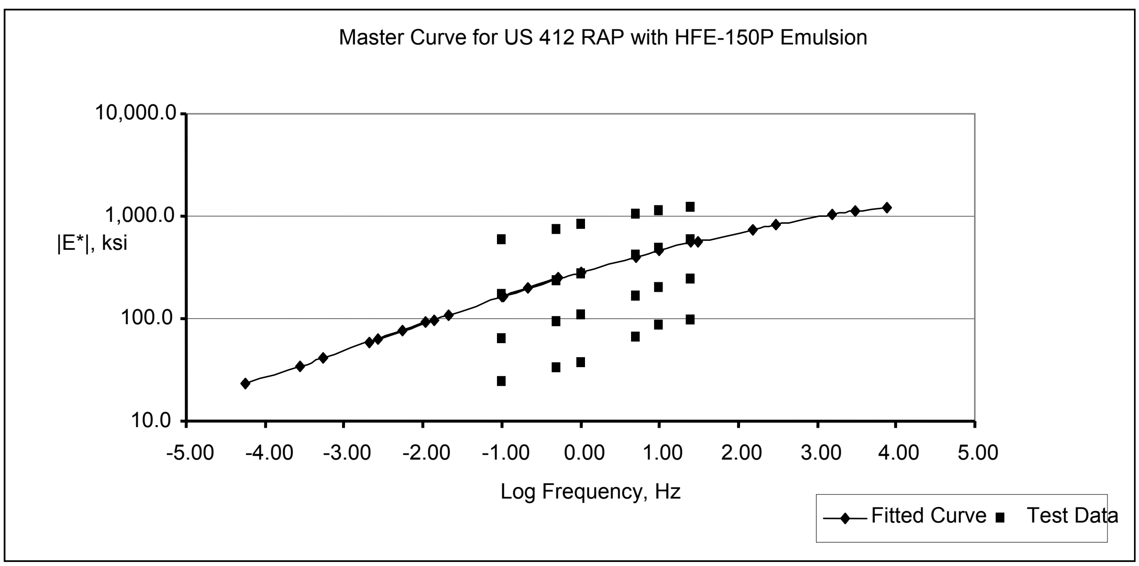

FIGURE 32 Master curve for US 412 RAP and HFE 150P emulsion.

E* PREDICTIVE EQUATION

One of the objectives of this study was to compare the experimental dynamic modulus data to the predicted values using Witczak's equation. The new M-EPDG uses the laboratory dynamic modulus data for input Level 1, while it uses dynamic modulus values from Witczak's predictive equation for input Levels 2 and 3. The Witczak predictive model was based upon 2,750 test points and 205 different HMA mixtures (34 of which are modified). Most of the 205 HMA mixtures were dense-graded using unmodified asphalts. The current version of the predictive equation, updated in 1999, is (18):

$$\log E^* = 1.249937 + 0.249937 + 0.02932\rho_{200} - 0.001767(\rho_4)^2 - 0.002841\rho_4 - 0.058097V_a$$
$$-0.802208\left(\frac{V_{beff}}{V_{beff}+V_a}\right) + \frac{3.871977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017(\rho_{38})^2 + 0.005470\rho_{34}}{1+e^{(-0.603313-0.313351\log(f)-0.393532\log(\eta))}} \quad [4]$$

Where,

E* = dynamic modulus, 10^5 psi
η = asphalt viscosity at the age and temperature of interest, 106 Poise (use of RTFO aged viscosity is recommended for short-term oven aged lab blend mix)
f = loading frequency, Hz
Va = air void content, %
Vbeff = effective asphalt content, % by volume
ρ34 = cumulative % retained on 3/4 in (19 mm) sieve
ρ38 = cumulative % retained on 3/8 in (9.5 mm) sieve
ρ4 = cumulative % retained on #4 (4.76 mm) sieve
ρ200 = % passing #200 (0.075 mm) sieve

The asphalt viscosity (η) required in equation [4] can be measured using a DSR or it can be calculated using equation [5] shown below. Default values for A and VTS, measures of the temperature susceptibility of the asphalt, are available in the M-EPDG if the grade of the asphalt cement is known. The grade of the typical base binder used with ReFlex® was available, PG 58-28 (24). Typical base PG asphalt grades for CSS-1h and HFE 150P are PG 64-22 and PG 64-28, respectively, and were used in equation [5].

$$\log\log\eta = A + VTS\log T_R \quad [5]$$

Where,

η = asphalt viscosity, cP
A, VTS = regression parameters
T_R = temperature, ° Rankine

Witczak's equation [4] was used to determine the predicted dynamic modulus for each of the samples tested. The predicted dynamic modulus data for all the samples tested are provided in tables 22–25. The volumetric properties used to determine the predicted dynamic modulus for each of the samples were listed in table 15 of chapter 4.

TABLE 22. Predicted Dynamic Modulus for US 412 RAP and ReFlex® Emulsion

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi)		
		Sample 1	Sample 2	Average
-10	25	2,357,046	2,553,931	2,455,488
	10	2,229,960	2,414,158	2,322,059
	5	2,127,763	2,303,519	2,215,641
	1	1,877,736	2,032,840	1,955,288
	0.5	1,765,796	1,911,653	1,838,725
	0.1	1,500,726	1,624,689	1,562,707
4.4	25	1,320,586	1,426,318	1,373,452
	10	1,169,271	1,265,855	1,217,563
	5	1,060,191	1,147,765	1,103,978
	1	822,120	890,028	856,074
	0.5	727,965	788,097	758,031
	0.1	533,099	577,134	555,116
21.1	25	490,610	531,135	510,872
	10	401,095	434,226	417,661
	5	341,430	369,633	355,532
	1	228,562	247,442	238,002
	0.5	190,191	205,901	198,046
	0.1	121,461	131,494	126,477
37.8	25	160,649	173,919	167,284
	10	124,109	134,361	129,235
	5	101,562	109,951	105,757
	1	62,945	68,144	65,545
	0.5	51,046	55,263	53,154
	0.1	31,335	33,923	32,629
54.4	25	57,842	62,619	60,231
	10	43,817	47,436	45,626
	5	35,506	38,439	36,972
	1	21,891	23,699	22,795
	0.5	17,849	19,323	18,586
	0.1	11,287	12,219	11,753

TABLE 23. Predicted Dynamic Modulus for US 412 and CSS-1h Emulsion

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi)		
		Sample 1	Sample 2	Average
-10	25	2,653,046	2,678,728	2,665,887
	10	2,543,367	2,567,988	2,555,677
	5	2,455,145	2,478,912	2,467,029
	1	2,233,600	2,255,222	2,244,411
	0.5	2,131,538	2,152,172	2,141,855
	0.1	1,881,761	1,899,977	1,890,869
4.4	25	1,638,948	1,654,814	1,646,881
	10	1,487,425	1,501,824	1,494,624
	5	1,372,974	1,386,265	1,379,620
	1	1,113,006	1,123,780	1,118,393
	0.5	1,005,656	1,015,391	1,010,524
	0.1	773,184	780,669	776,927
21.1	25	675,688	682,229	678,958
	10	565,394	570,867	568,131
	5	489,730	494,471	492,101
	1	340,746	344,045	342,396
	0.5	287,943	290,730	289,337
	0.1	189,759	191,596	190,677
37.8	25	229,693	231,917	230,805
	10	179,961	181,703	180,832
	5	148,627	150,065	149,346
	1	93,573	94,479	94,026
	0.5	76,194	76,931	76,562
	0.1	46,898	47,352	47,125
54.4	25	80,856	81,639	81,247
	10	61,443	62,038	61,741
	5	49,821	50,304	50,062
	1	30,586	30,882	30,734
	0.5	24,833	25,074	24,953
	0.1	15,471	15,621	15,546

TABLE 24. Predicted Dynamic Modulus for US 412 and CSS-1h+Lime

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi)		
		Sample 1	Sample 2	Average
E	25	2,958,157	3,015,293	2,986,725
	10	2,835,864	2,890,639	2,863,252
-10	5	2,737,497	2,790,372	2,763,934
	1	2,490,473	2,538,576	2,514,525
	0.5	2,376,673	2,422,578	2,399,626
	0.1	2,098,171	2,138,697	2,118,434
	25	1,827,434	1,862,731	1,845,082
	10	1,658,485	1,690,518	1,674,502
4.4	5	1,530,872	1,560,441	1,545,656
	1	1,241,006	1,264,976	1,252,991
	0.5	1,121,311	1,142,969	1,132,140
	0.1	862,104	878,755	870,430
	25	753,395	767,946	760,670
	10	630,417	642,593	636,505
21.1	5	546,051	556,598	551,325
	1	379,934	387,272	383,603
	0.5	321,058	327,259	324,158
	0.1	211,582	215,668	213,625
	25	256,109	261,055	258,582
	10	200,657	204,533	202,595
37.8	5	165,719	168,920	167,320
	1	104,334	106,349	105,342
	0.5	84,956	86,597	85,777
	0.1	52,291	53,301	52,796
	25	90,155	91,896	91,025
	10	68,509	69,833	69,171
54.4	5	55,551	56,624	56,087
	1	34,103	34,762	34,433
	0.5	27,689	28,224	27,957
	0.1	17,250	17,584	17,417

TABLE 25. Predicted Dynamic Modulus for US 412 and HFE 150P

Temperature (C)	Frequency (Hz)	Dynamic Modulus (psi)		
		Sample 1	Sample 2	Average
-10	25	2,511,416	2,561,097	2,536,257
	10	2,374,754	2,421,731	2,398,243
	5	2,266,537	2,311,373	2,288,955
	1	2,001,620	2,041,216	2,021,418
	0.5	1,882,935	1,920,183	1,901,559
	0.1	1,601,678	1,633,362	1,617,520
4.4	25	1,480,007	1,509,284	1,494,646
	10	1,320,930	1,347,060	1,333,995
	5	1,203,085	1,226,884	1,214,985
	1	943,293	961,953	952,623
	0.5	839,432	856,037	847,734
	0.1	622,056	634,362	628,209
21.1	25	609,195	621,246	615,221
	10	503,544	513,505	508,524
	5	432,150	440,698	436,424
	1	294,516	300,342	297,429
	0.5	246,801	251,683	249,242
	0.1	159,841	163,003	161,422
37.8	25	218,761	223,088	220,925
	10	170,597	173,972	172,285
	5	140,455	143,233	141,844
	1	87,934	89,673	88,803
	0.5	71,486	72,901	72,194
	0.1	43,927	44,796	44,361
54.4	25	82,728	84,365	83,546
	10	62,835	64,078	63,456
	5	50,940	51,947	51,444
	1	31,279	31,898	31,588
	0.5	25,405	25,907	25,656
	0.1	15,850	16,163	16,007

Comparison of Experimental and Predicted Master Curves

The predicted dynamic modulus values of the CIR mixtures were compared to the measured dynamic modulus values. The comparisons can be made by master curve, which would show the effect of both temperature and frequency. However, frequency has a consistent effect on dynamic modulus and making the comparison at one frequency simplifies the analysis. Table 26 shows the percent increase in predicted dynamic modulus compared to the measured dynamic modulus at a frequency of 5 Hz. The comparisons between the predicted and measured dynamic modulus values at a frequency of 5 Hz are shown graphically in figures 33-35.

TABLE 26. Percent Increase in Predicted E* Compared to Measured E*

Temp (C)	Reflex	CSS-1h	CSS-1h + Lime	HFE 150P
	Percent Increase in E*			
4.4	46.2	60.5	23.9	17.6
21.1	7.2	11.4	-21.9	7.1
37.8	-7.3	10.5	-39.2	-13.7
54.4	N/T	N/T	N/T	-20.5

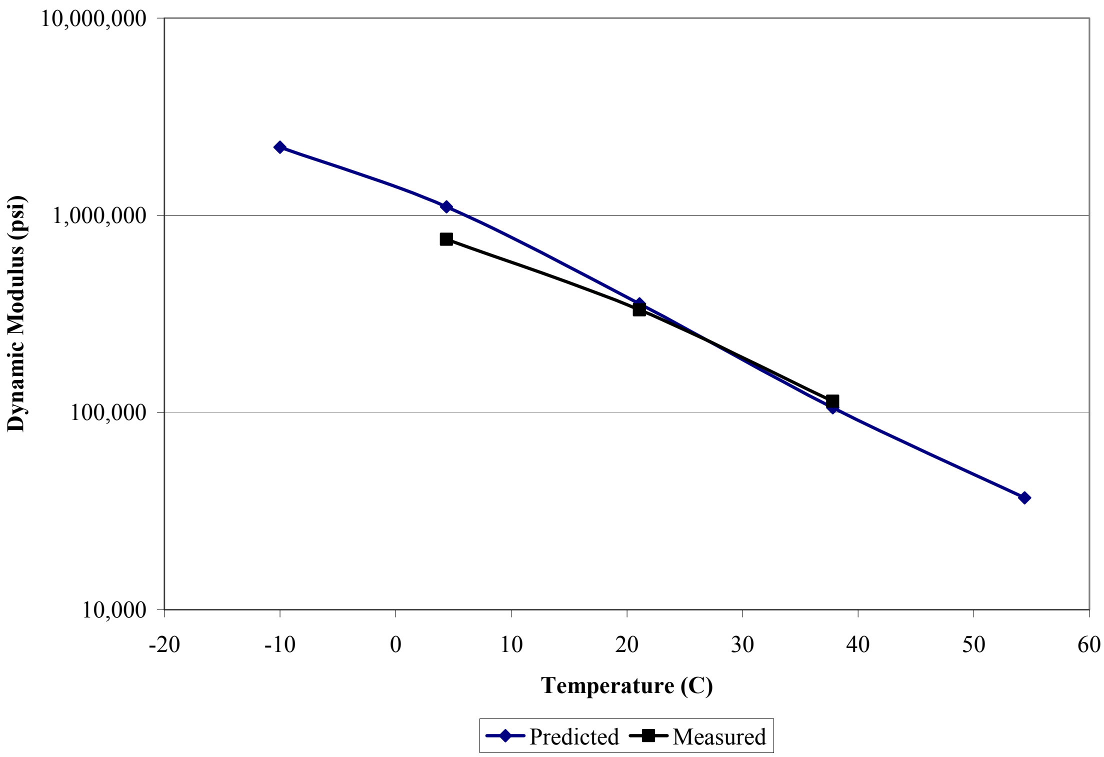

FIGURE 33 Experimental and predicted dynamic modulus at 5 Hz for ReFlex® emulsion.

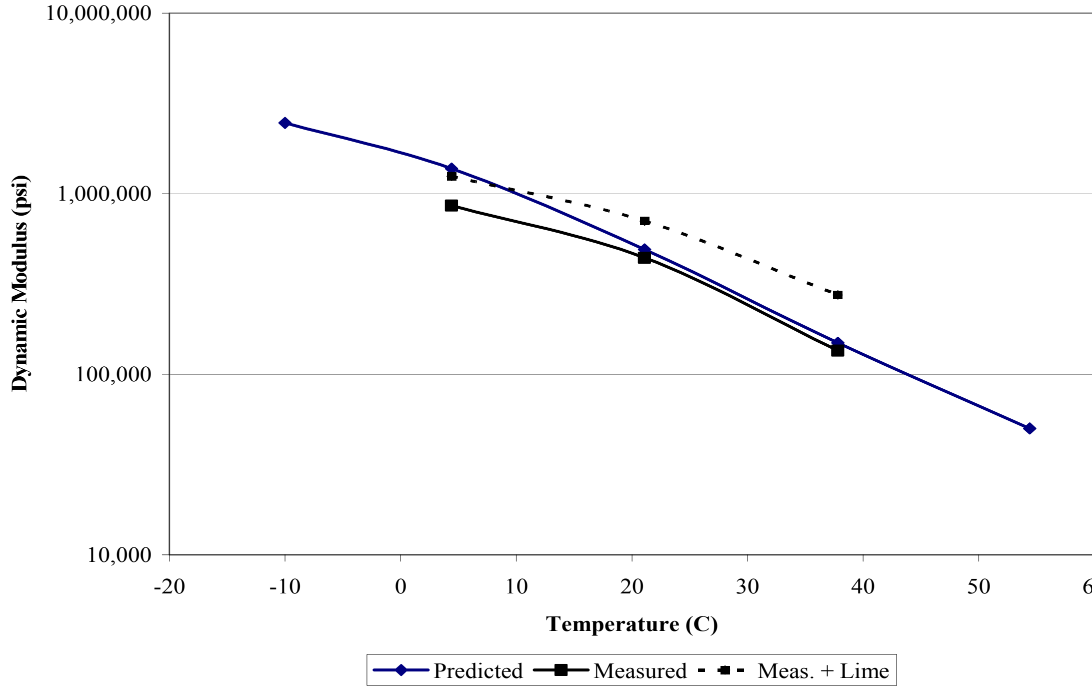

FIGURE 34 Experimental and predicted dynamic modulus at 5 Hz for CSS-1h and CSS-1h + Lime samples.

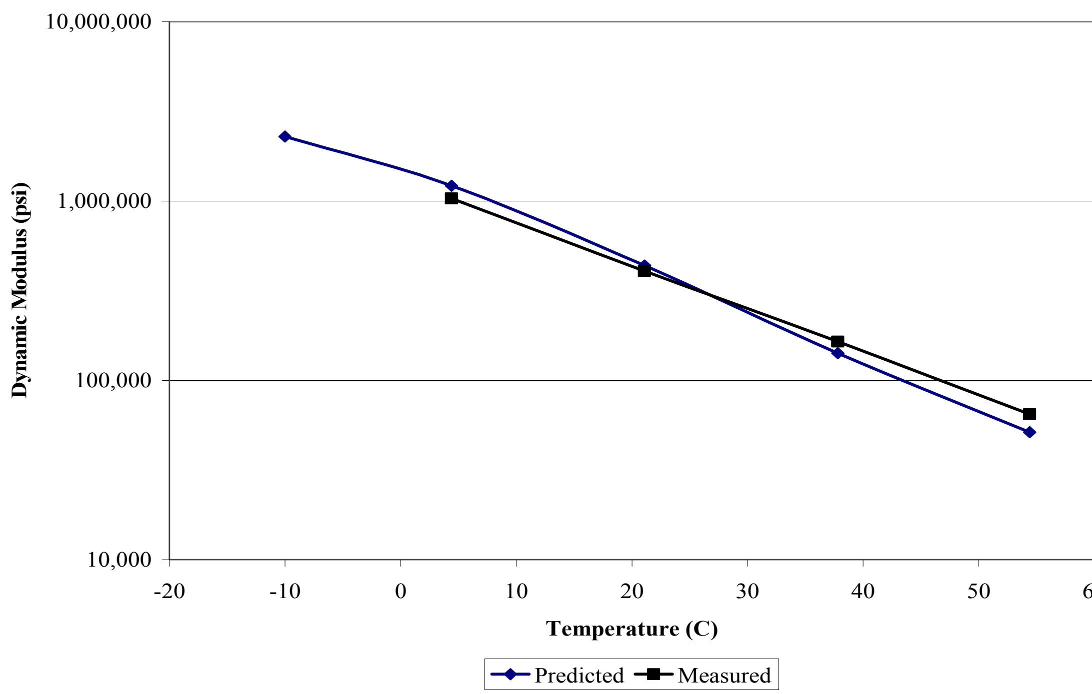

FIGURE 35 Experimental and predicted dynamic modulus at 5 Hz for HFE-150P emulsion.

Figures 33–35 showed that the experimental and predicted dynamic modulus values are similar. The predictive equation seems to over predict dynamic modulus at the lower test temperatures. Adding lime greatly increased the stiffness at all temperatures. Excluding samples with lime, the average predictive dynamic modulus at 5 Hz was approximately 41 percent higher than the average measured value at 4.4^{o}C, 8.5 percent higher at 21.1^{o}C and 3.5 percent lower at 37.8^{o}C. Dynamic modulus values are more critical at the higher test temperatures in the M-EPDG; therefore, it appears that the predictive equation in the M-EPDG can be used to estimate dynamic modulus by using the PG grade of the base asphalt and assuming the "black rock" model where the RAP is treated as aggregate.

M-EPDG

The ANOVA showed a significant effect of asphalt emulsion on dynamic modulus. To determine if the differences in dynamic modulus by emulsions would have an effect on pavement performance, the M-EPDG was used. The analysis was performed using a trial version of the M-EPDG that was available for a short time on the web. The web based version has been removed and version 1.0 has been made available to each DOT. There were numerous documented deficiencies and errors in the web based version. The researchers encountered problems with the program occasionally crashing and with getting different results on repeat runs of the same input file. The analysis needs to be rerun using the latest version of the M-EPDG to verify the following results.

Project Information

M-EPDG is an analysis tool that gives levels of distress for terminal IRI, longitudinal cracking, alligator cracking, thermal cracking and permanent deformation (rutting). A typical predicted performance plot for bottom-up fatigue cracking is shown in figure 36. When performing an analysis, the user selects failure criteria for each distress or selects default values. For this study the recommended default values were used (18) and are shown in table 27. Initial pavement smoothness is also required and the recommended default value of 63 in/mi, based on IRI, was used.

The M-EPDG requires traffic, climatic and soil information as well as material properties. The M-EPDG requires traffic in average annual daily truck traffic (AADTT), not ESALs. The project data was in ESAL's. The traffic count on US 412 in Beaver County was 1,800 vpd. Using this number, the AADTT required to produce the design ESALs was calculated using the axle weights tables in the M-EPDG. The traffic inputs were an AADTT of 810 and a 0.5 percent growth rate was used. The required climatic data used in the evaluation was for the western portion of US 412 in Beaver County. Default values for a CL soil for the subgrade were selected as typical for the project location. A 20 year design life was selected.

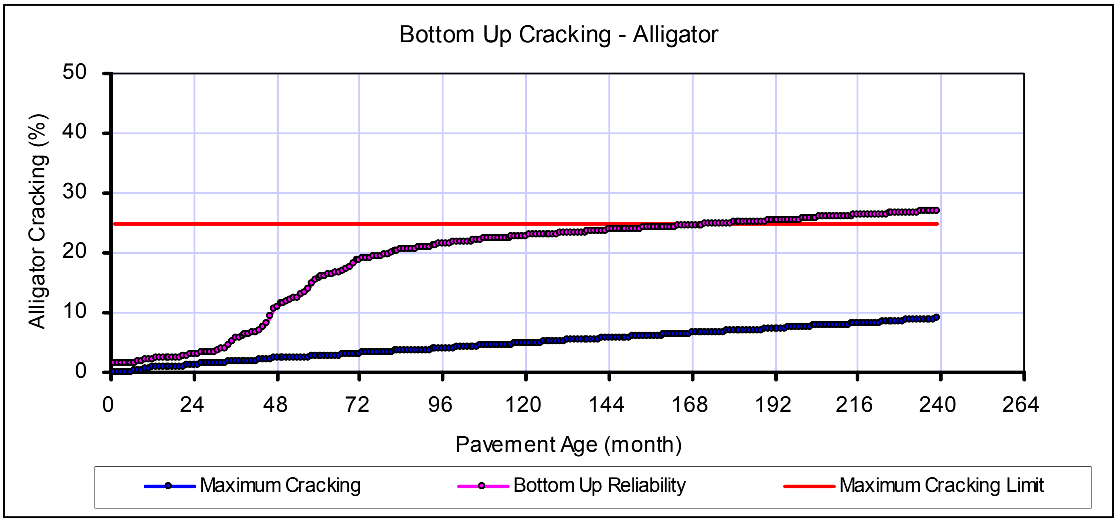

FIGURE 36 Typical M-EPDG performance plot.

TABLE 27. Default Performance Criteria (18)

Distress	Limit	Reliability
Terminal IRI (in/mile)	172	90
Longitudinal Cracking (ft/mile)	2000	90
Alligator Cracking (%)	25	90
Thermal Cracking (ft/mile)	1000	90
Permanent Deformation (in)	0.75	90

Structure

The total pavement thickness at the test sites on US 412 was unknown. The rehabilitation of the pavement at the test sites consisted of 4 inches of CIR using ReFlex® emulsion and a 2-inch HMA overlay using PG 76-28 asphalt cement. The thickness of the remaining pavement was unknown. The pavement structure modeled consisted of the 2-inch HMA overlay and the CIR layer. CIR goes back down slightly thicker than the milling depth so a 4.5 inch thick layer was used. The thickness of the old HMA layer used in the model was selected by running trial thicknesses in the M-EPDG until a failure in one or more performance parameters resulted in a 20 – 30 year time frame. Figure 37 shows the trial section used for analysis.

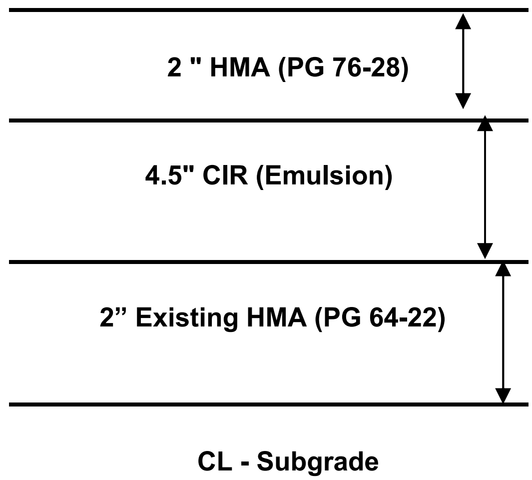

FIGURE 37 M-EPDG trial section.

Material Properties

The following material properties were used for the pavement layers to the model:

- Layer 1 HMA Overlay: Level 3 default input parameters were used with PG 76-28 asphalt cement.
- Layer 2 CIR: Level 1 input parameters were used. The measured dynamic modulus for each emulsion was used. A level 1 analysis requires binder properties of G* and delta (δ) values at a minimum of three temperatures. The data was not available for the base binders so default values were used. The default input values for mixture thermal properties were used.
- Layer 3 Existing HMA: Level 1 input parameters were used. For an existing HMA section the software only needs the gradation of the mix and the binder properties.
- Layer 4 Subgrade: Level 3 input parameters were used with clay (CL) soil for the subgrade.
- Thermal Cracking: Default parameters for thermal cracking were used.

Design Trials and Results

The trial section shown in figure 37 was considered as the baseline for trial runs. The dynamic modulus values for the other three emulsions were substituted for the ReFlex® emulsion data and the remaining parameters held constant. The thickness of the HMA overlay was varied until the pavement structure failed within a 20-30 year time frame or the minimum overlay thickness was reached. A summary of the results are provided in the table 28. The required minimum thickness of the HMA overlay with each of the emulsions is given in table 29. The results of the trial runs indicate that the stiffer CIR layers, CSS-1h with lime and HFE 150P, required thinner HMA overlays for similar performance.

TABLE 28. Trial M-EPDG Runs to Arrive at a Minimum HMA Overlay Thickness

Trial No.	Emulsions	HMA Overlay Thickness	Pavement Distresses				
			IRI	LC	AC	TC	R
1	Reflex	2.00"	√	√	√	√	√
2	Reflex	1.75"	√	√	√	√	√
3	Reflex	1.50"	√	√	**X**	√	√
4	CSS - 1h	2.00"	√	√	√	√	√
5	CSS - 1h	1.75"	√	√	√	√	√
6	CSS - 1h	1.50"	√	√	√	√	√
7	CSS - 1h	1.25"	√	√	√	√	√
8	CSS - 1h	1.00"	√	√	**X**	√	√
9	CSS-1h+Lime	2.00"	√	√	√	√	√
10	CSS-1h+Lime	1.75"	√	√	√	√	√
11	CSS-1h+Lime	1.50"	√	√	√	√	√
12	CSS-1h+Lime	1.25"	√	√	√	√	√
13	CSS-1h+Lime	1.00"	√	√	√	√	√
14	HFE-150P	2.00"	√	√	√	√	√
15	HFE-150P	1.75"	√	√	√	√	√
16	HFE-150P	1.50"	√	√	√	√	√
17	HFE-150P	1.25"	√	√	√	√	√
18	HFE-150P	1.00"	√	√	√	√	√

IRI = Terminal IRI (in/mi), LC = AC Surface Down Cracking (Long. Cracking) (ft/mile), AC = AC Bottom Up Cracking (Alligator Cracking) (%), TC = AC Thermal Fracture (Transverse Cracking) (ft/mi), R = Permanent Deformation (Total Pavement) (in)
√ = Pass, **X =** fail

TABLE 29. Minimum HMA Overlay Thickness

Emulsion	Overlay Thickness
Reflex	1.75"
CSS-1h	1.25"
CSS-1h+Lime*	1.00"
HFE 150P*	1.00"

* Both CSS-1h+Lime and HFE 150P pass with 1.00 inch thickness of HMA overlay but further trials cannot be run, because M-EPDG software does not accept thickness below 1.00 inch

A trial run was performed to evaluate the effect of using the default G* and δ values of the binder. This was accomplished by changing the PG grade for the base binder of the ReFlex® emulsion and holding all other input parameters constant. No major change in performance parameters was indicated by the M-EPDG. The base binder input parameters (G*, δ) of the emulsion did not appear to have a major effect on the predicted performance when level 1 dynamic modulus values were used.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

Based on the results of this study and for the materials, test methods and equipment evaluated, the following conclusions are warranted.

Field Test Sites

1. The two CIR test sites on US 412 in Beaver County were showing more distress prior to rehabilitation, as measured by longitudinal and transverse cracking, than the test site on US 412 in Harper County. The Harper County test site received a more conventional mill with fabric interlayer prior to HMA overlay.
2. The two CIR test sections on US 412 in Beaver County are showing less transverse cracking than the fabric interlayer test section on US 412 in Harper County.
3. The use of the fly ash slurry crack injection procedure on one of the CIR test sections has not resulted in any measurable difference in performance between the two CIR test sections.
4. There is longitudinal wheel path cracking starting to appear on both CIR test sections on US 412 in Beaver County. Longitudinal wheel path cracking is considered evidence of top down fatigue cracking. The pavement section in the test sections consists of two inches of S-4 mix with PG 76-28 asphalt cement and a minimum of four inches of CIR with a base binder grade of PG 58-28 asphalt cement in the asphalt emulsion. A typical ODOT HMA section would have four inches of either a PG 70-28 or PG 76-28 HMA mix over a PG 64-22 HMA mix. This considerable difference in high temperature stiffness between the two layers, and the relatively thin stiff top layer, could have contributed to the wheel path cracking. Further investigation is warranted.
5. The above comments are based on three years of traffic. Additional traffic is required to fully evaluate the performance of the test sections and treatments.

Dynamic Modulus Testing

1. AASHTO TP 62 can be performed on CIR samples with slight modification to the test procedures.
   a. Samples should be compacted to the expected in-place air void content.
   b. The sample curing procedures of the Kansas DOT (21) should be followed.
   c. Samples can be frozen before sawing and coring to lessen the chance of damage to the test samples.
   d. The required mass of material for the 150 mm diameter by 175 mm tall sample did not fit into the compaction mold. Slight tamping down of the material was required.

2. Test temperature and frequency of loading had a major effect on the dynamic modulus values, which was as expected.
3. The emulsified asphalt cement in the CIR mixture had a significant effect on dynamic modulus. The stiffness trend of the CIR mixtures followed the stiffness trend of the base asphalt.
4. The use of hydrated lime, introduced as quicklime slurry, significantly increased the stiffness of the CSS-1h mix at all temperatures and frequencies tested.
5. CIR mixtures were not as stiff as typical HMA mixtures. Oklahoma HMA mixtures are generally made with stiffer asphalts than CIR mixtures.
6. CIR mixtures showed the same increase in stiffness with long-term oven-aging as did conventional HMA samples.
7. The equation for predicted dynamic modulus in the M-EPDG can be used to calculate dynamic modulus for CIR mixtures. To use the predictive equation the RAP should be treated as a "black rock" and the binder properties should be based on the base binder in the asphalt emulsion. The effective specific gravity, back-calculated from the Gmm of the RAP plus new emulsion, can be used as the bulk specific gravity of the black rock by assuming the absorption of the RAP is zero.
8. There was good agreement between the predicted dynamic modulus and the measured dynamic modulus at the two highest test temperatures measured, 37.8^{o}C and 21.1^{o}C. At the coldest test temperature measured, 4.4^{o}C, the predictive equation overestimates the dynamic modulus.
9. The M-EPDG can be used with CIR mixtures. When using a level 1 design the shear modulus and phase angle of the base binder in the emulsion are required as input parameters. Preliminary trails with the M-EPDG indicate that these values do not have a significant effect on outcomes.
10. The trial runs using the M-EPDG did not indicate top-down fatigue cracking as a distress mode even though the field test sections showed evidence of top-down fatigue cracking.

RECOMMENDATIONS

1. The two CIR test sections in Beaver County have less transverse cracking than the Harper County test section. Based on the limited observation time, CIR appears to be a viable procedure for rehabilitation of transverse cracked pavements.
2. Proper project selection is critical to CIR pavement performance. There was evidence of top-down fatigue cracking on US 412 prior to rehabilitation. The HMA overlay was three grades stiffer than the base binder in the CIR layer. The literature indicated that thin stiff layers over soft layers can lead to poor pavement performance (23). However, the M-EPDG did not indicate top-down fatigue cracking as a failure mode. A thorough investigation of the cracking on US 412 in Beaver County is warranted. In the interim, to help prevent top-down fatigue cracking, it is recommended that a maximum difference of two high temperature PG grades between pavement layers be maintained.
3. Numerous assumptions were required to use CIR in the M-EPDG. These assumptions need to be evaluated before the design guide can be used with

confidence. Additional research needs to be performed with different sources of RAP to determine the effect of RAP, and the aggregate type in the RAP, on the dynamic modulus. In the interim, the values for dynamic modulus shown in table 30 are recommended for use.

TABLE 30. Preliminary Recommended E* Values for CIR

Test Temp. (C)	Frequency (Hz)	Dynamic Modulus (psi)			
		Reflex	CSS-1h	CSS-1h + Lime	HFE 150P
4.4	10	831,000	950,500	1,331,000	1,132,000
	5	755,500	860,000	1,248,000	1,033,000
	1	580,500	671,000	1,046,000	818,500
	0.5	522,500	602,500	972,000	743,000
	0.1	398,500	465,500	789,000	581,500
21.1	10	398,500	353,000	807,000	479,500
	5	331,500	442,000	705,500	407,500
	1	218,000	285,000	506,000	270,000
	0.5	182,000	228,500	438,500	230,500
	0.1	127,000	150,500	314,000	170,000
37.8	25	225,000	282,000	565,500	238,000
	10	148,000	176,500	346,500	197,500
	5	114,000	135,000	275,000	164,500
	1	74,500	83,500	174,500	108,500
	0.5	52,500	66,500	145,000	91,500
	0.1	27,500	42,500	96,000	63,000
54.4	25	56,500	71,000	158,000	97,500
	10	38,500	53,000	124,000	85,000
	5	31,000	44,000	98,000	64,500
	1	15,500	29,000	61,000	37,500
	0.5	10,500	26,000	55,000	33,500
	0.1	6,000	20,000	36,000	24,000

4. Numerous binder input parameters are required to use the M-EPDG. Some of these input parameters are difficult to obtain for CIR mixtures because the amount of blending of old asphalt cement in the RAP with new asphalt cement from the emulsion can not be easily determined. The effect of asphalt cement properties on pavement performance from the M-EPDG needs to be evaluated. Using the properties of the base binder appears to be a reasonable assumption in the interim.

5. There were occasional problems with the M-EPDG software crashing and providing inconsistent results with the same data. It is recommended that each trial be rerun with the latest version of the M-EPDG to verify the results obtained in this study.

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