AN ABSTRACT OF THE THESIS OF

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Title: Evaluation of Crumb Rubber Modified Mixtures Using Performance Based Analyses

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Abstract Approved:__

James R. Lundy

This study investigated the laboratory performance of crumb rubber modified mixtures compared to a standard bituminous mixture using performance based test procedures. This study was part of an asphalt resurfacing program for the Seattle Washington area. Laboratory analyses were used to estimate the long term pavement performance of these mixtures in the field. Six mixtures were tested: The standard Class 'A' surface mixture for the Seattle area, PlusRide II® base course gradation (dry process) using AC 5 and AR 4000W binder types, PlusRide II® surface course gradation (dry process) using AC 5 and AR 4000W binder types, and ARHM-GG surface course gradation using crumb rubber modified (CRM) AR 2000 (wet process). The performance based tests used on each mixture evaluated the different failure modes a pavement may encounter in the field: fatigue cracking, permanent deformation (rutting), thermal cracking, age hardening, and water sensitivity. Many of the tests used were developed by the Strategic Highway Research Program (SHRP) to

test for a mixtures susceptibility in these failure modes.

Test results indicate the CRM mixtures performed better than the Class 'A' surface mixture, with respect to fatigue cracking. All of the PlusRide II® mixtures performed inadequately when tested for permanent deformation. On the other hand, the ARHM-GG surface mixture performed well, even better than the Class 'A' surface mixture with respect to permanent deformation. The ARHM-GG surface mixture showed better low temperature characteristics when compared to the Class 'A' surface and PlusRide II® mixtures. The CRM mixtures were less susceptible to aging than the Class 'A' surface mixture. Finally, all of the mixtures demonstrated low moisture sensitivity.

The final conclusions were made relative to the Class 'A' surface mixture. The ARHM-GG surface mixture performed as well as, and in some cases better than, the Class 'A' surface mixture. The ARHM-GG surface mixture may be used where the Class 'A' surface mixture was specified. The PlusRide II® base and surface (AC 5 and AR 4000W) mixtures did perform better than the Class 'A' surface mixture in some tests, however it performed worse in others. Therefore, it was recommended that the PlusRide II® mixture designs be re-evaluated to provide adequate performance in the failed tests.

EVALUATION OF CRUMB RUBBER MODIFIED MIXTURES USING PERFORMANCE BASED ANALYSES

by

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GLOSSARY

(Heitzman, 1992)

Asphalt Rubber (AR) -- An asphalt cement modified with crumb rubber modifier (CRM).

Buffing Waste -- A high quality scrap tire rubber which is a by-product from the conditioning of tire carcasses in preparation for retreading.

Crackermill -- A process that tears apart scrap tire rubber by passing the material between rotating corrugated steel drums, reducing the size of the rubber to a crumb particle generally 4.75 mm to 0.425 mm square (No. 4 to No. 40 sieve) in size.

Crumb Rubber Modifier (CRM) -- A general term for scrap tire rubber that is reduced in size and used as a modifier in asphalt paving materials.

Dry Process -- Any method that mixes the crumb rubber modifier with the aggregate before the mixture is charged with asphalt binder. This process only applies to hot mix asphalt production.

Granulated CRM -- Cubical, uniformly shaped, crumb rubber particles with a low surface area. This is usually produced by a rubber granulator.

Granulator -- A process that shears apart the scrap tire rubber, cutting the rubber with revolving steel plates that pass at close tolerances, reducing the size of the rubber to a crumb particle generally 9.5 mm to 2.00 mm square (3/8 inch to No. 10 sieve) in size.

Reaction -- The interaction between asphalt cement and crumb rubber modifier when blended together. The reaction, more appropriately defied as polymer swell, is not a "chemical reaction". It is the absorption of aromatic oils from the asphalt cement into the polymer chains of the crumb rubber.

Rubber Aggregates -- Crumb rubber modifier added to hot mix asphalt mixtures using the dry process which retains a physical shape and rigidity.

Wet Process -- Any method that blends crumb rubber modifier with the asphalt cement prior to incorporating the binder in the asphalt paving project.

Recycled Rubber -- Any crumb rubber derived from processing whole scrap tires or shredded tire material taken from automobiles, trucks or other rubber tired equipment.

EVALUATION OF CRUMB RUBBER MODIFIED MIXTURES USING PERFORMANCE BASED ANALYSES

1.0 INTRODUCTION

In recent years, there has been an increase in the use of additives and modifiers in asphalt concrete (AC) mixtures. The use of rubber has, since the early 60's, been a contentious issue in AC mixtures (Heitzman, 1992). There has been much rhetoric and research generated over the use of rubberized mixtures. This is a comprehensive evaluation of Crumb Rubber Modified (CRM) mixtures placed in the Seattle, Washington area. Virtually all components of pavement performance (developed through performance based tests) were evaluated and compared to the standard pavement specified by the Seattle Engineering Department (SED). Cutting through all the speculation and rhetoric, this evaluation of CRM mixtures may shed some light on field performance.

As part of the SED 1993 Arterial Asphalt Resurfacing Program, three road sections around the Seattle area were paved with asphalt mixtures containing CRM. An opportunity to conduct extensive pavement evaluations was provided by support from the Clean Washington Center (CWC), which is a division of The Department of Trade and Economic Development for the State of Washington. Specifically, the project was aimed at testing the mixtures using performance based testing equipment and protocols, most of which were developed under the Strategic Highway Research Program (SHRP). Evaluation of CRM mixture performance was not feasible with conventional Marshall or Hveem AC tests. The very empirical nature of the Hveem and Marshall methods do not lend themselves to be adequate predictors of field performance of unconventional mixtures (Asphalt Institute, 1989). Therefore, it was anticipated that the performance based test results and accompanying analyses would provide insight into the expected field performance of CRM mixtures.

The CRM mixtures investigated in this study have been dubbed the "dry" or "wet" process by the CRM industry (Heitzman, 1992). The "dry" process is defined as any method that mixes the CRM with the aggregate before the mixture is charged with asphalt binder. The "wet" process involves any method of mixing the CRM with the asphalt binder, then charging the aggregate mixture with the binder-CRM combination. This report investigates and compares both types of CRM mixtures, two involving the dry process--following the PlusRide II® specifications, and one using the wet process--following the Eagle Crest specifications.

1.1 Background

To appropriately evaluate mixtures with performance based analysis, the modes of failure for these mixtures must be defined. This background defines the various categories of pavement performance. A discussion of the Seattle Engineering Department's pavement evaluation program and the location of test sections, for future reference, is included. Also, a background on CRM legislation and a discussion of previous studies using CRM mixtures is given.

1.1.1 Legislation of CRM Use

The Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) contains legislation for the use of recycled material in asphalt concretes (Envirotire, 1992). Within the transportation engineering profession, there was much uproar because Congress legislated the use of CRM materials without adequate performance knowledge of CRM pavements (Kuennen, 1993). It was argued that engineers, not Congress, should specify material usage. Item 45: Section 1038. USE OF RECYCLED PAVING MATERIAL states that there would be no government disapproval (or any state acting as the governments authority) of patented products or procedures which use "recycled rubber". The term "recycled rubber" is defined as any crumb rubber derived from processing whole scrap tires or shredded tire material taken from automobiles, trucks, or other equipment owned and operated in the United States.

The most controversy, over Section 1038, was generated when congress mandated a percentage of recycled rubber in all federally funded projects. This percentage would then increase by 5 percent until 1997:

> *(1) State certification--Beginning on January 1, 1995, and annually thereafter, each State shall certify to the Secretary that such State has satisfied the minimum utilization requirement for asphalt pavement containing recycled rubber established by this section. The minimum utilization requirement for asphalt pavement containing recycled rubber as a percentage of the total tons of asphalt laid in such State and financed in whole or part by any assistance pursuant to title 23, United States Code, shall be--*
>
> *(A) 5 percent for the year 1994;*
> *(B) 10 percent for the year 1995;*
> *(C) 15 percent for the year 1996; and*
> *(D) 20 percent for the year 1997 and each year thereafter.*

The Transportation Secretary has the option of increasing the minimum utilization requirement of paragraph (1) above for CRM pavements. A Secretarial waiver is available to states that prove a health risk from CRM pavements, that CRM pavement can not be recycled, or that CRM pavements do not perform adequately. If the DOT does not follow these guidelines, the Secretary has the option to withhold federal funds from any state that fails to make the certification that they did indeed use CRM in their federally funded projects.

Part of this legislation included a provision for the study of CRM materials in pavements (Envirotire, 1992). Studies would be federally funded through the FHWA. Results would be collected and coordinated by the Secretary of Transportation and the Administrator of the Environmental Protection Agency. Issues to be addressed by these studies include:

- Determination of a threat to human health and environment associated with the production and use of asphalt pavement containing recycled rubber,
- Determination of the recyclability of CRM pavements, and
- A performance evaluation of CRM asphalt pavement, in the field.

1.1.2 Development of Performance Based Tests

Just as the ISTEA legislation was enacted, another federally funded project was winding down. The Strategic Highway Research Program (SHRP) was a five year study funded under the 1987 Surface Transportation and Uniform Relocation Assistance Act (Bell and Leahy, 1994). This $150 million research project involved many aspects in the transportation field. Of the $150 million, $50 million was reserved for research and development of state of the art tests and specifications for binders and asphalt-aggregates mixtures. Part of SHRP's scope involved development of performance based tests and procedures, to not only assess the viability of standard bituminous concrete mixtures but also modified mixtures. Many of the performance based tests for bituminous mixtures were developed by the University of California at Berkeley and Oregon State University under the SHRP A-003A Contract (Bell and Leahy, 1994).

Conventional mixture design methods used by highway agencies, were usually based on either the Hveem or Marshall procedures (Asphalt Institute, 1989). In these procedures air voids, mixture stability, and other factors were used to help determine the optimum asphalt content. However, a shortcoming of the Hveem and Marshall procedures used to judge mixture quality are empirical and do not apply to unconventional mixtures (Bell and Leahy, 1994). Unconventional mixtures often include modifiers, such as CRM or polymers, which will not have a history of field performance. Hveem or Marshall test results cannot be extrapolated to evaluate unconventional mixture performance in the field. Hence, performance based testing procedures are better predictors of field performance.

1.1.3 Previous studies with CRM materials

Paving projects using various forms of CRM asphalt concrete were initiated in Washington and Oregon circa 1982 (Terrel et al, 1993). Most studies have involved extensive field surveys with some laboratory evaluations (Miller, 1992).

Research by EnviroTire Inc. on their CRM PlusRide® product (EnviroTire, 1992), claims:

1) Reduced reflective and thermal pavement cracking,
2) Increased resistance to studded tire wear,
3) Increased friction (skid resistance) between tires and pavement,
4) Easier ice removal through elastic deformation of the rubber granules under traffic loading and vehicle generated winds, and
5) Increased suppression of pavement tire noise.

In Washington (Anderson and Jackson 1992), CRM experience has been mixed; some pavements have performed well for up to 7 years, while others pavements were reported as dramatic failures. Open graded CRM friction courses appear to do as well, but no better than conventional binders. The WSDOT does not have pavements containing Asphalt Rubber Hot Mix -- Gap Graded (ARHM-GG) mixtures, only open graded friction courses. Their experience with dense-graded or gap-graded mixtures (PlusRide®) was not exceptional as noted in their report (Anderson and Jackson, 1992).

> *"The performance of PlusRide® pavements ranges from satisfactory to immediate failure and replacement with standard ACP. In WSDOT's*

> *experience, there is no indication of better performance or longer life. In fact, the opposite appears to be true. Maintenance forces note no savings in snow removal nor any less ice forming on rubber asphalt test sections."*

The Oregon DOT has utilized CRM asphalt mixtures in several locations throughout the state dating back to 1985 (Nodes, 1992) with experimental installations still underway. ODOT reports that the PlusRide® product has resisted cracking. However, the loss of large aggregate from the wheel tracks casts doubts on the long-term pavement durability (Miller, 1990). Oregon projects do have mixture evaluations with resilient modulus (M_R) data, fatigue data, OSU Wheel Tracker data, and stripping evaluations. Final pavement performance assessment on CRM projects in Oregon is pending.

1.1.4 Seattle Engineering Department's Pavement Evaluation Program

Three test sites were selected from the Seattle 1993 paving program. The general contractor for the Seattle Engineering Department's (SED) paving program was Lakeside Industries, Inc.; the subcontractors supplying CRM technology and materials to Lakeside were EnviroTire, Inc. for the PlusRide II® mixtures, and Eagle Crest Construction, Inc. for the ARHM-GG mixtures. The evaluated construction sites which include CRM and standard mix (Class 'A') control sections are provided here for future reference and study associated with this laboratory research. The location and cross-sections of the field evaluation sites are:

1. 5th Avenue (Denny Way to Olive Street),

- Control section with 5 cm (2-in.) of Class 'A' surface course, and
- CRM section with 4.3 cm (1.7-in.) of PlusRide II® (AR 4000W) surface course,

2. Airport Way South (5th Avenue South to South Spokane Street),
   - Control section with 3.8 cm (1.5-in.) of Class 'A' surface course,
   - CRM section with 3.1 cm (1.2-in.) of ARHM-GG (AR 2000) surface course over 24.1 cm (9.5-in.) of PlusRide II® (AC 5 and AR 4000W) base course.
3. North East 145th Street,
   - Control section with 3.1 cm (1.2-in.) of Class 'A' surface course over 14.0 cm (5.5-in.) of standard asphalt treated base (ATB),
   - CRM section with 4.1 cm (1.6-in.) of ARHM-GG (AR 2000) surface course over 14.0 cm (5.5-in.) of PlusRide II® (AR 4000W) base course.

Eight test sections, approximately one block in length, within each of the three sites were identified for detailed evaluation. The pavement information collected prior to construction included the following:

1. History of pavement construction,
2. Structural pavement design,
3. Pavement condition survey,
4. Detailed crack map of the test sections to be overlaid,
5. Falling Weight Deflection (FWD) measurements with back calculation of base and subgrade modulus values, and

6. Maps of each project section showing test sites.

The N.E. 145th Street project does not have data on deflection or surface conditions since it was reconstructed rather than overlaid.

1.1.5 Categories of Performance Based Mixture Analyses

The performance of pavements was divided into five categories by SHRP (Bell and Leahy, 1994). These categories describe the modes of failure a pavement may encounter during its operating life. These categories are:

- Fatigue Cracking,
- Permanent Deformation (rutting),
- Thermal Cracking,
- Age Hardening, and
- Water Sensitivity.

Fatigue Cracking

A flexible pavement is subject to continuous flexing and relaxation through repeated traffic loading. Fatigue cracking occurs when the underside of bituminous layers are subject to repeated tensile strains. Fatigue is defined as: *"The phenomenon of fracture under repeated or fluctuating stress having a maximum value generally less than the tensile strength of the material."* (Bell and Leahy, 1994)

Fatigue cracking is mainly dependent on mixture stiffness, mix type, traffic loading, supporting layer strength, and temperature conditions. For pavements with thick bituminous layers, good fatigue performance is associated with high stiffness, dense grading, and low air voids. Where granular layers are utilized as the main structural component and surfaced with thin bituminous layers, better performance is achieved with flexible mixes (Roberts et al, 1991).

Permanent Deformation

The cause of permanent deformation (rutting) is the progressive movement of materials under repeated loads whether in the bituminous mixture, underlying base course layers, or within the subgrade (Roberts et al, 1991). Three stages of permanent deformation of a mix have been defined (Carpenter, 1993):

1) Primary -- initial densification of a mixture,

2) Secondary -- stable shear period in a mixture, and

3) Tertiary -- rapid unstable shear failure of a mixture.

Figure 1.1 shows the three stages of this permanent deformation. To judge the long-term performance, two factors are considered; (a) the number of repetitions at which a critical rut depth is reached, and (b) how rapidly a mixture reaches the tertiary failure zone. The criteria discussed are not mutually inclusive. A mixture may reach critical rutting before it becomes unstable or it may become unstable before it develops a critical rut depth. For evaluation, it is important to separate these two occurrences and describe how they develop (Carpenter, 1993).

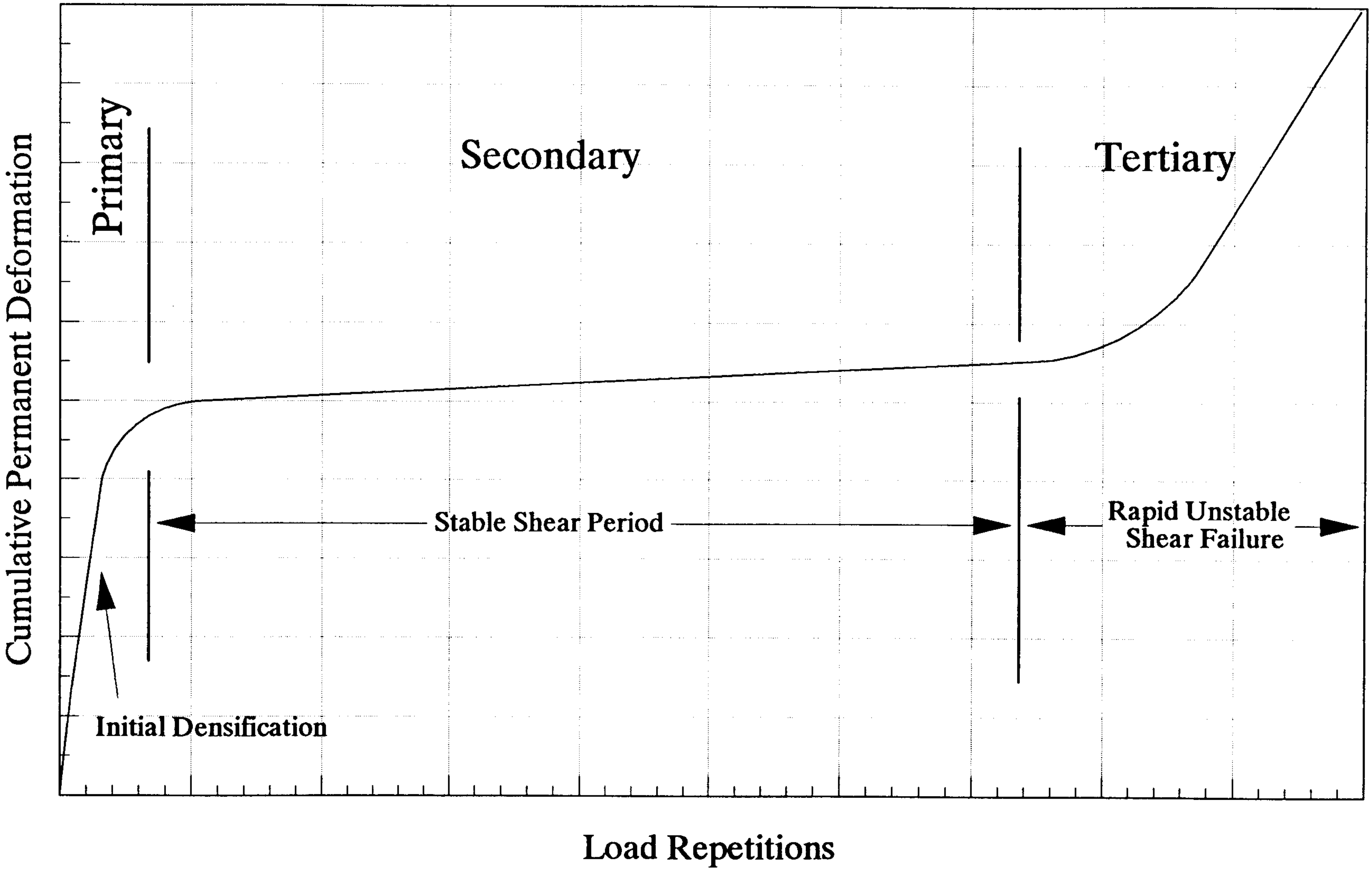


Figure 1.1 Three stages of Permanent Deformation (Carpenter, 1993)

Permanent deformation may also result from lateral plastic flow. If a mix design fails due to lateral plastic flow, it may be from excess binder. Within a mixture excess binder causes aggregates to float past each other and hence lose aggregate interlock. Minimization of plastic flow can be achieved through the use of large sized aggregates, angular and rough textured coarse and fine aggregates, and appropriate mixture compaction in the field (Roberts et al, 1991).

Thermal Cracking

When pavements cool they contract (Kanerva et al, 1992). The friction between the base and the surface layer prevent some of this contraction. Low-temperature cracking is attributed to tensile stresses induced in asphalt concrete pavements. When the tensile stresses equal the asphalt concrete mixture strength, a micro-crack develops at the edge or surface of the asphalt mixture. Under repeated temperature cycles, the crack eventually propagates through and across the surface layer. Once a crack is through a pavement structure, migration of water and fines into and out of the structure can create pavement degradation. Thermal cracking tests simulate this process such that a mixture can be evaluated for its low temperature cracking resistance (Jung et al, 1993). Generally, mixtures with the combination of colder fracture temperatures and higher fracture strengths are preferable (Terrel et al, 1993).

Several factors are reported to contribute to the thermal cracking effects in an asphalt concrete pavement. These are broadly categorized under: material, environmental, and pavement structure geometry (Jung et al, 1993). Specific factors within each of these categories are:

- Material Factors -- asphalt properties (stiffness or consistency), aggregate type, gradation of aggregate, asphalt content, and air void content within a mixture,
- Environmental Factors -- air temperature, rate of cooling, and pavement age, and
- Pavement Structure Geometry -- pavement width, thickness, friction between the pavement and base course, subgrade type, and construction flaws.

Age Hardening

It has been reported that significant aging is associated with the initial mixing of the material in a drum mixer or pugmill (Roberts et al, 1991). During mixing, the binder is exposed to air at temperatures which range from 133C to 163C (272°F to 325°F). Substantial rheological changes occur when the binder is exposed in a thin film state. These changes include a decrease in penetration and increase in viscosity due to air oxidation and loss of volatile components contained within the binder (Roberts et al, 1991).

After this initial short term aging, the bituminous mixture is exposed to six components of long term aging, any one of which may be prevalent depending on the environment to which the mixture is exposed. These six components are (Roberts et al, 1991):

Oxidation: Where the reaction of oxygen with asphalt cement stiffens the mixture. The rate of oxidation depends on the bituminous mixture character and the temperatures to which the pavement is exposed.

Volatilization: Where the evaporation of the lighter binder constituents from the mixtures stiffens the pavement. This loss is mostly a function of in service pavement temperature.

Polymerization: Where the molecules, similar in nature, form larger molecules, causing progressive hardening. There is much speculation as to the effect of this in the field performance of pavements, however it is believed to be a small component related to mixture aging.

Thixotropy (stearic hardening): This progressive hardening is due to the formation of a structure within the asphalt cement over a long period of time. This type of aging occurs in pavements which almost never have repeated loading (traffic) applied to them, such as road shoulders. A combination of higher temperatures and repeated loading will reverse this thixotropic process.

Syneresis: This hardening effect is caused by the exudation reaction of the thin oily liquids to the binder surface. With the elimination of these oily constituents, the binder becomes stiffer.

Separation: This occurs when porous aggregate removes the oily constituents, resins, or asphaltenes from the binder.

Volatilization and oxidation are the most important factors explained above. Also, these are the factors over which the engineer has the most control (Bell and Leahy, 1994). These two hardening effects are mostly controlled by the air void amount and air void nature within the mixture. When a pavement contains low air voids or is impervious, oxidation and volatilization will be minimized with respect to environmental factors (Bell and Leahy, 1994).

Water Sensitivity

Water sensitivity involves a mixtures ability to retain its original strength when exposed to environmental conditions (Allen, 1993). Degradation resulting from this "water damage" is usually from a combination of traffic and water within an asphalt mixture. This environmental damage appears as potholes, permanent pavement deformations, flushing, mixture raveling, or loss of mixture stiffness (Allen, 1993). Three main components of failure in moisture sensitive mixtures have been identified as loss of adhesion, loss of cohesion, and aggregate degradation.

Loss of adhesion occurs when there is a loss of bond between the asphalt binder and the aggregate due to water between the asphalt film and the aggregate. The aggregate is then left "stripped" of its asphalt film coating. Pavement failure occurs in two stages: first, stripping failure occurs; then the pavement failure due to the action of traffic (Allen, 1993).

The loss of cohesion occurs when water enters the asphalt binder matrix. Saturation and expansion of the void system may then occur within the asphalt concrete mixture (Al-Swailmi, 1992). It has been documented that asphalt mixtures have swelled, or increased in volume due to water intrusion. Finally, this could cause elongation and weakening of the asphalt films that bind the aggregate matrix.

Aggregate degradation has shown to have similar effects to geological weathering. A loss of integrity in the aggregate is mainly due to the effects of chemical and/or mechanical weathering. The cycling of water and temperature, as in native rock, are the major components of this degradation (Allen, 1993).

1.2 Objectives

The objective of this study was to determine which of the three CRM mixtures meet or exceed the performance of the standard (non-modified) surface mixture used by the Seattle Engineering Department (SED). This comparison was based on performance tests, including those used to characterize fatigue cracking, permanent deformation, thermal cracking, aging, and water sensitivity. Within each of these criteria, each mixture will be compared and ranked according to its resistance to failure.

2.0 EXPERIMENTAL PROGRAM

To understand the characteristics and properties of each CRM mixture and how they differ from a conventional mixture, a material description, sample preparation, and a brief description of each procedure is presented. Also, a brief description of the statistical methods used to compare and rank the mixtures is discussed.

The materials used in this study involved one aggregate source, one source of granulated rubber for the "dry process", and another source of asphalt rubber binder for the "wet process". Mixing and compaction of test specimens followed the Rolling Wheel Compaction procedure (Appendix A). After the samples were either cut or cored, they were evaluated under the criterion of the following performance based test procedures:

- Flexural Beam Fatigue Test--Controlled Strain (FBFT-CS)
- Repetitive Shear Strain Test--Constant Height (RSST-CH)
- OSU Wheel Tracker
- Thermal Stress Restrained Specimen Test (TSRST)
- Aging (Short and Long Term)
- Environmental Conditioning System (ECS)

From the data, multiple range analysis was used to evaluate and rank the results of each mixture. Hence, conclusions and recommendations were based on the laboratory performance of each mixture.

2.1 Materials

2.1.1 Binders

Three asphalt binders were used in this project: AC 5, AR 4000W and CRM modified AR 2000. Pavebond™, an antistrip agent, was added to all binders by binder weight prior to mixing with the aggregates. The U.S. Oil refining company provided the AC 5 asphalt. The AR 4000W binder was provided by Chevron, USA. The CRM modified AR 2000 asphalt was supplied by Chevron, USA; it arrived at the OSU laboratory with rubber granules premixed and in suspension. All unmodified binders met the specifications required by ASTM D 3381.

With the addition of rubber granules to the AR 2000 binder, asphalt properties change. Table 2.1 shows the results of binder tests performed by Petroleum Sciences, Inc. on the specially blended AR 2000. The chosen rubber concentration was 17.5%, based on the total weight of binder.

2.1.2 Aggregate

The aggregate source for all the mixtures was obtained from a Lakeside Industries quarry in Issequah, Washington. The job mix gradation for each mixture is shown comparatively in Table 2.2 with Figures 2.1 through 2.4 showing a graphical representation of each mixture's gradation. These figures include the gradation of CRM added to the mixtures.

Table 2.1 CRM Modified (AR 2000) Binder Properties

Test Type	Time (Hrs)					Specified at First Hour
	1	1.5	2	6	24	
Brookfield Viscosity (177C (350 F), Sp 3, 20 RPM)	2850	2850	2250	2800	2800	1500-6000
Softening Point C (F)	69.4 (157)	72.8 (163)	67.2 (153)	71.1 (160)	66.1 (151)	130 min
Penetration (needle) 25C (77 F)	50	52	47	57	56	25-75
4C (39.2 F)	27	30	30	30	28	15 min
Penetration (cone) 25C (77 F)	47	47	47	52	54	--
Resilience (%)	41.5	50	43.5	48	45	20 min
RTFO Residue Penetration (needle) 4C (39.2 F)	22					--
% Retained	81.4					75 min

Table 2.2 Job Mixture Formulas

Specified Sieve Designation (Standard)	Class 'A' Gradation (Percentage Passing)	PlusRide II® Gradation (Percentage Passing)			ARHM-GG Gradation (Percentage Passing)		Specified Sieve Designation (Alternative)
	Aggregate	Aggregate		CRM	Aggregate	CRM	
	Surface	Base	Surface	Base & Surface	Surface	Surface	
19.0 mm	100	100	100	100	100	100	3/4 inch
16.0 mm	100	98	100	100	100	100	5/8 inch
12.5 mm	NS*	NS	88	100	98	100	1/2 inch
9.5 mm	85	53	61	100	74	100	3/8 inch
6.3 mm	68	40	42	100	50	100	1/4 inch
4.75 mm	NS	NS	NS	95	38	100	#4
2.36 mm	NS	NS	NS	NS	22	100	#8
2.00 mm	37	30	27	36	NS	100	#10
1.18 mm	NS	NS	NS	NS	NS	94	#16
850 m	NS	NS	NS	24	NS	NS	#20
600 m	17	16	16	NS	13	34	#30
300 m	NS	NS	NS	NS	NS	8	#50
75 m	6	7	9	NS	5	0	#200
Binder Content (%)	4.7	7.5			8.0†		Binder Content (%)
Pavebond‡ (%)	0.5	0.75			0.5		Pavebond‡ (%)
CRM (%)	None Added	3			1.2‡‡		CRM (%)

*Note: Not Specified (NS) in specifications.
‡Note: Percentage by weight of asphalt.

†Note: Contains 6.8% asphalt and 1.2% CRM.
‡‡Note: Added as part of 8.0% binder.

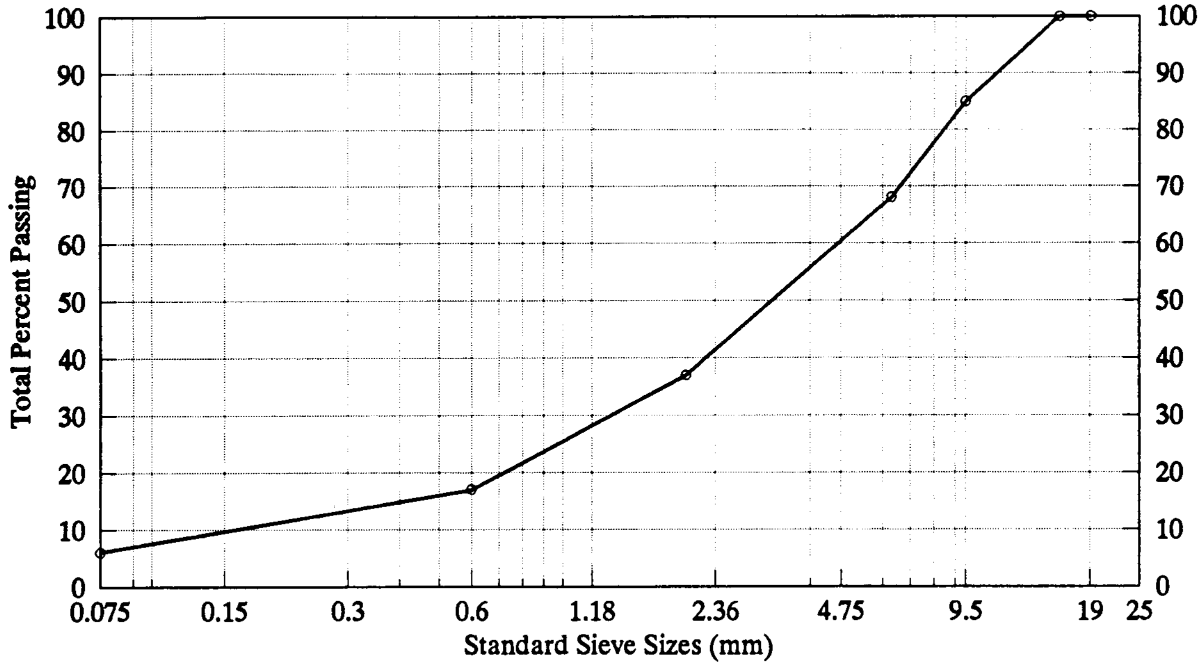


Figure 2.1 Aggregate Gradation for Class 'A' Surface Mixture

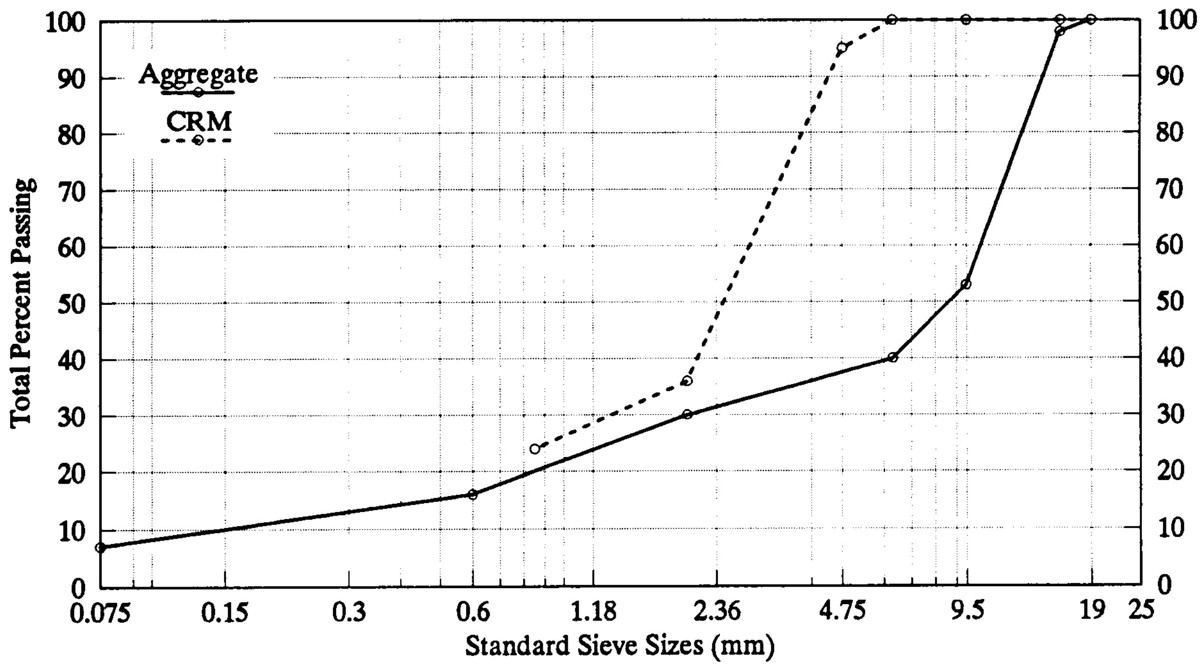


Figure 2.2 Aggregate Gradation for PlusRide II® Base Mixture

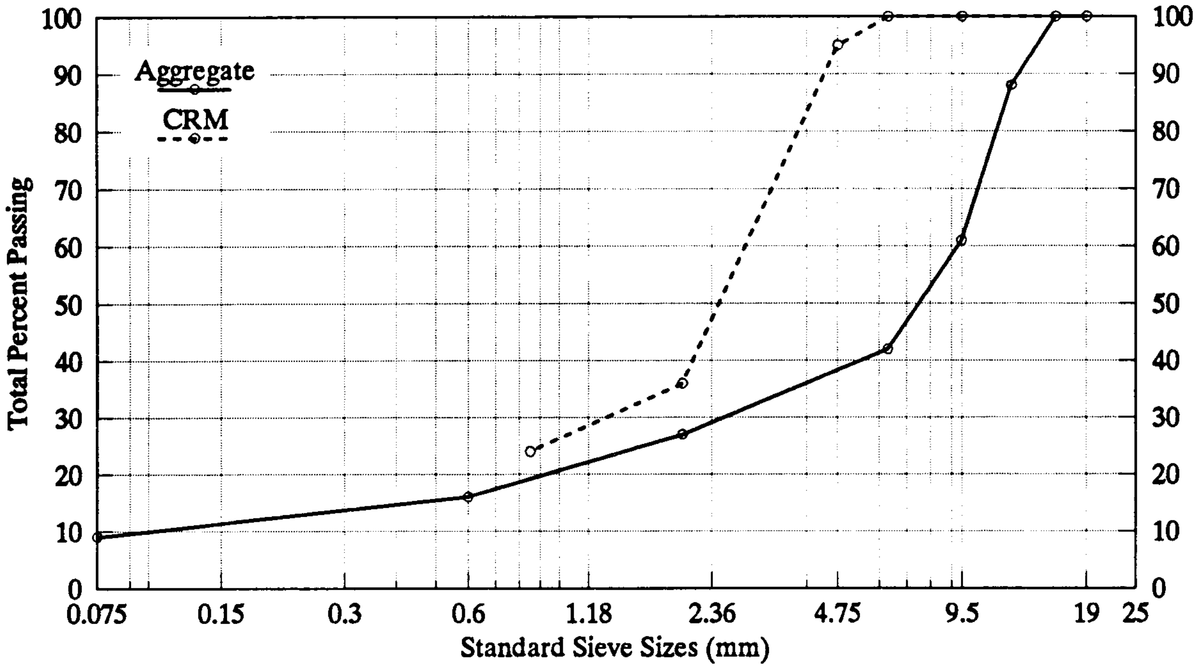


Figure 2.3 Aggregate Gradation for PlusRide II® Surface Mixture

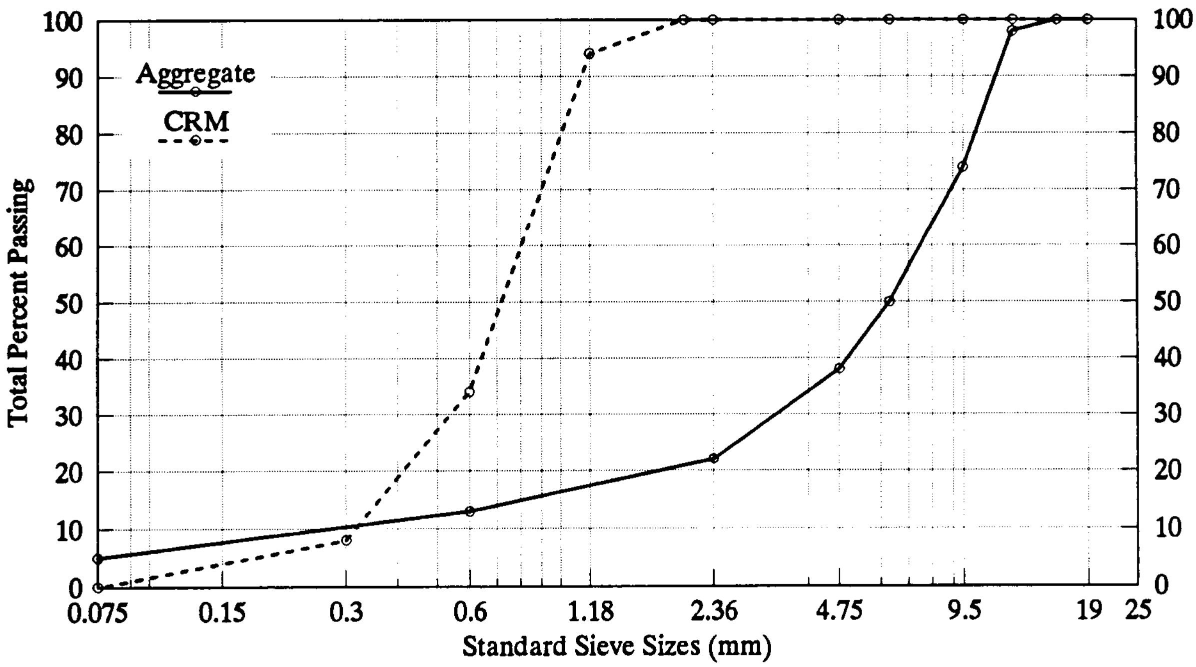


Figure 2.4 Aggregate Gradation for ARHM-GG Surface Mixture

Granulated rubber for the dry process was provided in two size fractions from Rubber Granulators Inc. in Everett, Washington. The coarse rubber ranged from the 6.3 mm (1/4-in.) to the 2.00 mm (No.10) sieve. The fine rubber particles were between the 2.00 mm (No. 10) and 850 m (No. 20) sieves. These sizes were blended to achieve the final gradation shown in Table 2.2 and Figures 2.2 and 2.3.

2.2 Specimen Preparation

Three different agencies were involved in the design of the four mixtures used in this investigation. These include:

1. Conventional Class 'A' surface (Control Mix)--designed by WSDOT,
2. PlusRide II® base (dry process)--designed by Ground Engineering Inc.,
3. PlusRide II® surface (dry process)--designed by Ground Engineering Inc., and
4. ARHM-GG surface (wet process)--designed by Petroleum Sciences Inc.

2.2.1 Preparation of Test Specimens

Fourteen roller-compacted slabs approximately 61 cm x 76 cm x 10 cm (24-in. x 30-in. x 4-in.) thick were produced as shown in Table 2.3. Each slab was cut or cored to the desired testing size. The first slab (number 1) of Class 'A' surface mixture is not shown in Table 2.3 This was a laboratory compaction test slab and was not used in any analysis. The first slab (number 1) of PlusRide II® (AC 5) base did not meet the three percent air void requirement, as specified by Ground Engineering Inc. However, this slab presented an opportunity to perform some tests and compare the PlusRide II® (AC 5) mixtures at a normal and low air void content.

During the investigation of the PlusRide II® mixtures, it was discovered that the AC 5 made the mixtures susceptible to permanent deformations. The use of AC 5, as originally specified, proved to be too soft. Evidence of this potential for permanent deformation was observed in the laboratory and in the field sections. Standing loads (such as a pick-up truck) caused significant deformation on the day following construction. In light of these problems, the SED chose to substitute AR 4000W for AC 5 in the PlusRide II® mixtures. The laboratory research was extended to include the PlusRide II® (AR 4000W) mixtures.

Pavebond™, an anti-strip additive, was added to the control mixtures, ARHM-GG mixtures and PlusRide II® (AR 4000W) mixtures (see Table 2.3). In order to simulate harsher conditions experienced with the asphalt/aggregate/CRM mixtures, Pavebond™ was not used in the PlusRide II® (AC 5) mixtures. It may also be noted that 0.55% pavebond was added to Class 'A' surface mixture. The normal amount of Pavebond™ added should have been 0.50%. This additional amount was added inadvertently to the mixture and is only noted here.

Table 2.3 Laboratory Mixture Summary

Mixture	Slab Number	Asphalt Type	Binder Content* (%)	Pavebond† (%)	Target Air Voids (%)	Mix Temperature C (F)	Compaction Temperature C (F)
Class 'A' Surface	2	AR 4000W	4.7	0.50	7.0	160 (320)	135 (275)
	3			0.55			
PlusRide II® Base	1	AC 5	7.5	0	2.9	150 (302)	140 (284)
	2						
	3						
PlusRide II® Base	1	AR 4000W	7.5	0.75	2.9	160 (320)	135 (275)
	2						
PlusRide II® Surface	1	AC 5	7.5	0	3.2	150 (302)	140 (284)
	2						
PlusRide II® Surface	1	AR 4000W	7.5	0.75	3.2	160 (320)	135 (275)
	2						
ARHM-GG Surface	1	AR 2000	8.0	0.50	7.1	163 (325)	135 (275)
	2						

* Note: Percentage by weight of total mixture.

†Note: Percentage by weight of asphalt.

2.2.2 Mixing and Compaction Method

When all the materials and mixture designs were received from the suppliers, large batches of each mixture were prepared using the roller compaction procedure developed at OSU (Appendix A). About 136.1 kg (300 lbs.) of pre-heated aggregate, binder and additives were mixed in a heated rotary mixer. Short term oven aging was used as a standard procedure (4 hours at 135C (275°F)) to simulate conventional field construction. The mixture was placed in a specially fabricated mold and compacted into a uniform slab, approximately 76 cm (30-in.) long by 61 cm (24-in.) wide by 10 cm (4-in.) high. The density and compaction of each mixture was controlled by weighing sufficient materials to produce a slab with the specified air void content. This method simulates field compaction and gives more representative results (Scholz et al, 1993).

After overnight cooling, the slab was de-molded, and the required test specimens were sawed or cored from the slab, as described in each respective test procedure.

2.3 Test Procedures

2.3.1 Flexural Beam Fatigue Test--Controlled Strain (FBFT-CS)

Beam specimens 5.1 cm x 6.4 cm x 38.1 cm thick (2-in. x 2.5-in. x 15-in.) were sawed from the roller compacted slabs produced in the OSU laboratory and tested in repeated flexure at University of California at Berkeley (UCB). After cutting, the specimens were measured for air void content. It is proposed that, in the most basic form, the fatigue resistance of a mixture is represented by the four point bending beam fatigue test (Sousa et al, 1993). This test imposes mixture displacement reducing the stiffness modulus to an assumed failure condition. This point of failure defines the fatigue characteristic by a relationship between level of strain applied to the mixture and the number of load repetitions to failure (Sousa et al, 1993).

The procedure basically consists of placing the specimen horizontally into the FBFT-CS testing frame, then clamping it into place. A computer controls the testing system, applying a load to produce a pre-determined constant strain. Loading was applied at third points along the beam to create a constant strain throughout the middle third of the beam. Sinusoidal loading was applied at a frequency of 10 Hz, at a constant temperature of 20C (68°F). Data were collected at pre-determined loading cycles during the testing sequence. The initial stiffness reading was taken after fifty loading cycles. Failure and test termination were defined when the specimen stiffness modulus was reduced to one-half its original value. Figure 2.5 shows a schematic view of the FBFT-CS apparatus.

2.3.2 Repetitive Shear Strain Test--Constant Height (RSST-CH)

The Repetitive Shear Strain Test--Constant Height (RSST-CH) was used to evaluate the susceptibility of an asphalt mixture to permanent deformation (i.e. rutting). Although permanent deformations may be caused by soft underlying layers, the asphalt concrete contributes to rutting by deforming in plastic shear flow in the

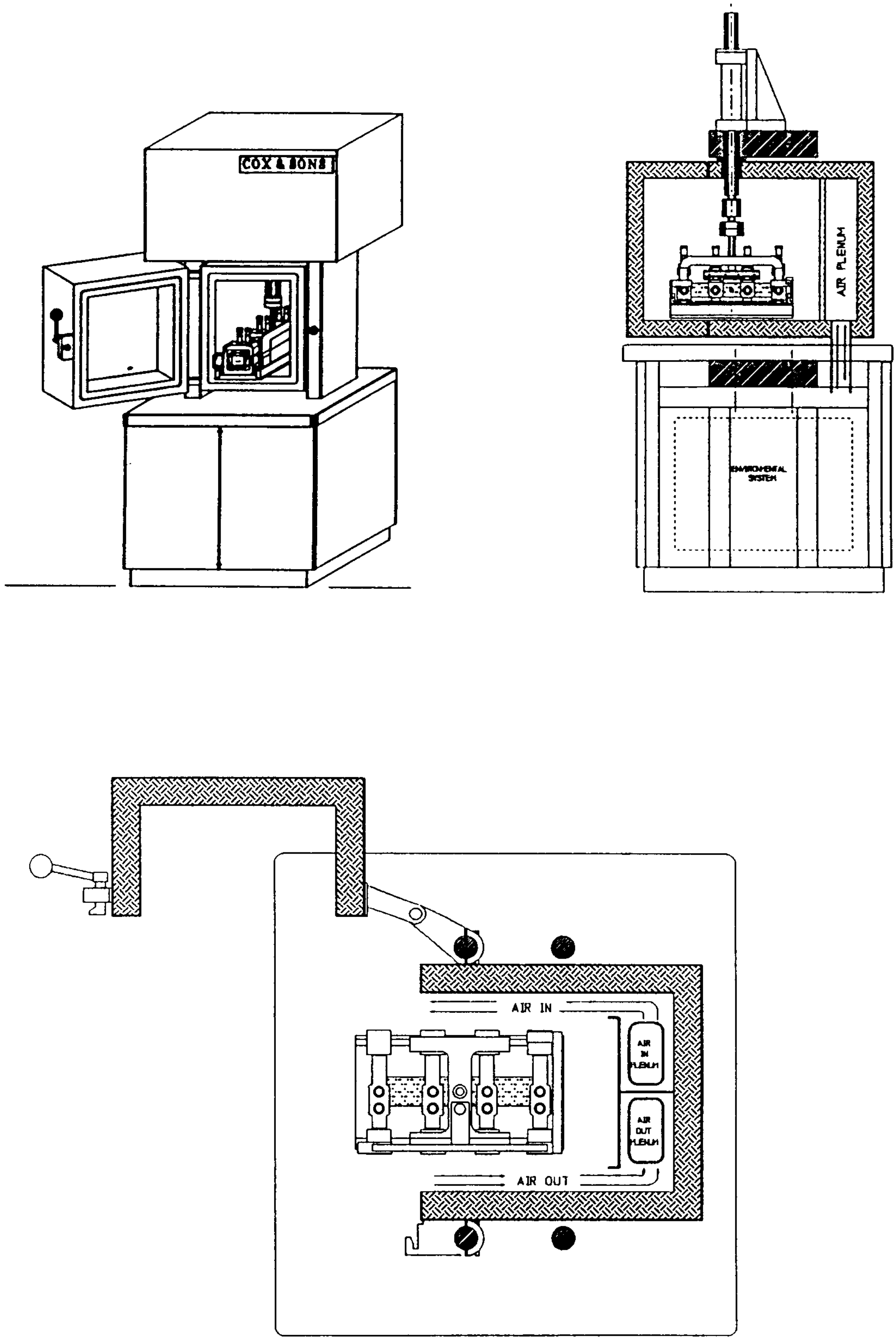


Figure 2.5 Schematic of Flexural Beam Fatigue Test--Controlled Strain

upper 5 cm to 8 cm (2 to 3-in.). The test results, coupled with appropriate design concepts, can be used to predict rutting depth for particular mixtures (Sousa et al, 1993).

Core specimens 15.2 cm dia x 10.2 cm (6-in. dia x 4-in.) high were cut from the roller compacted slabs. The specimens were then transported to UCB, where a 5.1 cm (2-in.) slice was extracted from the middle of the specimen leaving a 15.2 cm dia x 5.1 cm (6-in. dia x 2-in.) thick specimen. After cutting, the specimens were measured for thickness and air void content.

The specimens were epoxied to end platens. The end platens and the specimen were placed in a gluing jig to ensure that the platens were parallel and the specimen was square with the platens. After the epoxy had set, four holes were drilled in the specimen. Screws were glued into these holes to allow the attachment of a Linearly Variable Differential Transformer (LVDT) to monitor the shear deformation.

Prior to testing, the specimen was placed in an oven at the testing temperature of 50C (122°F) for a minimum of two hours and a maximum of four hours. Simultaneously, the Universal Testing Machine (UTM) was pre-conditioned to the same temperature. When both the specimen and the UTM reached 50C (122°F), the specimen was transported to the UTM in an insulated box. A vertical LVDT was attached to the end platens and a horizontal LVDT was attached to the screws mounted in the specimen. The entire specimen-platen assembly was then placed in the UTM and the hood was lowered into place and left for approximately 10 minutes for temperature stabilization.

Testing consists of applying a haversine wave pulse (a positive load where no load was applied at the wave bottom) to one of the end platens and holding the other fixed. The horizontal load was applied for 0.1 second followed by a 0.6 second rest period. A vertical compressive or tensile load was also applied to keep the specimen at a constant height. A micro-computer controls the horizontal and vertical loading and collects data at appropriate time intervals. The repeated shear load was applied until 5 percent permanent shear strain or 5,000 repetitions was reached. Figure 2.6 shows a schematic of the Repetitive Shear Strain Test--Constant Height procedure.

2.3.3 OSU Wheel Tracker

One concern of most transportation agencies is the potential for rutting of bituminous mixtures. The OSU Wheel Tracker was obtained from Laboratoires des Ponts et Chausse'es (LPC) in France to test the rutting sensitivity of mixtures (Figure 2.7). The Wheel Tracker is used extensively in France to determine rutting susceptibility of mixtures (Brousseaud et al, 1993).

In order to compare the mixtures, prismatic sections 17.7 cm wide x 48.3 cm long x 10.2 cm thick (7-in. x 19-in. x 4-in.) were sawed from the roller compacted slab and placed in a metal frame. The samples were confined with slices of foam matching the sample surface profile to which the frame was fitted. The sample and frame were attached to the metal base plate in the OSU Wheel Tracker and heated to a constant temperature of 50C (122°F). A pneumatic tire with an internal pressure of

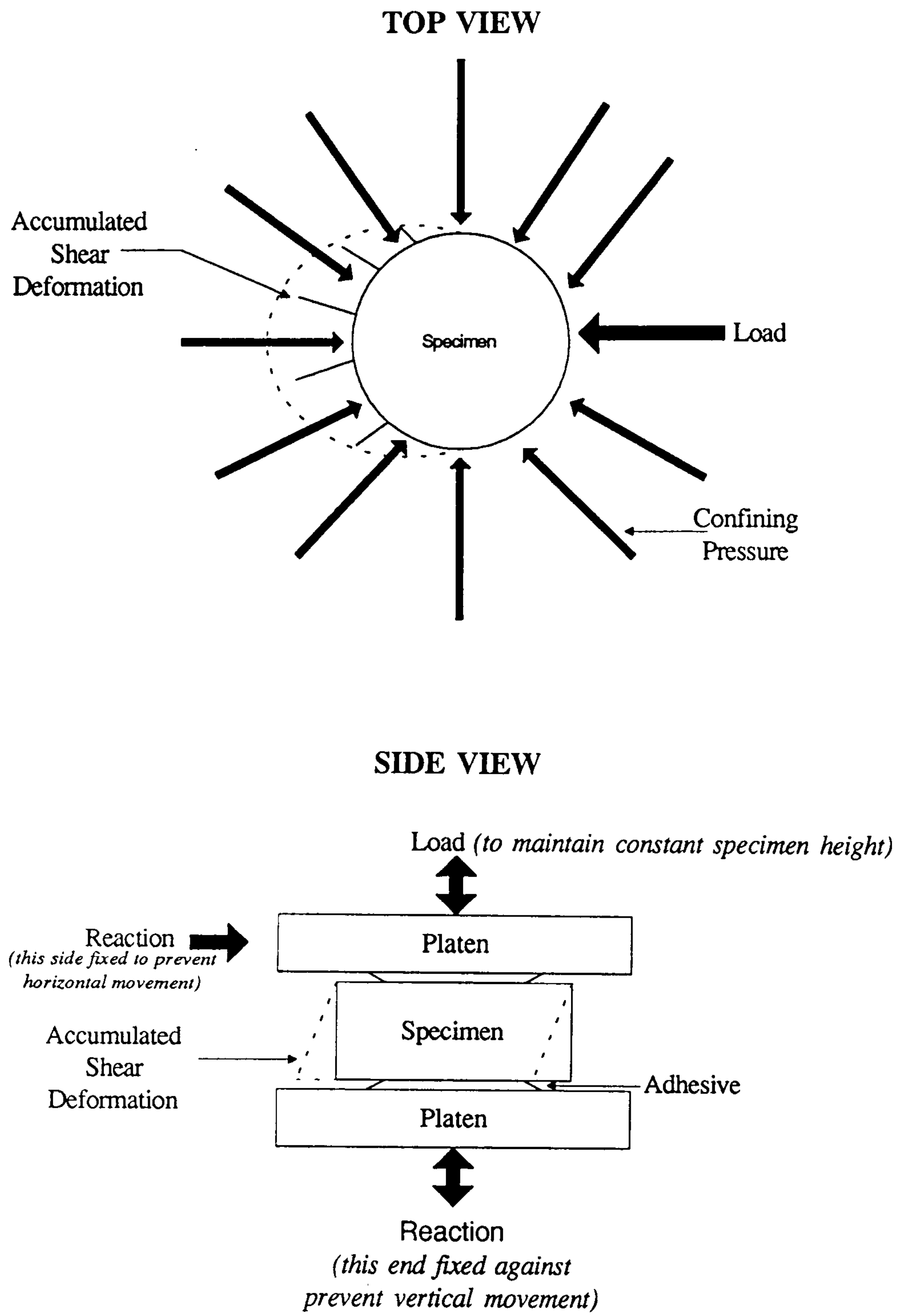
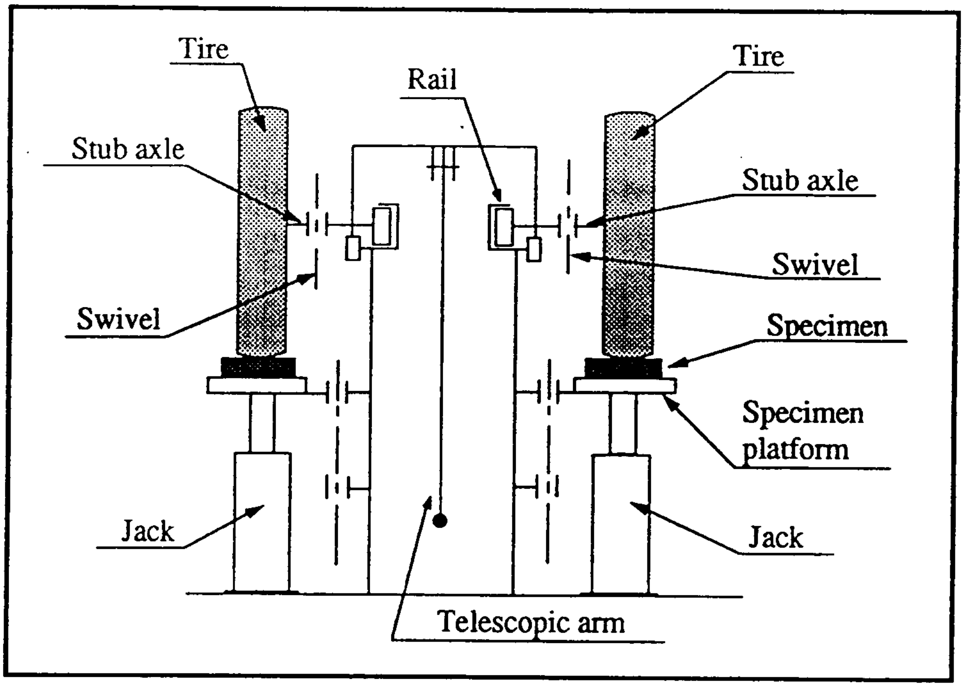


Figure 2.6 Schematic of Repetitive Shear Strain Test--Constant Height

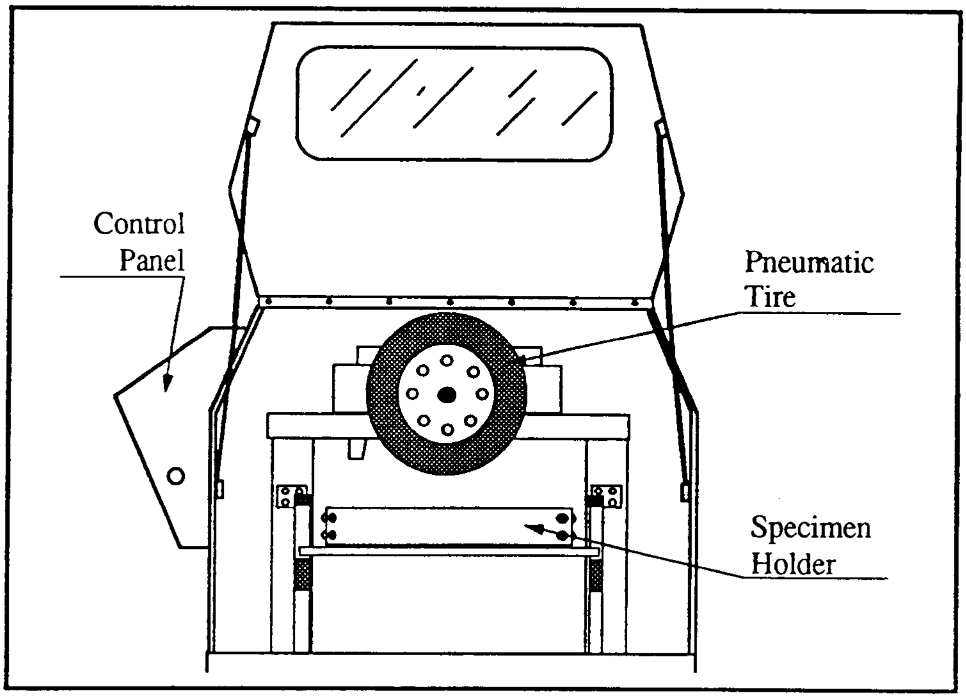
0.689 MPa (100 psi) was applied to the prismatic sample at 0.689 MPa (100 psi). The reciprocating wheel passes over the sample center (two samples tested simultaneously) twice per second, executing an alternating movement with an amplitude of 205 mm (8.07 in). Load time at the plate center was approximately 0.1 second, comparable with roadway loading conditions (Brousseaud et al, 1993).

At predetermined wheel pass intervals of 100, 200, 500, 1000, 2000, 5000, 10,000, 20,000, 30,000, 40,000, and 50,000, a rutting profile was obtained as an average of five positions across the sample surface at three locations along the length of the sample. By measuring the height of material pushed out of the wheel path, called shove, and the depth of the wheel path created, called rut, the addition of these two measurements gives an overall rut depth. Comparison of rut depth with cumulative wheel passes provides a relative assessment of rutting susceptibility. The test was terminated at 50,000 wheel passes or until the sample fails.

The OSU Wheel Tracker was not a SHRP developed method, it was used by SHRP to obtain a sense of expected mixture performance before the full scale field test results were completed. (Allen, 1993) Although the results were adequate for comparing relative mixture performance, it can not be used to predict actual rutting experienced in the field pavement. Currently, in Oregon or Washington, there is no correlation of mixtures tested between the laboratory and the field.



Schematic



Side View

Figure 2.7 OSU Wheel Tracker Schematic

2.3.4 Thermal Stress Restrained Specimen Test (TSRST)

The Thermal Stress Restrained Specimen Test (TSRST) provides a susceptibility evaluation of asphalt concrete mixtures to low temperature cracking. Specimens 5.72 cm dia x 25.4 cm long (2.25-in. dia x 10-in.) were cored from the roller compacted slabs and epoxied perpendicular to two end platens. This setup was hung from the top swivel jig (Figure 2.8) and restrained by a bottom swivel jig. Invar Rods were attached from the top end platen and hung downward where Linearly Variable Differential Transformers (LVDT) were attached to the bottom end platen, touching the invar rods. This allows the data acquisition equipment to adjust the servo motor and maintain the original specimen length.

Thermistors were attached to the specimen at the top, middle, and bottom to record temperature change as vaporized liquid nitrogen was introduced into the environmental chamber. The average temperature was used to control the amount of vaporized nitrogen entering the chamber to control the rate of cooling. A fan, located at the chamber bottom, was used to circulate the cooled air. As the sample cools, the specimen contracts, which was sensed by the LVDT's. The data acquisition equipment records this shrinkage and counteracts it by adjusting the servo motor to return the sample to its original length. The amount of load incurred on the sample was recorded by a load cell located at the bottom swivel jig .

As the temperature continues to decrease, the amount of load related to the temperature drop was recorded until the specimen fractured. The specimen thermal properties were defined by the temperature and induced stress at failure.

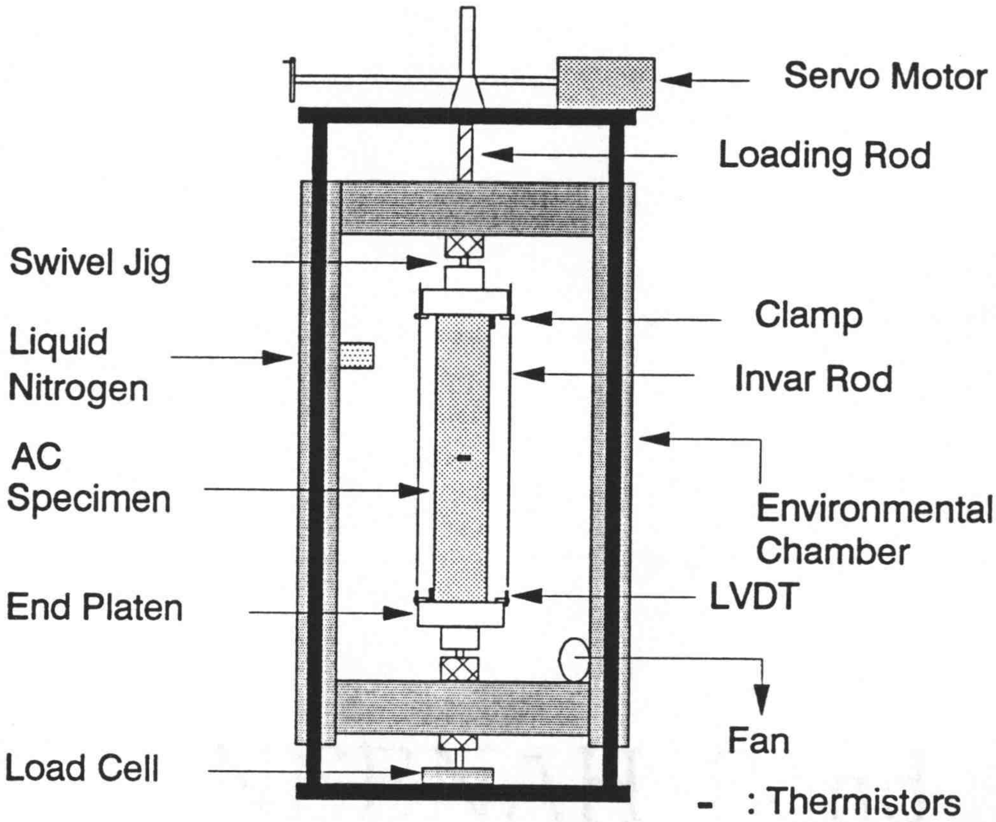


Figure 2.8 Schematic of Thermal Stress Restrained Specimen Test

2.3.5 Aging (Short and Long Term)

Rheological changes take place in the binder during hot mix asphalt (HMA) production (Roberts et al, 1991). During mixing, storage, and placement, binder penetration decreases while viscosity increases from the losses of volatile asphalt components and asphalt oxidation. Appendix B contains the standard test procedure for both short term and long term aging methods.

The Short-Term Oven Aging (STOA) procedure was developed for SHRP to simulate the stiffening of binders that normally occur due to mixing, field placement, and one year of service life (Bell et al, 1992). The method, consists of curing freshly mixed samples in a force draft oven at 135C (275°F) for four hours prior to compaction. Mixtures were stirred every hour to ensure uniform effects of aging throughout. Upon completion of this curing period, the mixture temperature was brought to compaction temperature and compacted using the roller method discussed earlier. This method of STOA was found to slightly increase the resilient modulus (M_R) values of the compacted specimens (Bell et al, 1992).

The Long-Term Oven Aging (LTOA) procedure was used to simulate field aging of asphalt concrete due to extended oxidation and continued loss of volatile asphalt components during the pavement's service life. LTOA was designed to simulate the total aging a compacted field mixture may experience in a 5 to 10 year service life (Bell and Sosnovske 1992). This procedure is explained in Appendix B. Specimens from each mixture were artificially aged at 85C (185°F) for five days in a force draft oven. After the five day period, the specimens were removed from the oven and allowed to cool to 25C (77°F). The diametral resilient modulus test (ASTM D-4123) was used to characterize the stiffness of asphalt mixtures. A cylindrical specimen was repeatedly loaded across the diameter, while deformations caused by this loading were recorded. A ratio of applied load to the recoverable strain was calculated as the resilient modulus (M_R). A comparison of initial and final modulus values ($M_{R\ final}$ $M_{R\ initial}$) were used to determine the relative effects of oven aging on AC mixtures.

2.3.6 Environmental Conditioning System (ECS)

Many asphalt-aggregate combinations are susceptible to water damage, particularly stripping (Terrel et al, 1992). The Environmental Conditioning System (ECS) was developed at OSU to evaluate mixtures and their water sensitivity (Allen, 1993).

Cores 10.2 cm dia x 10.2 cm high (4-in dia x 4-in high) were taken from the roller compacted slabs produced in the laboratory. These samples were encased in a latex rubber membrane so water may flow from the specimen bottom to the top during conditioning. After air permeability was measured, the ECS resilient modulus (ECS M_R) was determined at 25C (77°F), followed by measurement of water permeability. ECS M_R differs from Diametral M_R in that the specimen was axially loaded and the specimen was sealed within a membrane. The sample was then conditioned with water and heated to 60C (140°F) for at least 6 hours (hot cycle). During this hot cycle, a repetitive load of 1.37 MPa (200 lbs) was applied for 0.1 second on and 0.9

second off. After six hours, the sample was cooled to 25C (77°F) and another ECS M_R was determined and water permeability measured. The hot cycle and subsequent measurements were repeated two more times. For mixture evaluation in areas susceptible to freezing, the last temperature cycle was dropped to -18C (0°F) for 6 hours (freeze cycle). There was no repetitive loading during the freeze cycle. Again, the sample was returned to 25C (77°F) for an ECS M_R and measurement of water permeability. An ECS modulus ratio was calculated by dividing each cycle's ECS M_R by the preconditioned ECS M_R. From this, an evaluation of a mixture's water sensitivity was made. Following testing, the amount of visual stripping was evaluated as a percentage over the sample's cross section using a standardized chart. Figure 2.9 shows the ECS system while Figure 2.10 shows the load frame schematic with the specimen in place.

2.4 Analysis of Results

Two methods of statistical analysis were used to compare the CRM mixture characteristics to the Class 'A' surface mixture. Multiple range analysis was used to show the mean values obtained from an experiment and to give a relative mixture ranking.

Several analysis tools exist for comparing the data group means. The Least Significant Difference (LSD) method and Duncan's Multiple Range Analysis (Duncan) will be used for this comparison (Montgomery, 1991).

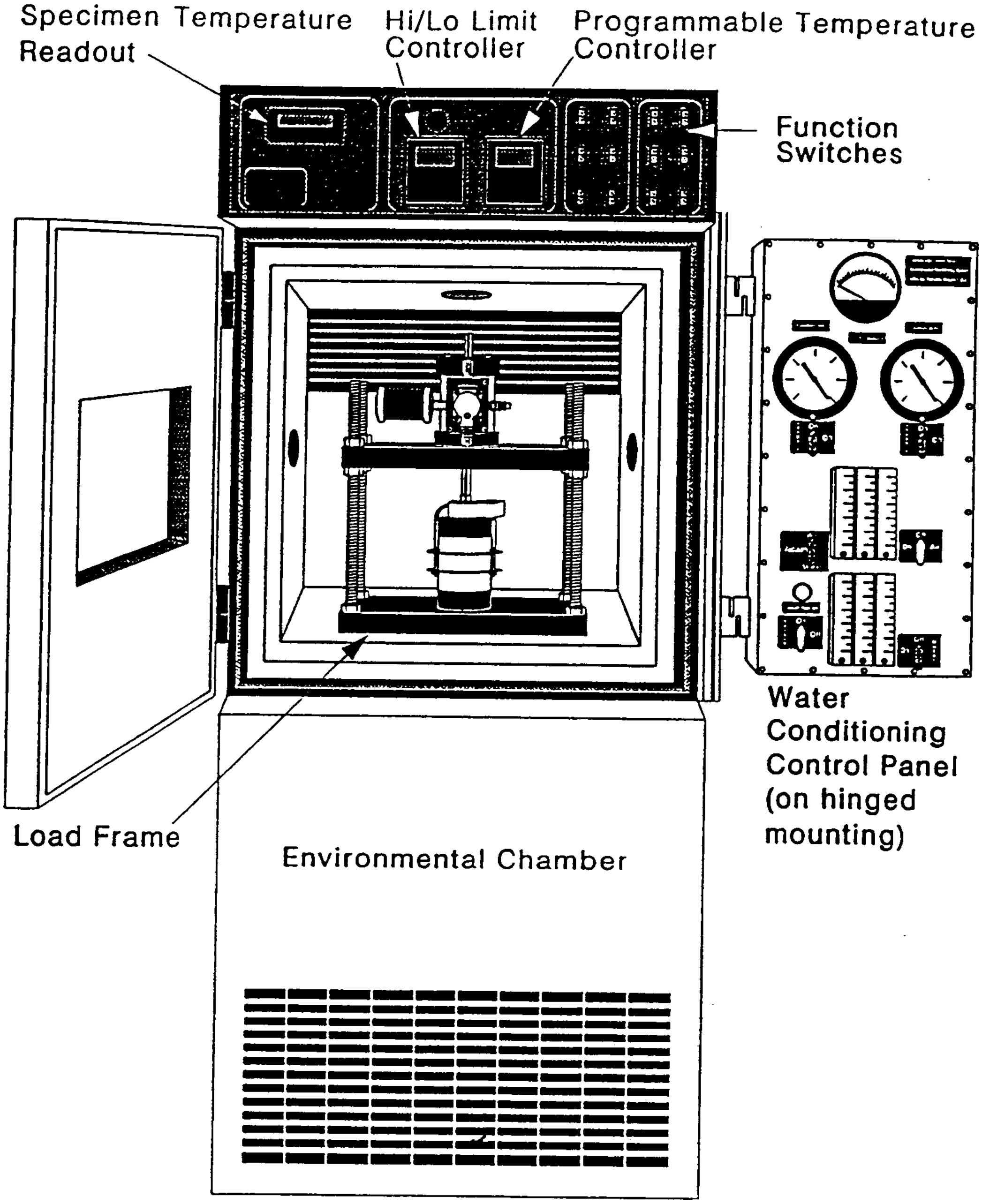


Figure 2.9 Environmental Conditioning System (front view)

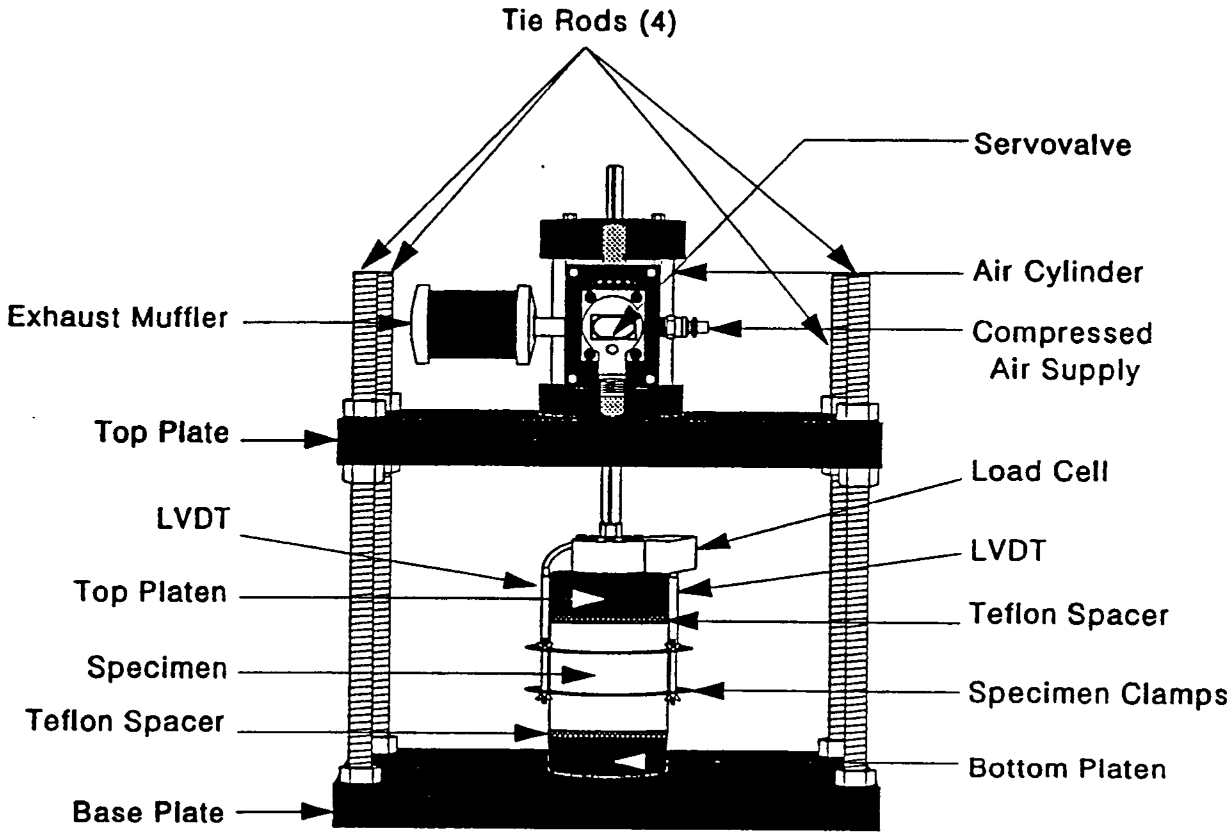


Figure 2.10 Schematic of Load Frame with Specimen

The LSD method is used in situations where the same degrees of freedom exist between each mixture group. The method simply compares the observed difference between each pair of averages to the corresponding LSD (Montgomery, 1991).

Duncan's Multiple Range Test is widely used for comparing pairs of means where there are unequal sample sizes between groups. Duncan is popular because the analysis is sensitive enough to detect mean differences where LSD can not (Montgomery, 1991). Applications of these procedures are complicated to perform by hand. However, a statistical software package was used to perform the LSD and Duncan analysis (Manguistics, 1992).

To present the multiple range analysis, with either technique, a table similar to that shown in Table 2.4 would be used.

Table 2.4 Example of Multiple Range Analysis Results

Mixture	Number of Observations	LS Mean	Homogeneous Groups
Control	4	1.5	
CRM_1	4	2.0	
CRM_2	5	3.0	

The shaded bar spanning the Control and CRM_1 groups show that the means (LS Mean column) for four samples in the Control and CRM_1 groups were statistically similar at the 95% confidence interval ($\alpha = 0.05$). Similarly, CRM_1 and CRM_2 means were statically similar, even with varying sample sizes between groups. However, means for the Control and CRM_2 groups were different, and not considered to be from the same sampling population. Means for CRM_2 and CRM_1 were shown to overlap. This indicates that no significant difference between these groups exist ($\alpha = 0.05$) and may be considered to be from the same sampling population. This was only an example, but sets the stage for the analysis to follow.

3.0 RESULTS

Once the specimens were prepared and tested with the procedures outlined in section two, the results of each test were graphed and compared to the other mixtures. The results of each test are presented and discussed herein.

The investigated mixtures used different quantities of material, as specified. Figure 3.1 shows a graphical representation of the components included in each mixture. The components are shown as a percentage of total mixture weight. The graph also shows the percentage of CRM contained in each mixture. The PlusRide II® mixtures contain the most CRM at 3 percent by total mixture weight, whereas the ARHM-GG material contains only 1.2 percent of CRM. The CRM mixtures have higher asphalt contents when compared to the Class 'A' surface mixture. PlusRide II® mixtures have lower air void contents than either the Class 'A' or ARHM-GG surface mixtures, as noted in Table 2.3.

3.1 Fatigue Cracking

Each mixture was evaluated using 6 specimens, 3 each at 2 different strain levels. However, for the ARHM-GG surface mixture the results for only 4 specimens were reported. This was due to one specimen failing prematurely and one specimen never reaching failure. Because of project timing, only the PlusRide II® surface and base mixtures using AR 4000W were tested, along with the ARHM-GG and Class 'A' surface mixtures.

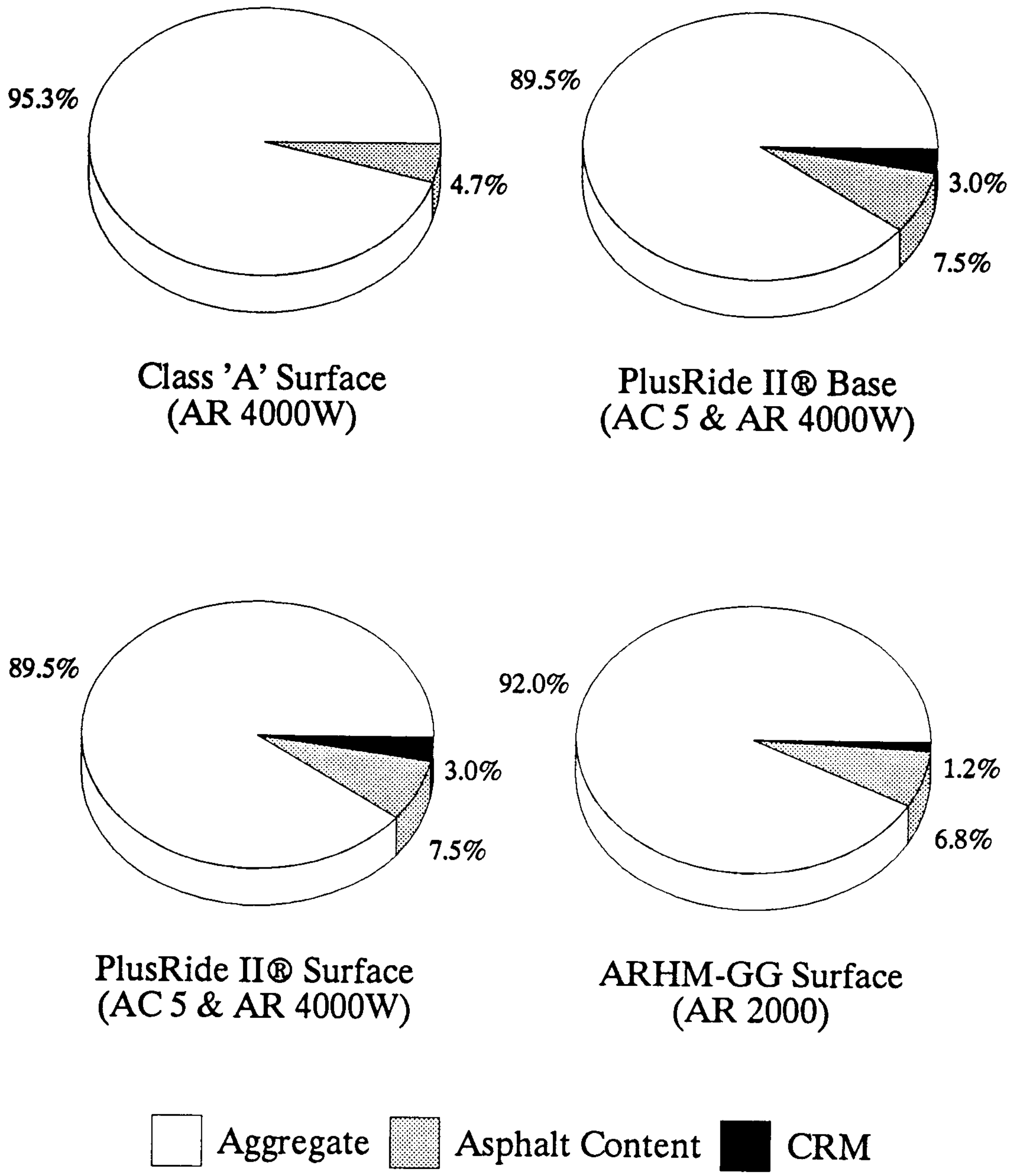


Figure 3.1 Specified Mixture Components (Percentage by Total Mixture Weight)

Figure 3.2 shows the initial dynamic modulus of each sample. Samples A though F for the Class 'A' surface mixture were shown to be very stiff. The dynamic modulus of the CRM modified mixtures were relatively low. The PlusRide II® surface and base mixtures have relatively the same initial dynamic modulus. The dynamic modulus of the ARHM-GG mixtures were slightly lower, but overall, the results were approximately the same as the PlusRide II® mixtures.

The number of cycles to failure were recorded in addition to the strain (held at a constant). The results are shown in Figure 3.3. Each data point represents a single sample. Regression lines are shown for each mixture. The slope of the regression line may be interpreted as the rate of cycles per strain. As in Figure 3.2, there was a discernable difference between the Class 'A' surface mixture and the CRM mixtures. The Class 'A' surface mixture showed lower cycles to failure at a given strain level than the CRM mixtures. However, there was little difference among the CRM mixtures.

It should be noted that these regression lines have been extrapolated beyond their original points. This may make proper regression interpretation unreliable beyond the relevant range. However, an easy visual comparison may be made between the mixtures.

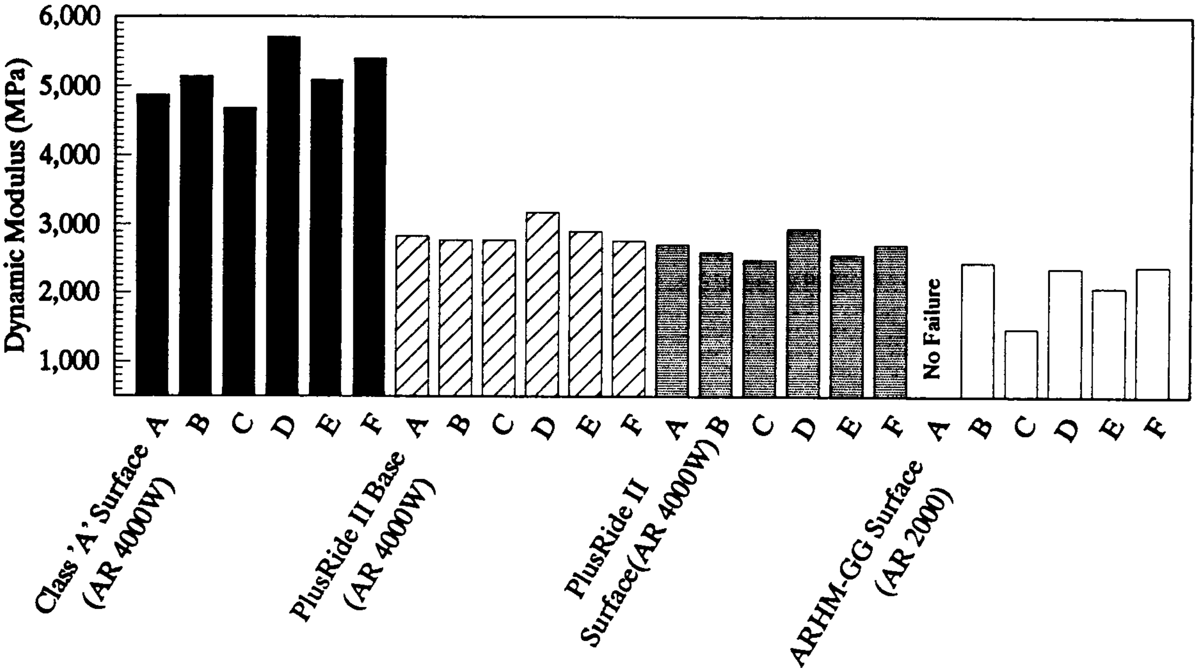


Figure 3.2 Initial Dynamic Modulus Results

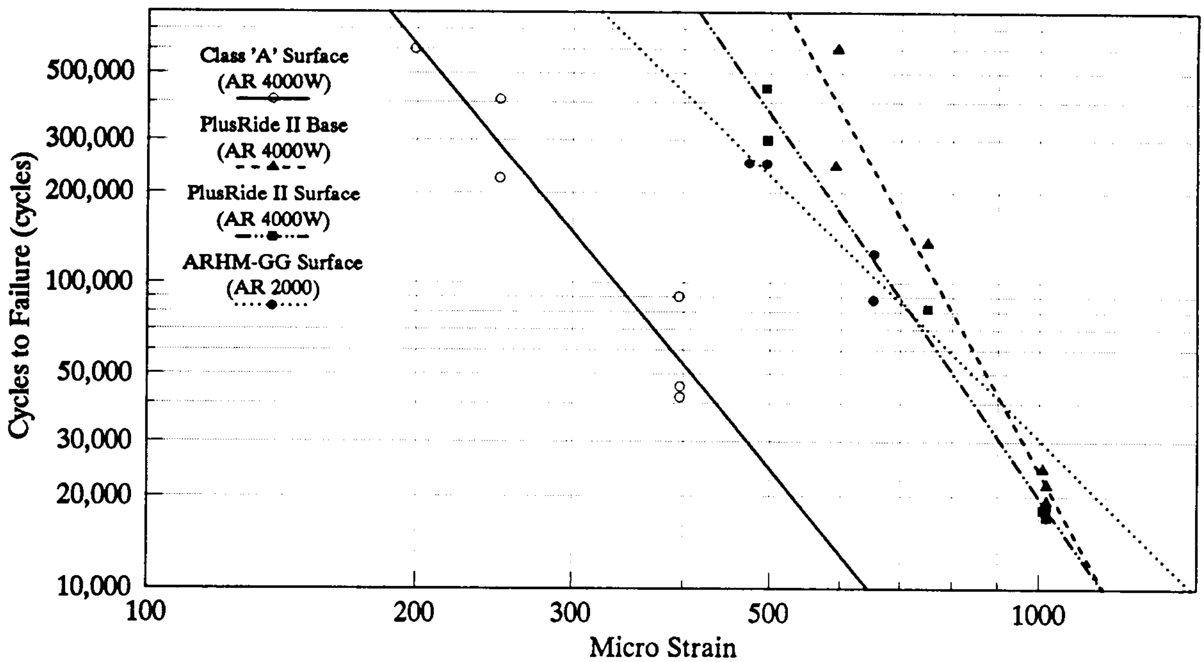


Figure 3.3 Flexural Beam Fatigue Test Results for All Mixtures

3.2 Rutting Susceptibility

3.2.1 Repetitive Shear Strain Test--Constant Height (RSST-CH)

The RSST-CH was conducted at two temperatures. All but two samples were tested at 50C (122°F). One sample each from the Class 'A' and PlusRide II® surface mixtures were tested at 40C (104°F) to compare the mixture performance. Results of the 40C (104°F) tests were not discussed since only limited data was available.

Again, only the PlusRide II® (AR 4000) surface and base mixtures were evaluated. From Figures 3.4 through 3.7 it can be seen that there was good repeatability among the mixtures. Failure was defined as 5% permanent shear strain. Therefore, the number of shearing repetitions to 5% strain was used for comparison between mixtures.

Figures 3.4 through 3.7 show the rate of permanent shear strain to the number of repetitions to failure. Relative comparison of the PlusRide II® base (Figure 3.5) and surface (Figure 3.6) mixtures seem to show similar performance. Both mixtures ultimately failed around 200 repetitions. The ARHM-GG surface mixture (Figure 3.7) out performed all the mixtures with failure at 20,000 repetitions. This was two orders of magnitude greater than the PlusRide II® base and surface mixtures and one order of magnitude greater than the Class 'A' surface mixture.

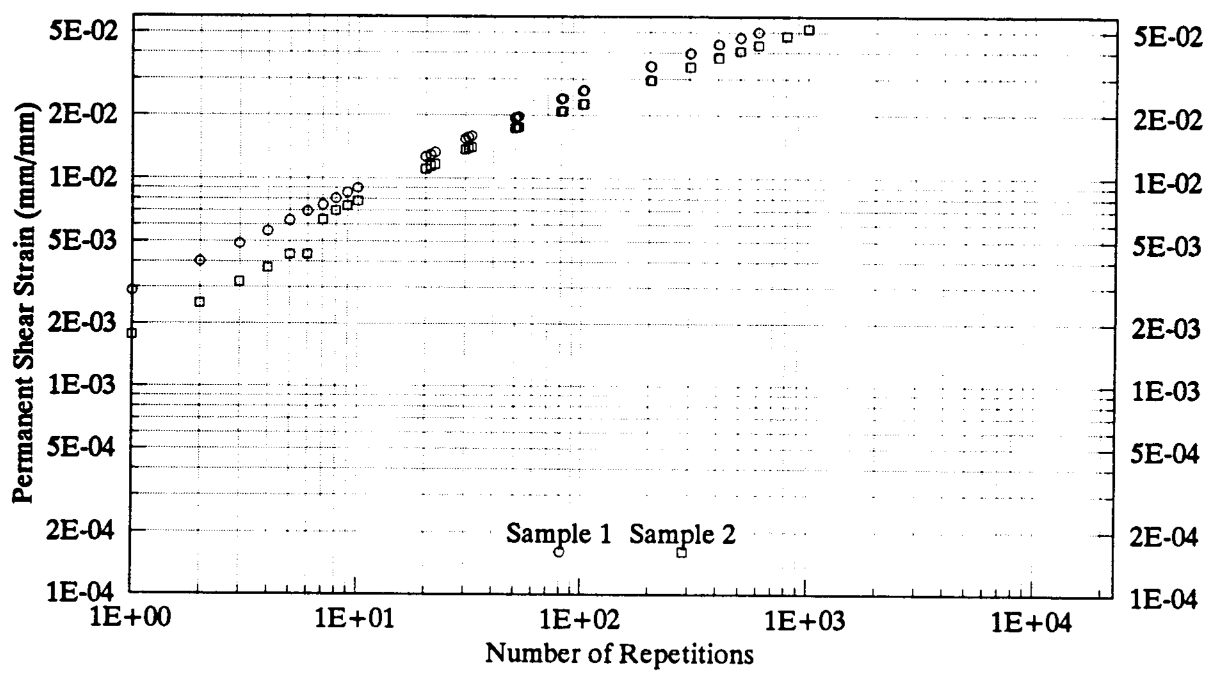


Figure 3.4 Repetitive Shear Strain Test Results for Class 'A' Surface (AR 4000W) Mixture

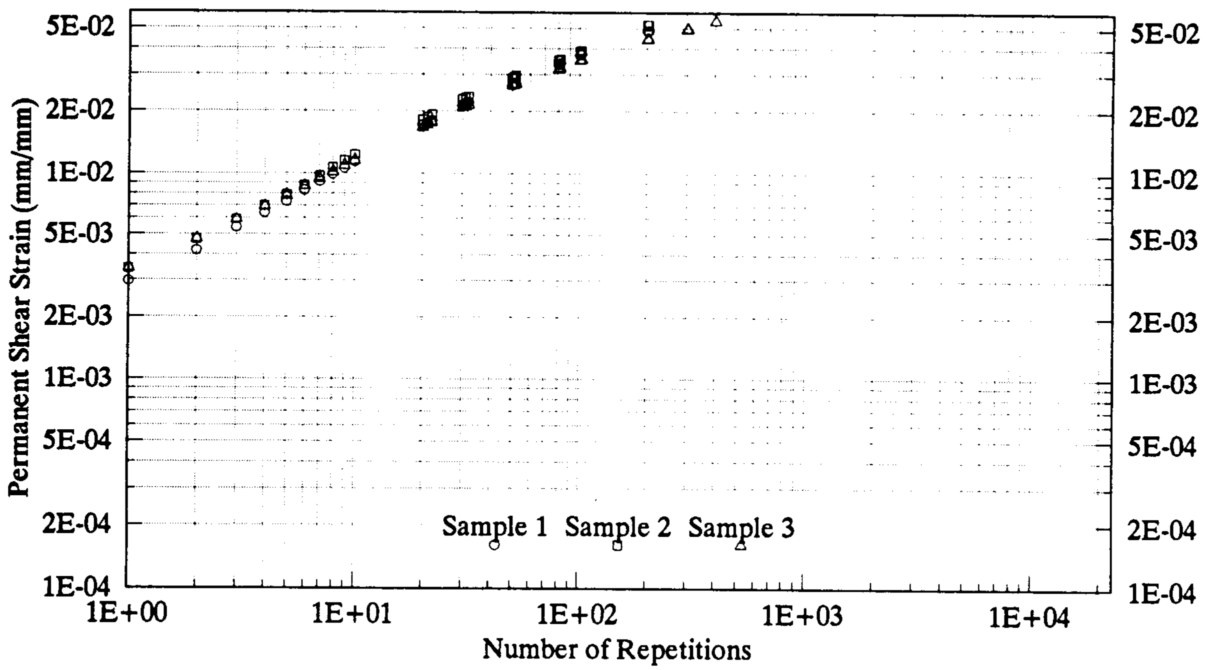


Figure 3.5 Repetitive Shear Strain Test Results for PlusRide II® Base (AR 4000W) Mixture

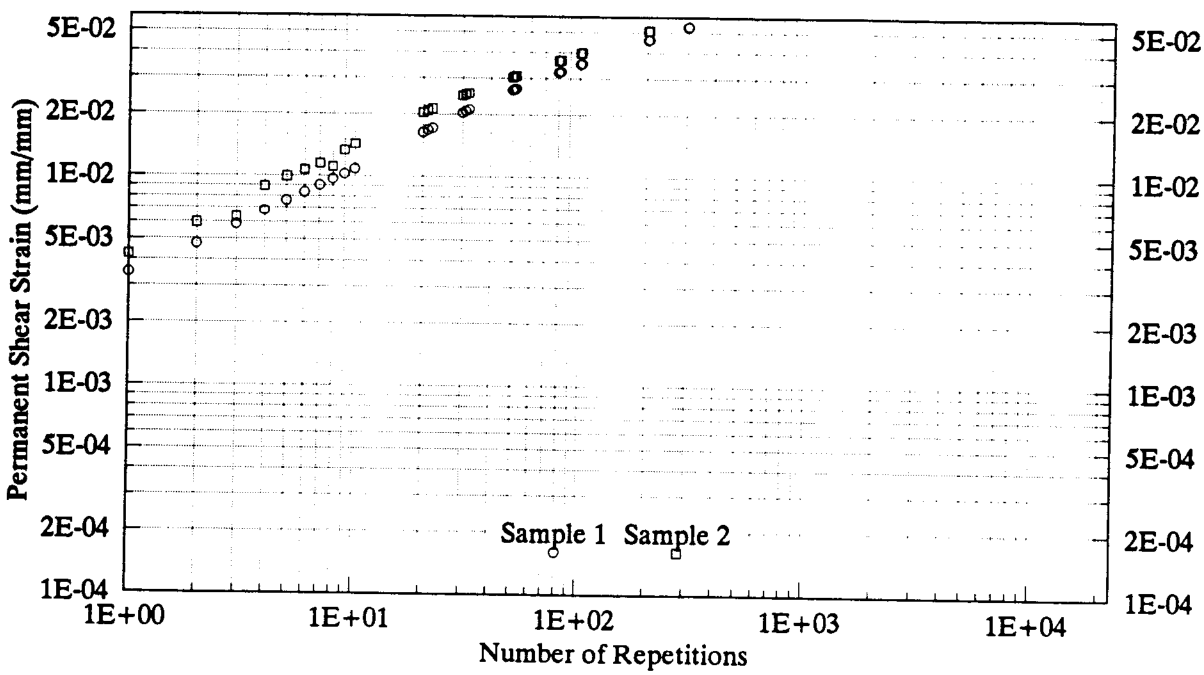


Figure 3.6 Repetitive Shear Strain Test Results for PlusRide II® Surface (AR 4000W) Mixture

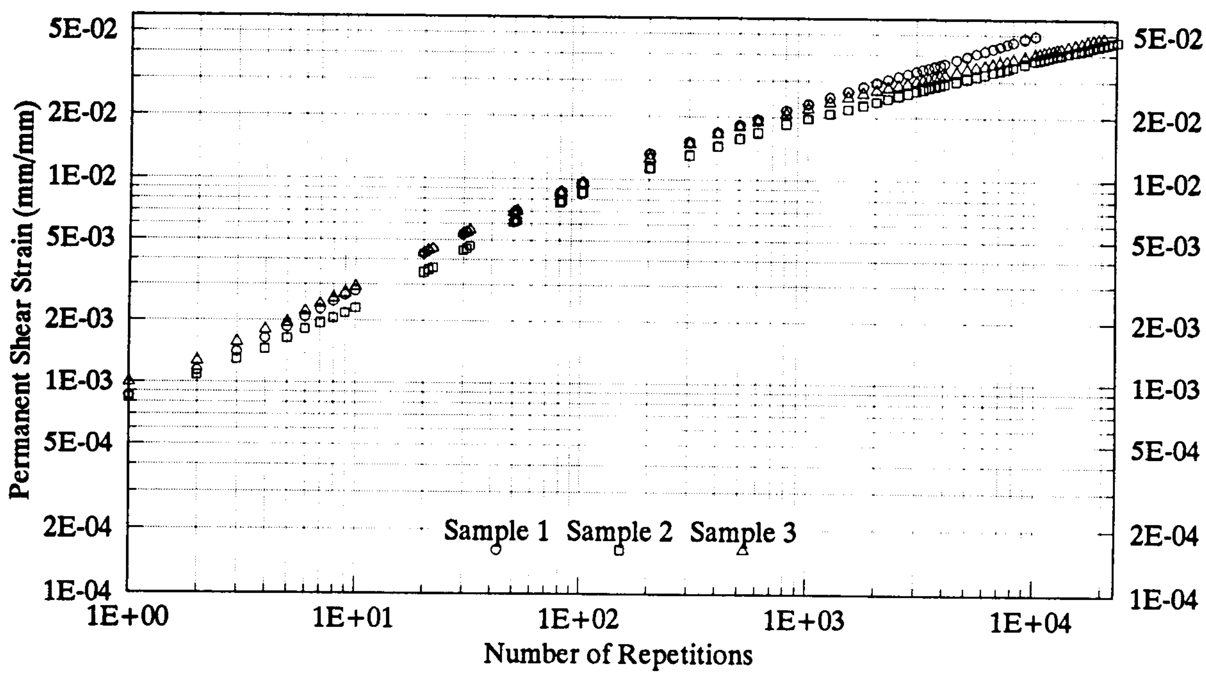


Figure 3.7 Repetitive Shear Strain Test Results for ARHM-GG Surface (AR 2000) Mixture

3.2.2 OSU Wheel Tracker

All mixtures (AC 5 and AR 4000W binder types) were tested in the OSU Wheel Tracker. Graphical results for the OSU wheel tracker are presented in Figures 3.8 through 3.11 It should be noted that the PlusRide II® base and surface mixtures (both AR 4000W and AC 5 binder types) could not be tested to the full 50,000 wheel passes. Tertiary failure of the mixture was not defined by a predetermined rutting depth, but from the limitations of the measuring equipment. Heave from the sample sides, around the tire path, was so great that the measuring device could not be accurately placed for measurement without disturbing the sample.

The Class 'A' and ARHM-GG surface mixtures performed equally well. The degree of rutting was comparable between the two mixtures. Both mixtures were tested to 50,000 wheel passes with around 3.81 mm (0.15 inches) of total rutting. From the figures, it can be seen that these mixtures maintained the stable shear zone. The PlusRide II® base mixtures did not fare as well. The PlusRide II® mixtures did not reach the 50,000 wheel pass interval. As seen in Figure 3.9 the PlusRide II® base mixtures failed around a rut depth of 12.7 mm (0.5 in). The rate of rutting was slightly higher for the AC 5 than for the AR 4000W mixtures. It should be noted for sample number 2 (AR 4000W) that either a decrease in average rut depth at 2,000 wheel passes or an increase in rut depth at 1,000 wheel passes occurred. This is more likely to be an erroneous data point, in either case.

Of all the tests performed in this CRM investigation, rutting resistance with the OSU Wheel Tracker was the most visual to interpret. After each sample was rutted, a cut was made through the middle, perpendicular to the direction of wheel tracking. Figure 3.12 shows the actual cross sectional views of each rutted mixture. A reference scale is observed at the top of each sample. Each division on the vertical member represents 0.25 cm (0.1-in.). A visual evaluation of Figure 3.12 shows the dramatic results of each mixtures rutting susceptibility. The Class 'A' surface (a) and ARHM-GG surface (b) mixtures performed equally well, with very little rutting. The PlusRide II® (AR 4000W) base (e) and surface (f) mixtures performed slightly better than the PlusRide II® (AC 5) base (c) and surface (d) mixtures. Notice that the PlusRide II® mixtures have severe rutting and have entered the tertiary stage of permanent deformation.

3.3 Thermal Cracking

The Thermal Stress Restrained Specimen Test (TSRST) results were collected on all mixture types (AC 5 and AR 4000W binder types). Figure 3.13 shows the Class 'A' surface mixture results. The ultimate stress and fracture temperature results were fairly consistent, as with all the TSRST results. Figures 3.13 through 3.19 show TSRST results for each mixture. Tables 4.5 through 4.7 summarizes fracture stress, temperature, and rate of thermal stress with temperature, respectively.

The PlusRide II® base mixture with the low volume of voids (Figure 3.14) gives similar results to the Class 'A' surface mixture; however, a premature fracture of sample number 3 occurred during testing. Therefore, only data for two samples was

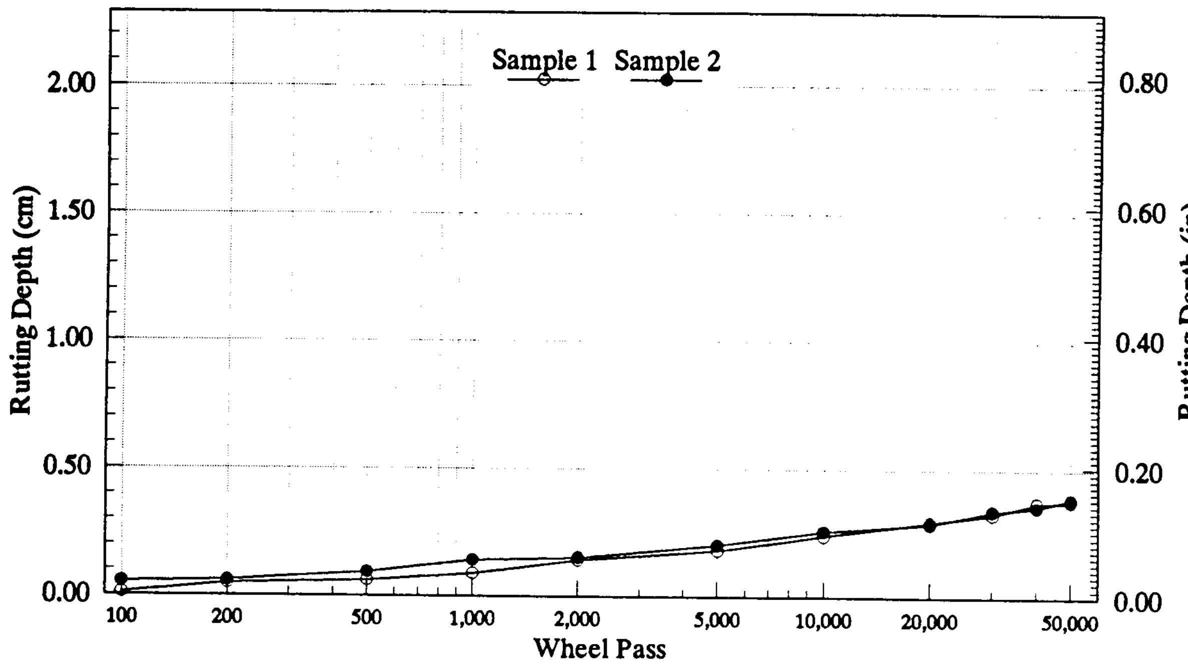


Figure 3.8 OSU Wheel Tracker Test Results for Class 'A' Surface Mixture

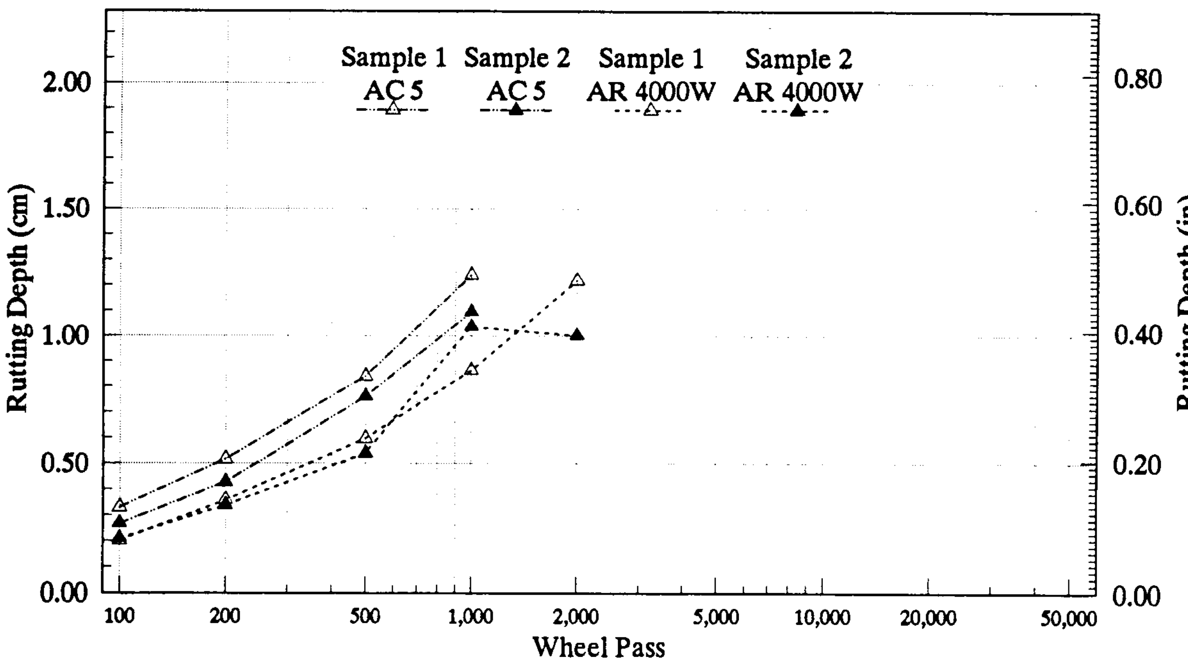


Figure 3.9 OSU Wheel Tracker Test Results for PlusRide II® Base Mixtures

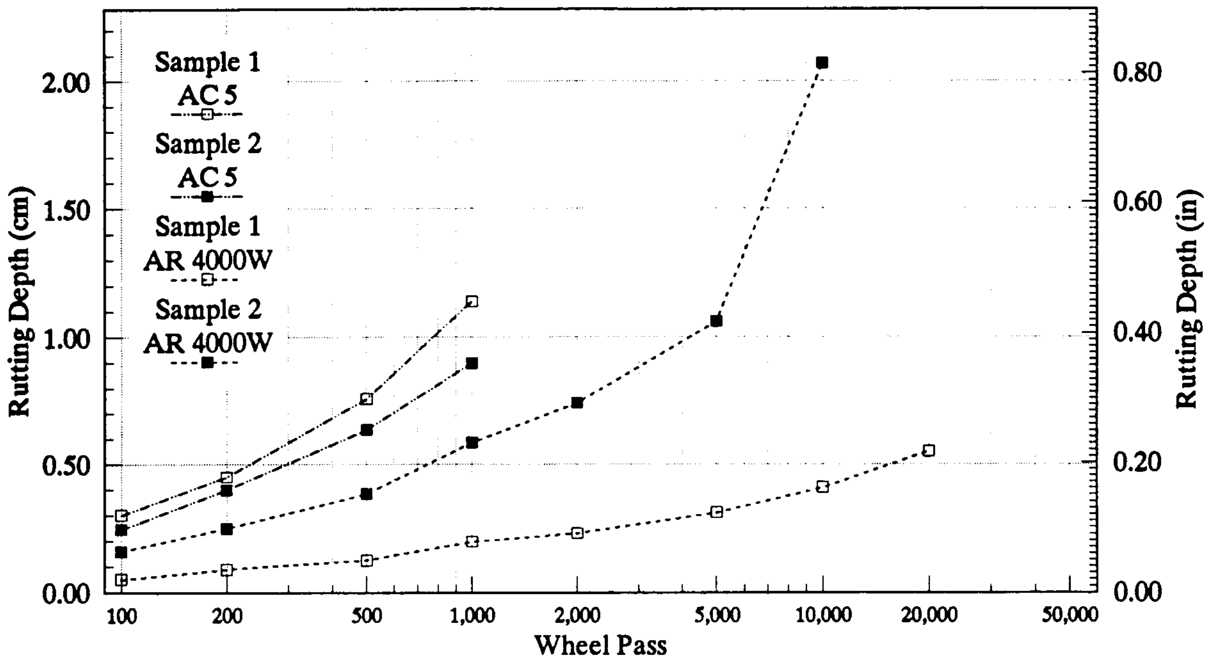


Figure 3.10 OSU Wheel Tracker Test Results for PlusRide II® Surface Mixtures

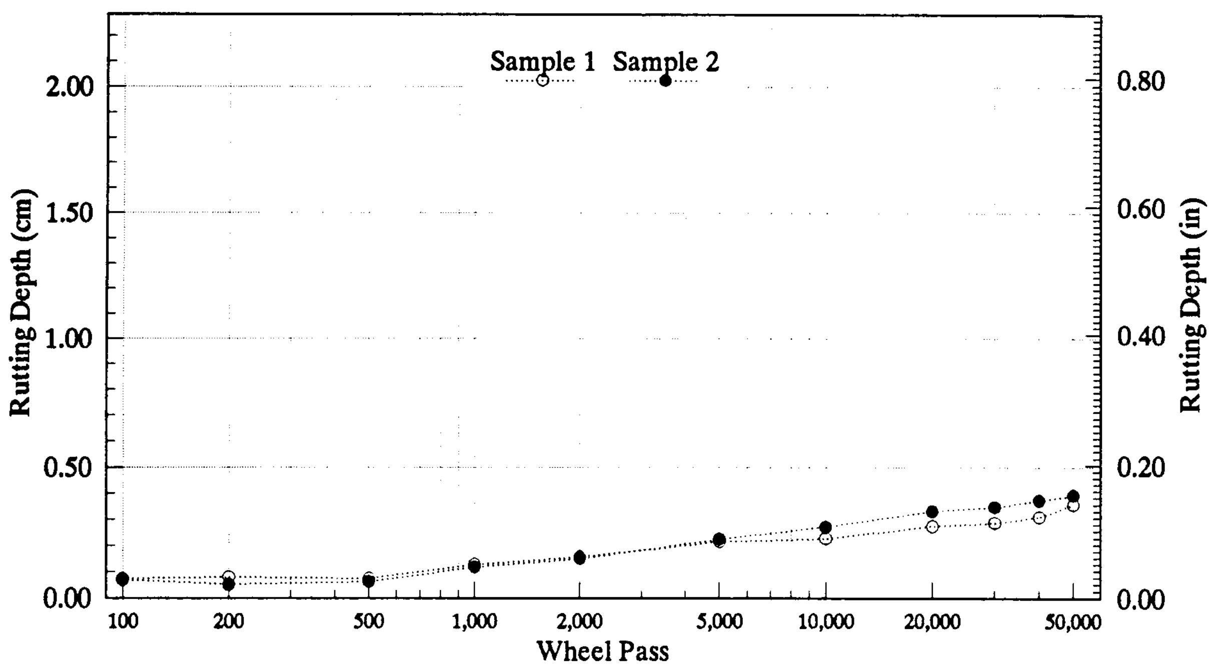
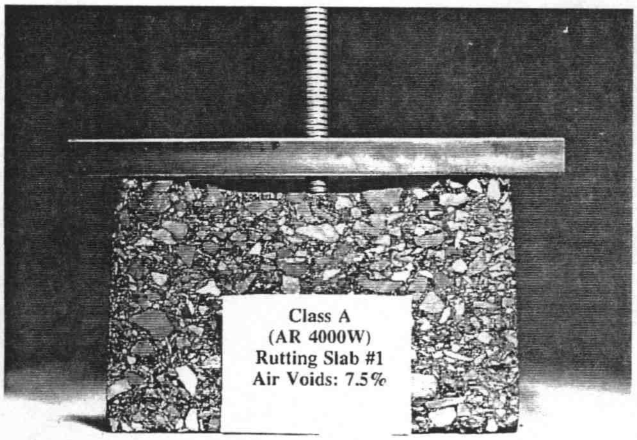
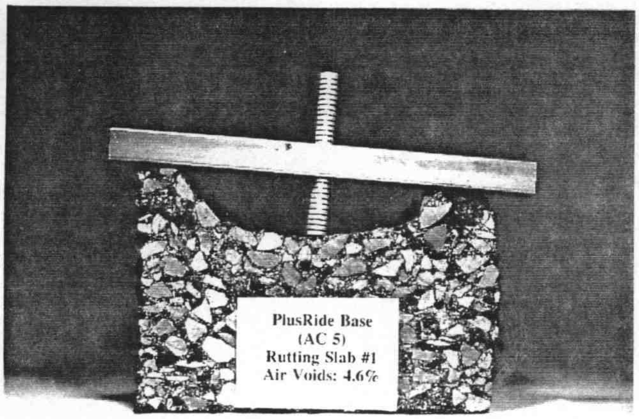


Figure 3.11 OSU Wheel Tracker Test Results for ARHM-GG Surface Mixture

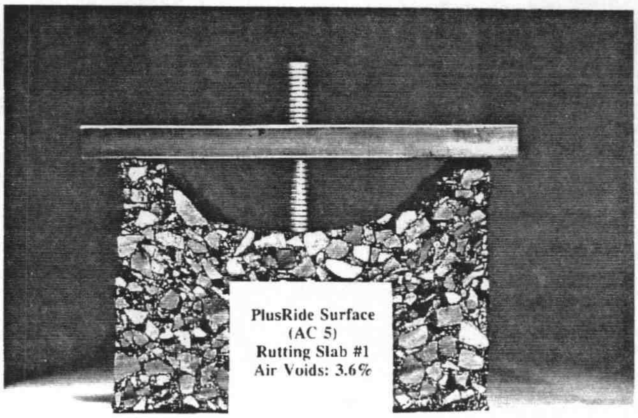


(a) Class 'A' Surface Mixture
(AR 4000W)

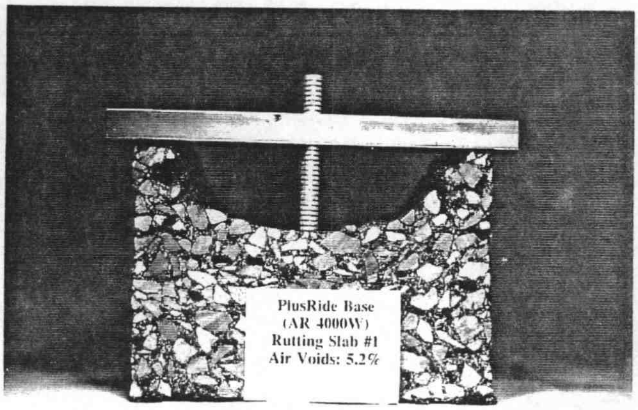
(b) ARHM-GG Surface Mixture
(AR 2000)



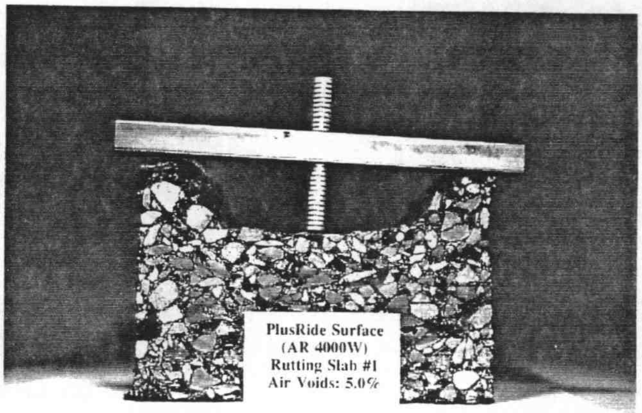
(c) PlusRide II® Base Mixture
(AC 5)



(d) PlusRide II® Surface Mixture
(AC 5)



(e) PlusRide II® Base Mixture
(AR 4000W)



(f) PlusRide II® Surface Mixture
(AR 4000W)

Figure 3.12 Visual Evaluation of Rutting Susceptibility

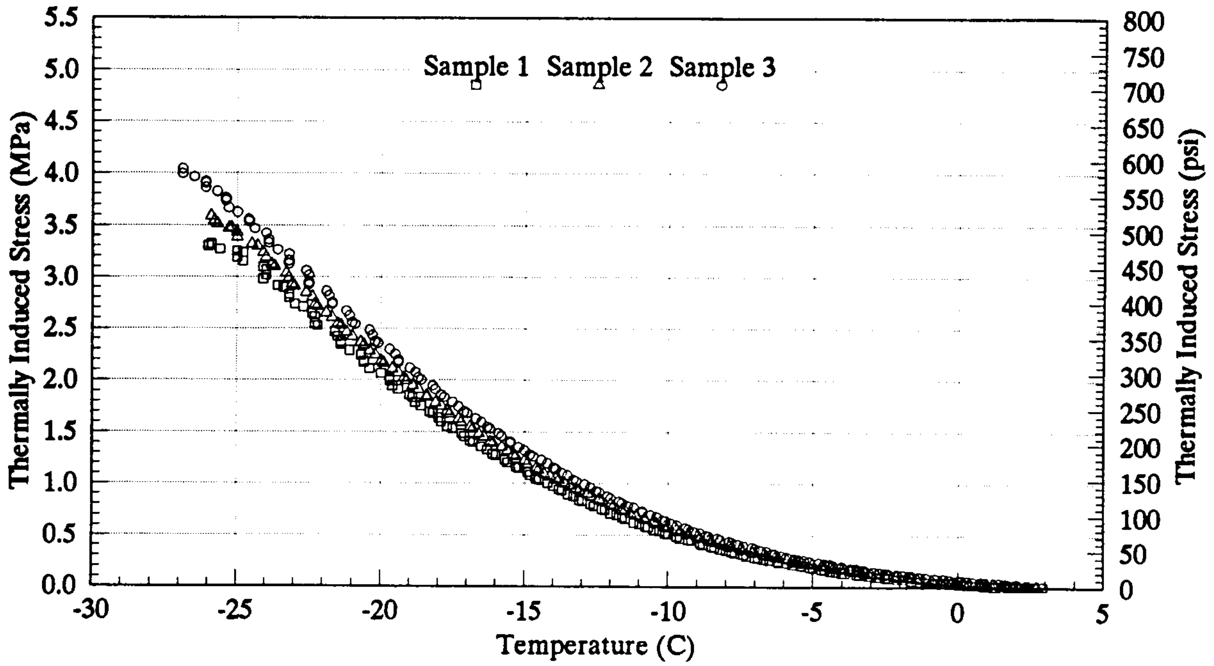


Figure 3.13 Thermally Induced Stress Curves for Class 'A' Surface (AR 4000W) Mixture

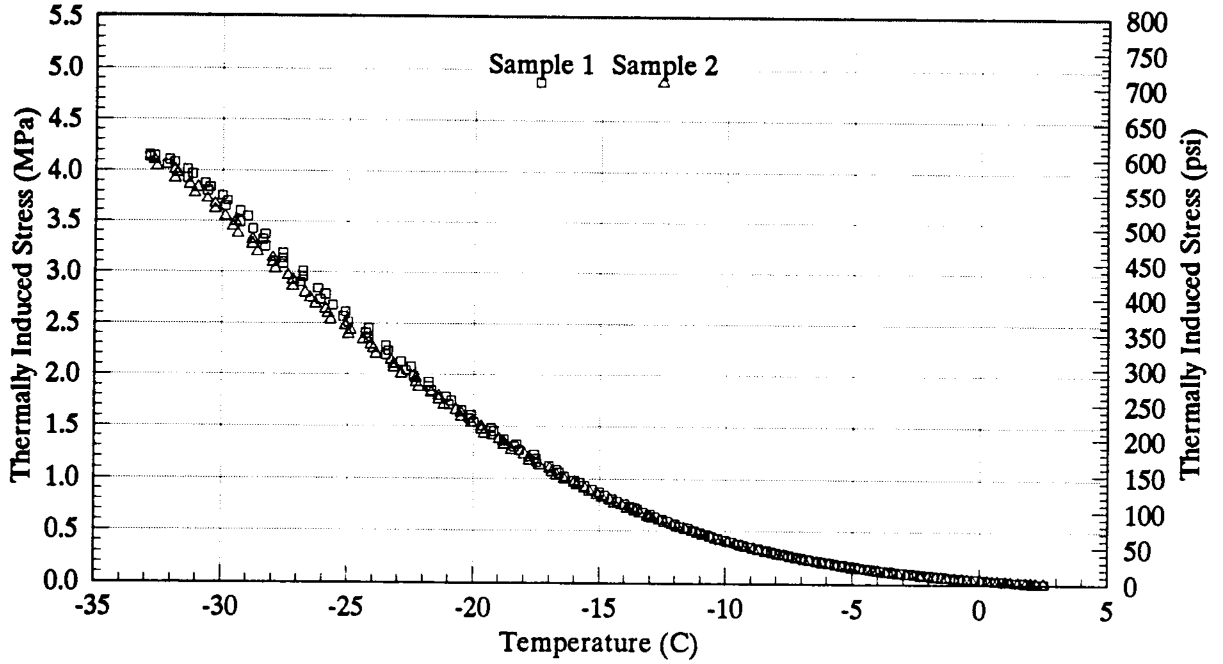


Figure 3.14 Thermally Induced Stress Curves for PlusRide II® Base (AC 5) Mixture with Low Air Voids

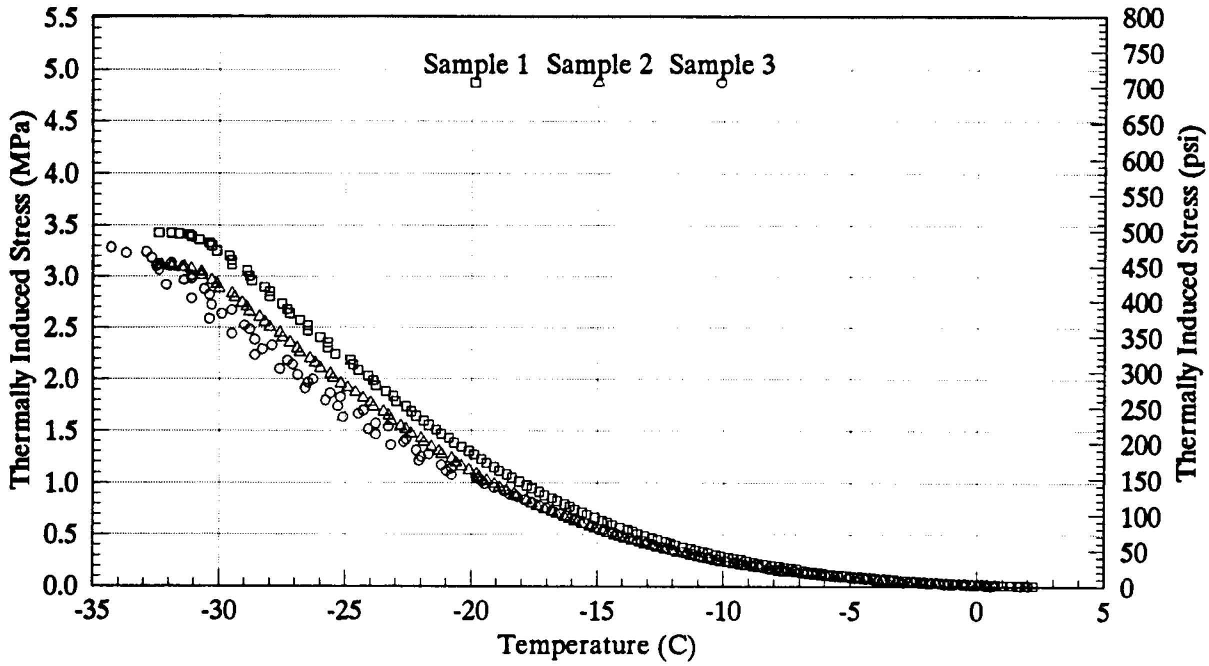


Figure 3.15 Thermally Induced Stress Curves for PlusRide II® Base (AC 5) Mixture

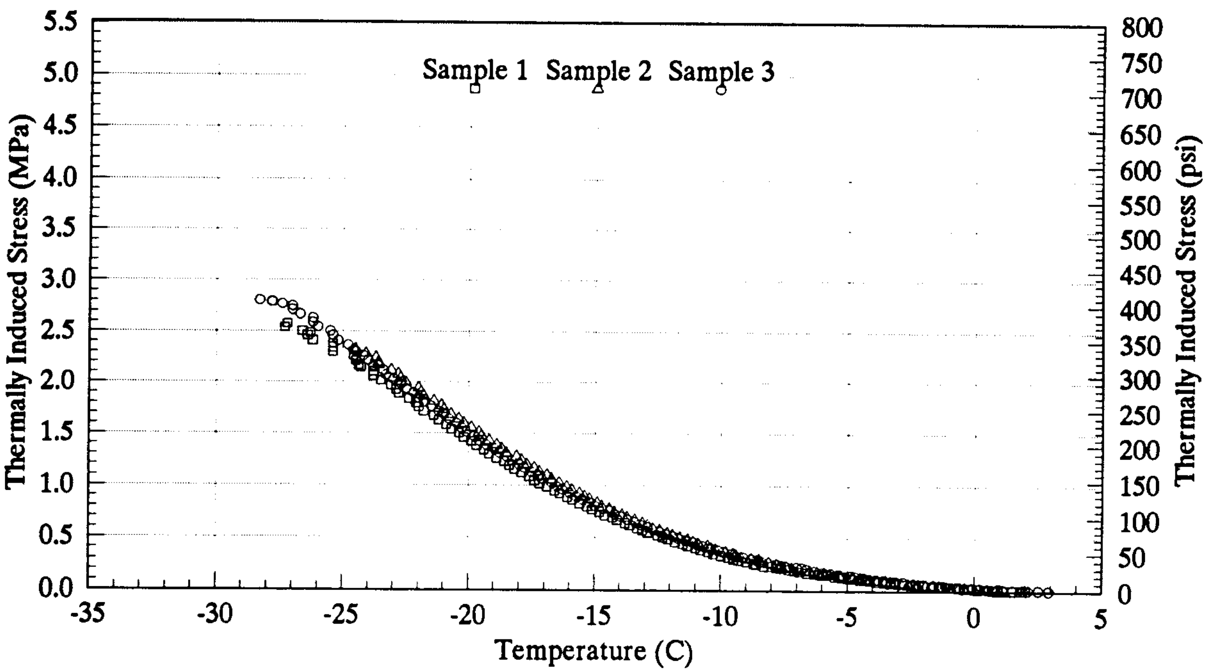


Figure 3.16 Thermally Induced Stress Curves for PlusRide II® Base (AR 4000W) Mixture

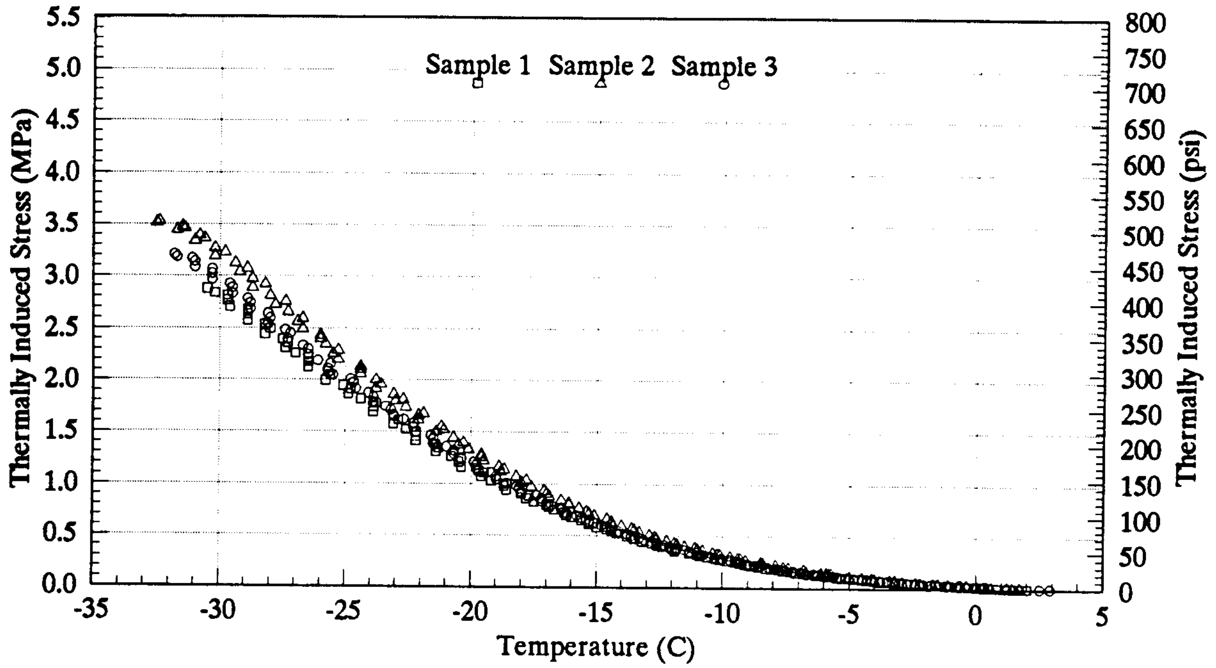


Figure 3.17 Thermally Induced Stress Curves for PlusRide II® Surface (AC 5) Mixture

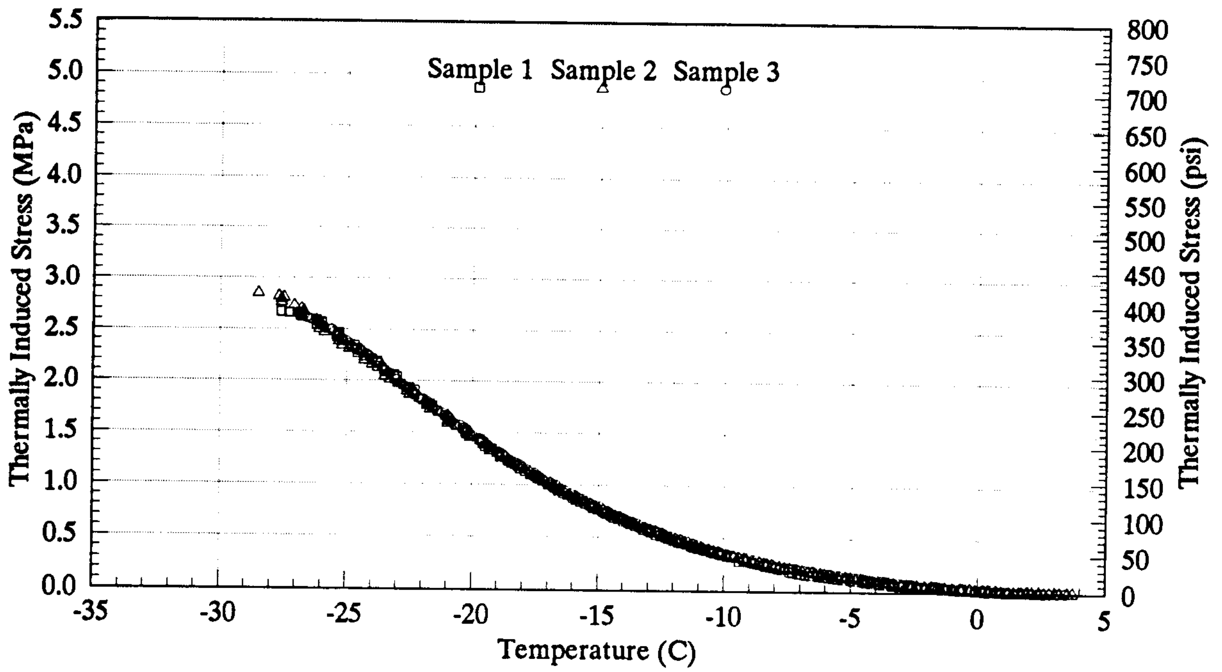


Figure 3.18 Thermally Induced Stress Curves for PlusRide II® Surface (AR 4000W) Mixture

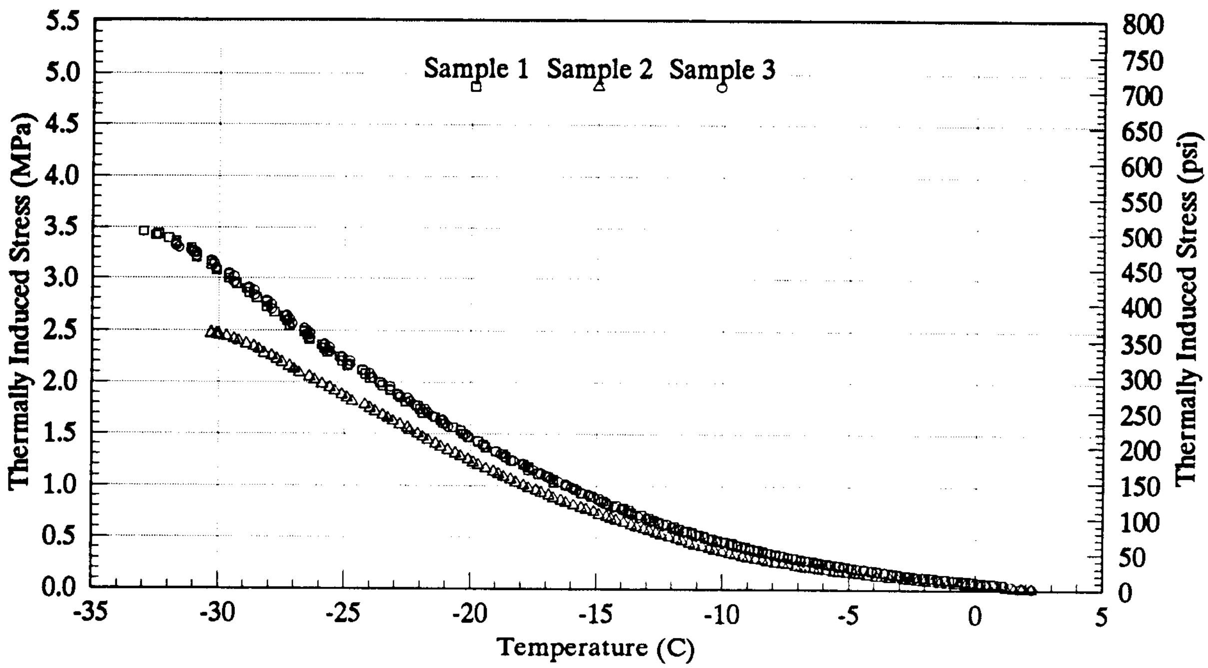


Figure 3.19 Thermally Induced Stress Curves for ARHM-GG Surface (AR 2000) Mixture

shown. It was noted that this fracture was due to an unusually high amount of PlusRide II® CRM material in one area of the sample.

3.4 Aging (Long Term)

Figure 3.20 displays the results of long term oven aging. All mixture types were tested. Since all samples were short term oven aged (STOA), initial M_R values are shown as STOA results. Three samples from each mixture type were tested with the LTOA procedure outlined in Appendix B. These LTOA results were plotted against the initial STOA results. The horizontal axis in Figure 3.20 represents the original STOA M_R of each sample. The vertical axis represents the M_R of each sample after LTOA. The diagonal line represents no change in M_R with respect to STOA. All the data points lie above the diagonal line, showing the degree of LTOA. It was interesting to note that all the CRM mixtures aged at the same rate. Conversely, the Class 'A' surface mixture aged significantly, as shown by its relative position in the figure. The CRM mixtures resisted LTOA better than the Class 'A' surface mixture.

3.5 Water Sensitivity

All mixtures were tested for water sensitivity. However, due to the low M_R of the PlusRide II® mixtures, no repetitive loading was applied during any temperature cycle. A preliminary investigation in the ECS showed excessive deformations with the

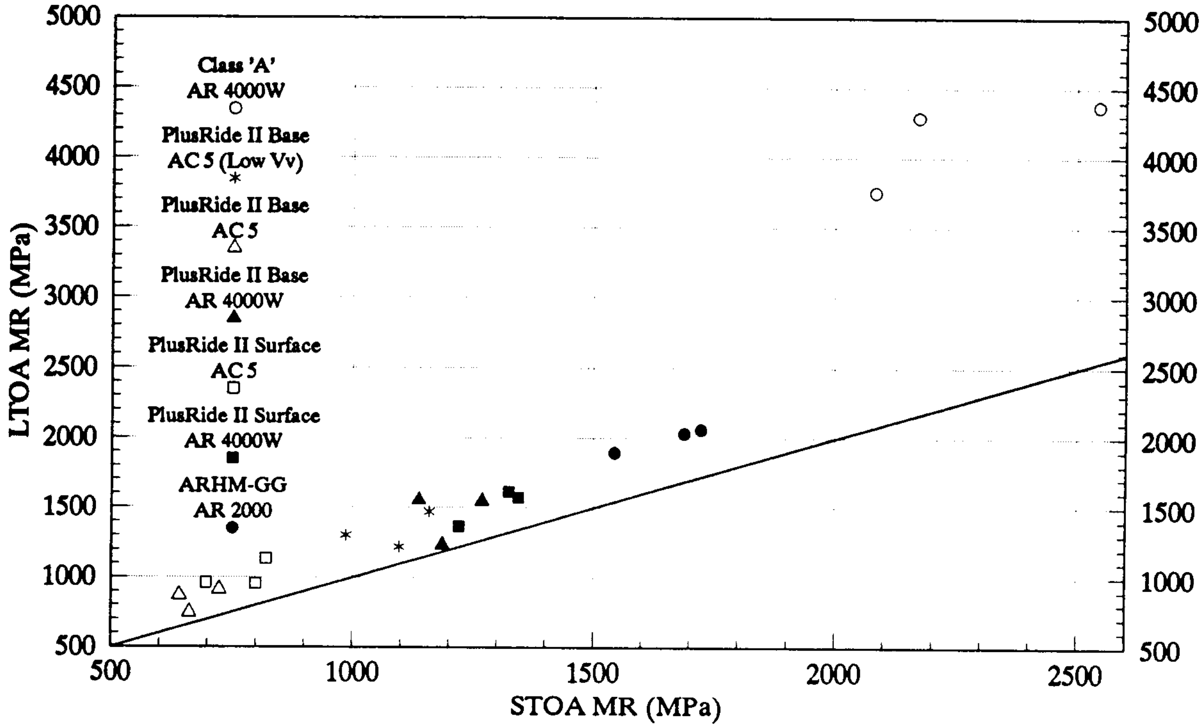


Figure 3.20 LTOA vs. STOA M_R Test Results

PlusRide II® mixtures after only one hot cycle, with repeated loading. The other two mixture types were tested according to the ECS protocol. Figures 3.21 through 3.27 show the ECS modulus ratio results from each mixture with respect to conditioning cycle. Three samples were tested from each mixture. After ECS testing, samples were split and a visual evaluation was made using the standard stripping and binder migration evaluations (Allen, 1993). The potential loss of adhesion between the aggregate and asphalt was evaluated as a percentage of the entire sample cross section. However, virtually all PlusRide II® mixtures were impermeable to water. When the samples were split, the interior was dry.

Only the Class 'A' surface mixture could be measured for air permeability. All other mixtures, initially, were either permeable below the range of the test equipment or had no air permeability. The Class 'A' samples were permeable to water and increased in permeability with each cycle. The ARHM-GG surface mixture was initially not water permeable, however after the first conditioning cycle, voids within the mixture became interconnected and thus the samples became permeable. The PlusRide II® mixtures showed no measurable water permeability. All samples showed visual stripping of five percent or less.

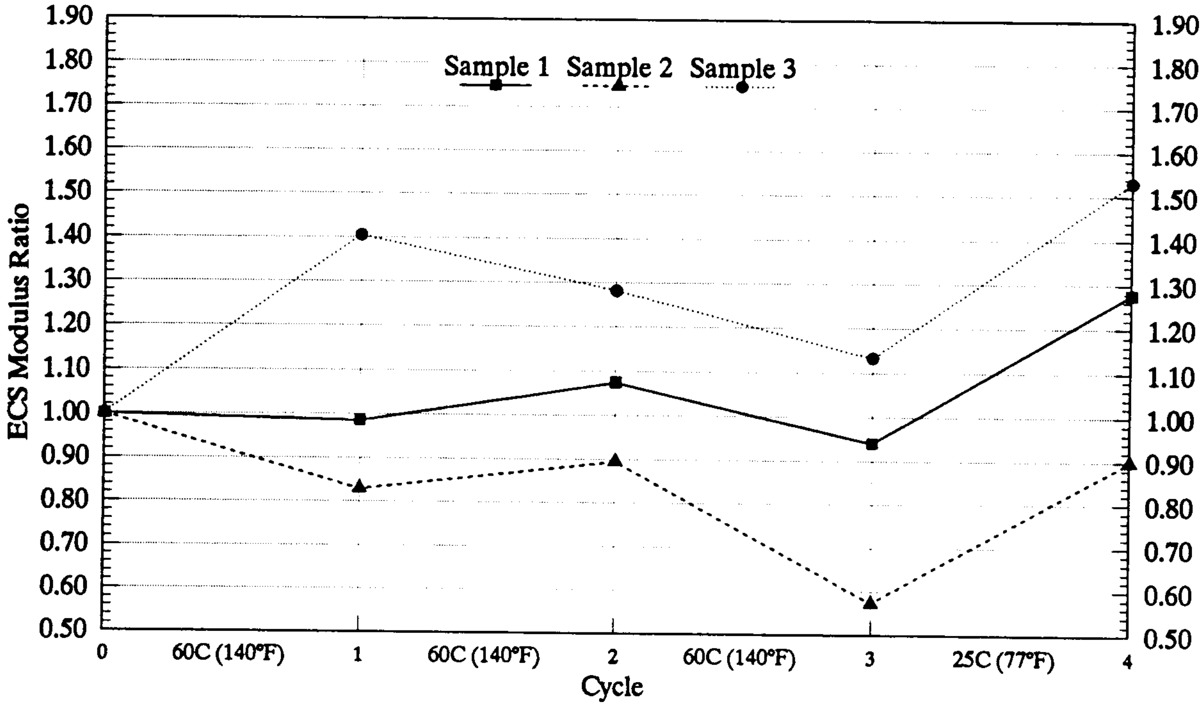


Figure 3.21 ECS Test Results for Class 'A' Surface (AR 4000W) Mixture

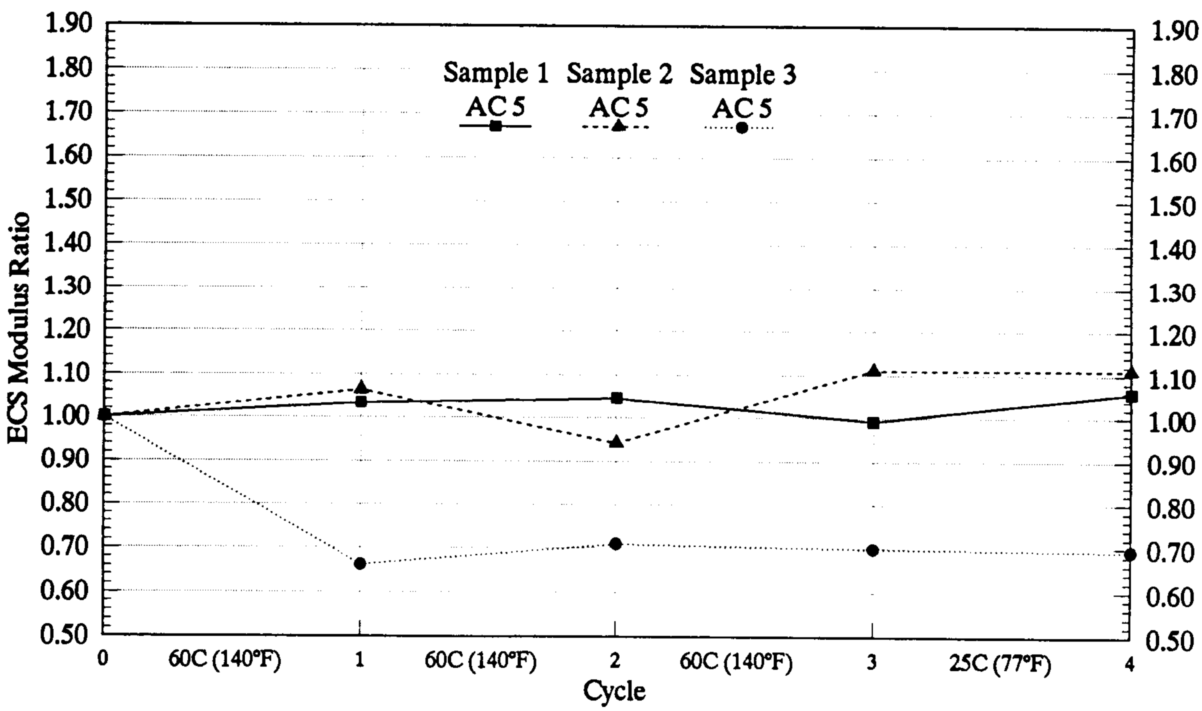


Figure 3.22 ECS Test Results for PlusRide II® Base (AC 5) Mixture with Low Air Voids

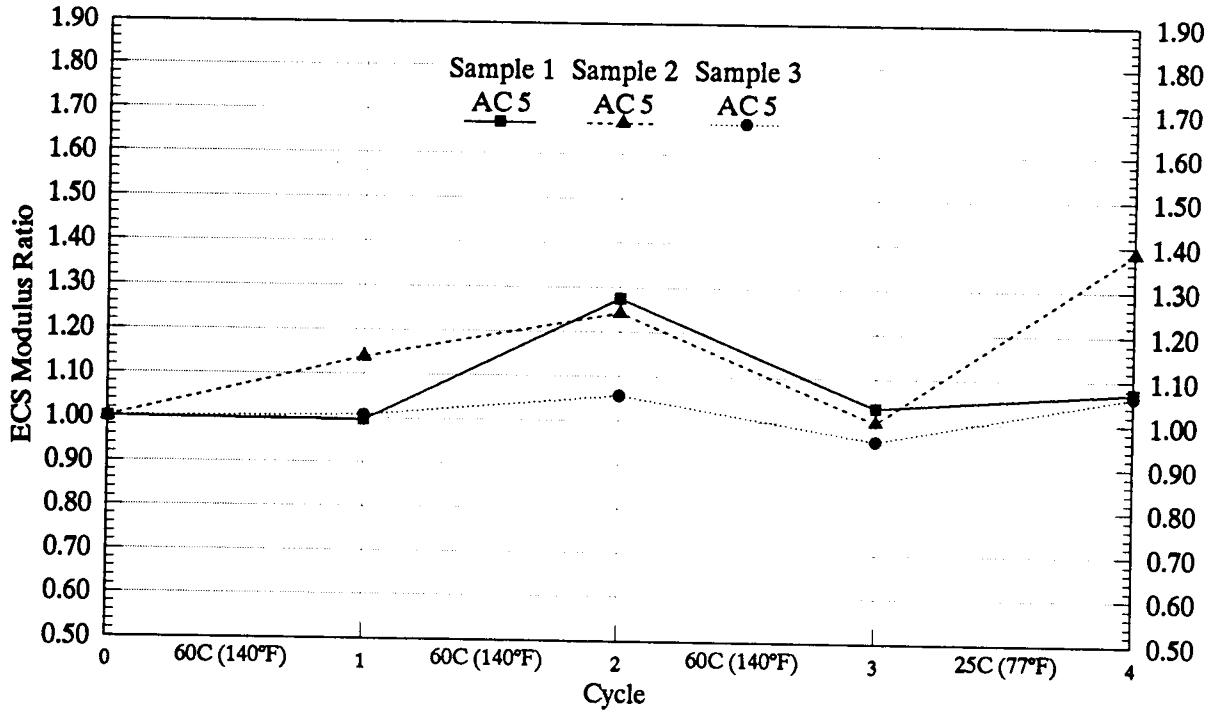


Figure 3.23 ECS Test Results for PlusRide II® Base (AC 5) Mixture

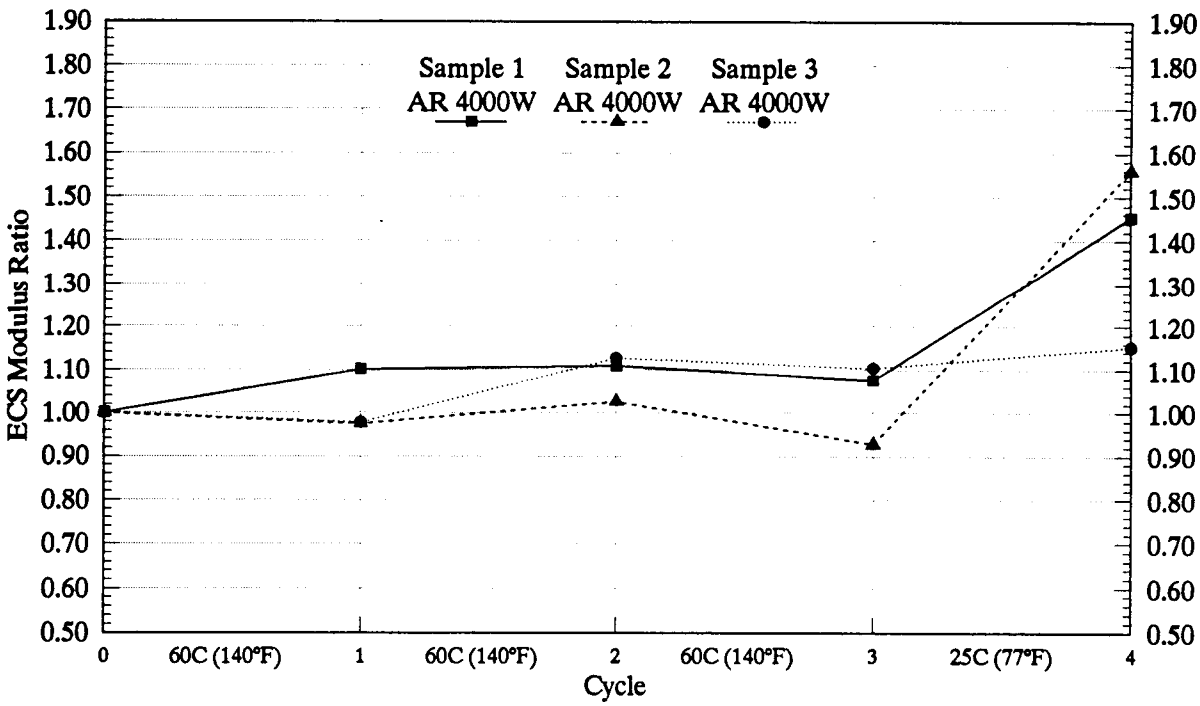


Figure 3.24 ECS Test Results for PlusRide II® Base (AR 4000W) Mixture

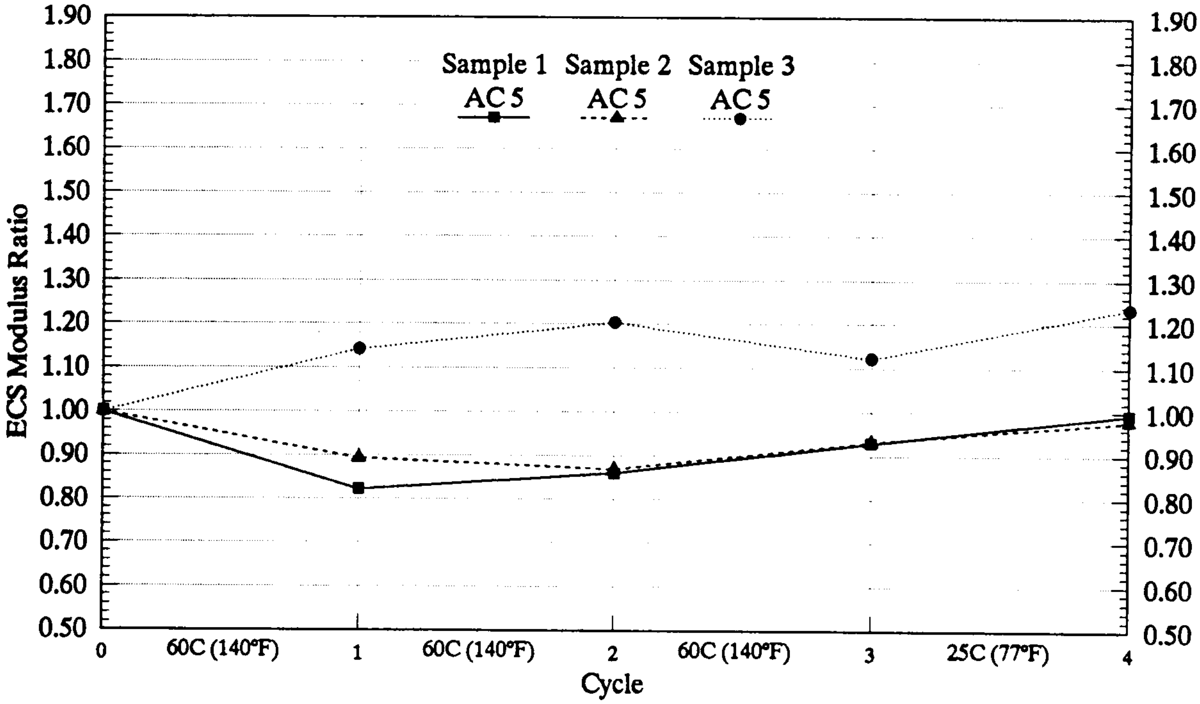


Figure 3.25 ECS Test Results for PlusRide II® Surface (AC 5) Mixture

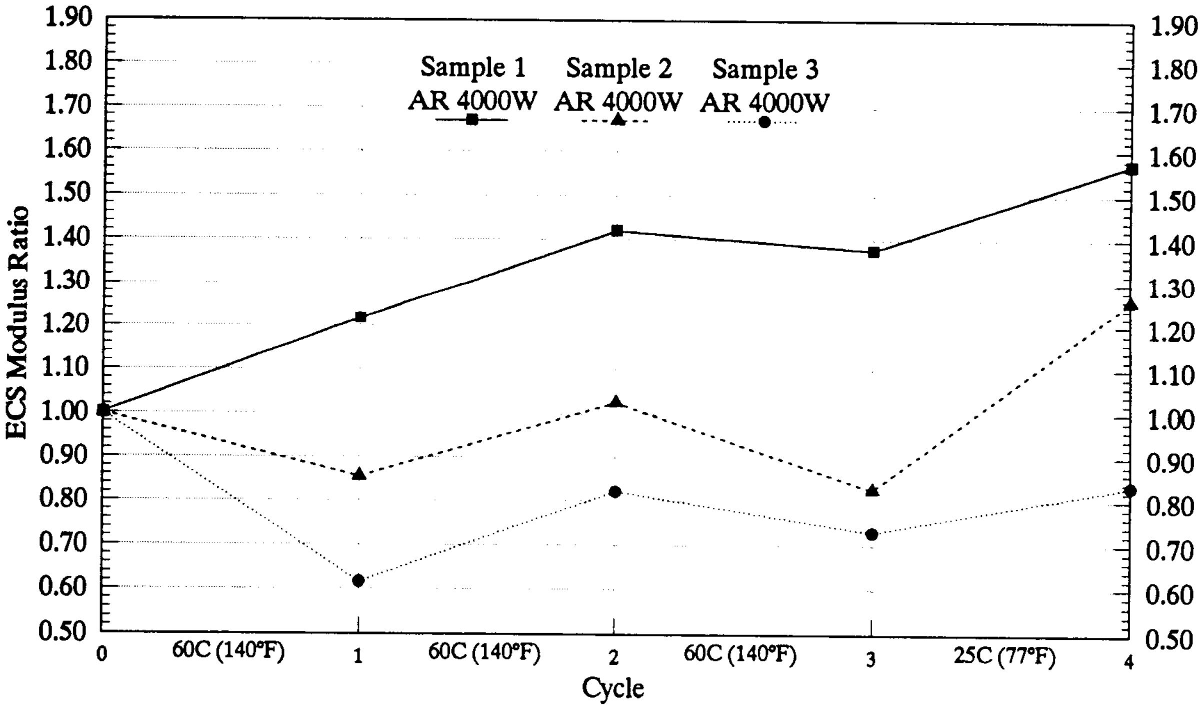


Figure 3.26 ECS Test Results for PlusRide II® Surface (AR 4000W) Mixture

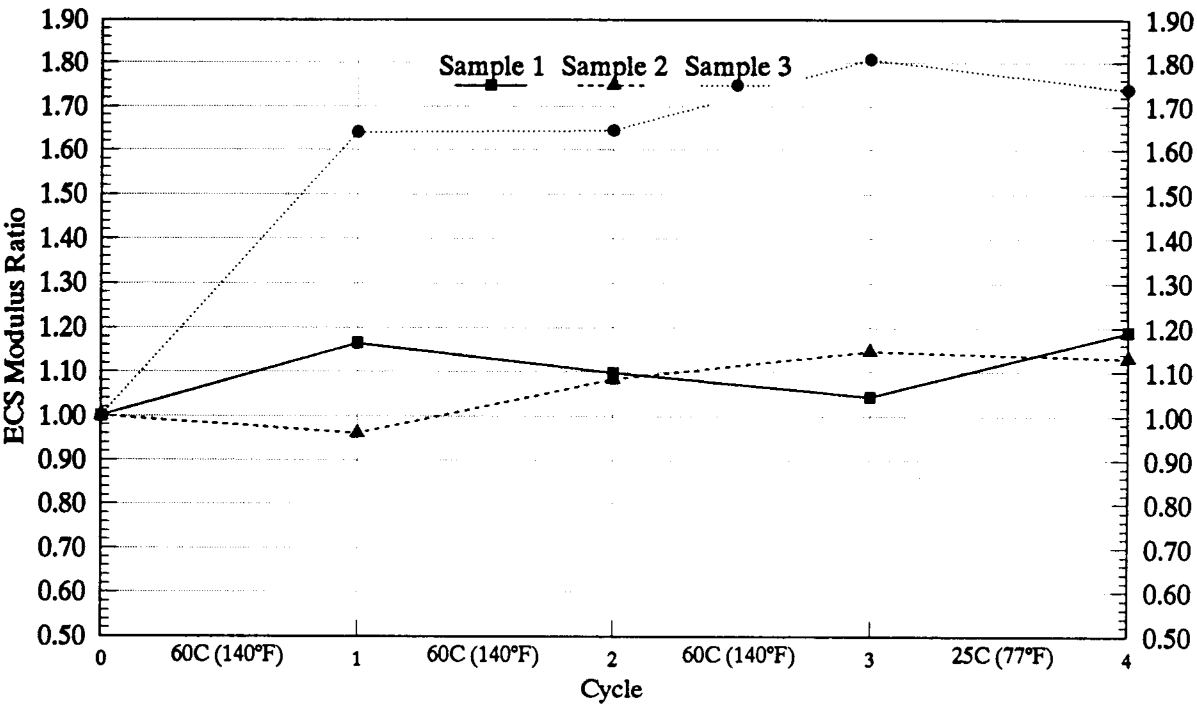


Figure 3.27 ECS Test Results for ARHM-GG Surface (AR 2000) Mixture

4.0 ANALYSIS AND DISCUSSION

With the results obtained in chapter three, an analyses was performed to compare the mixtures. A ranking of mixtures was made using the either Least Significant Difference (LSD) or Duncan's Multiple Range Analysis (Duncan). From the figures presented in section three, averages results within each mixture were determined. These are plotted with other mixture averages for a graphical comparison, and to further emphasize the multiple range analysis.

4.1 Fatigue Cracking

The regression equations associated with the relationships shown in Figure 3.3 are shown in Table 4.1. The rate of cycles to failure (cycles) to strain can be determined from the regression equation coefficient. From regression analysis, the cycle to strain rate of each CRM modified mixtures and Class 'A' surface mixture were statistically similar at a 95% confidence interval. However the relationship for the Class 'A' surface mixture was translated down by a factor of 8. The Class 'A' surface mixture will fail in flexural fatigue before the CRM mixtures. There was virtually no difference between all three CRM mixtures when ranking based on regression criteria. At a constant strain level, the flexural fatigue failure of the CRM materials were 8 times greater than the Class 'A' surface mixture.

Table 4.2 shows the average initial dynamic modulus for each mixture. Duncan analysis was used due to the difference in sample sizes between mixtures.

Table 4.2 also ranks each mixture by dynamic modulus, with increasing modulus from top to bottom. The homogeneous groups show the PlusRide II® base and surface mixtures were statistically similar, whereas the ARHM-GG and Class 'A' surface mixtures were shown to be different from the PlusRide II® mixtures.

Even though the ARHM-GG surface mixture showed a lower dynamic modulus, its performance under fatigue was the same as the PlusRide II® mixtures. The higher dynamic modulus of the Class 'A' surface mixture showed lower resistance to fatigue than either CRM mixture.

Several factors affect the response of an asphalt concrete mixture in the fatigue mode of failure (Harvey et al, 1993). The factors include but were not limited to: asphalt content, air void level, rubber content, and aggregate gradation. As shown in Table 2.3, the CRM mixtures all have a much higher asphalt cement content than the Class 'A' surface mixture, giving them a much better resistance to fatigue failure. Also, as found with most other bituminous mixtures, a lower air void level (found in the PlusRide II® mixtures) provides a much higher resistance to fatigue failure.

4.2 Rutting Susceptibility

Permanent deformation susceptibility was judged using two test procedures. Neither the RSST-CH nor the OSU Wheel Tracker test could be directly correlated to field performance, yet both tests show good relative comparisons between mixtures.

Table 4.1 Summary Statistics for the Flexural Beam Fatigue Test

Mixture	Asphalt Type	Number of Observations	Mean Air Voids (%)	Multiplicative Regression Equation	
				Number of Cycles to Failure (N_f)	R^2
Class 'A' Surface	AR 4000W	6	6.1	$(8.0 \times 10^{13})(\varepsilon)^{(-3.526)}$	91.6%
PlusRide II® Base	AR 4000W	6	3.0	$(2.9 \times 10^{20})(\varepsilon)^{(-5.358)}$	95.5%
PlusRide II® Surface	AR 4000W	6	4.0	$(7.9 \times 10^{16})(\varepsilon)^{(-4.200)}$	98.9%
ARHM-GG Surface	AR 2000	4	6.0	$(1.3 \times 10^{13})(\varepsilon)^{(-2.880)}$	91.3%

Table 4.2 Multiple Range Analysis for Initial Dynamic Modulus

Mixture	Asphalt Type	Number of Observations	LS Mean MPa (psi)	Homogeneous Groups		
ARHM-GG Surface	AR 2000	5	2,144.0 (310,953.2)	■		
PlusRide II® Surface	AR 4000W	6	2,657.5 (385,439.0)		■	
PlusRide II® Base	AR 4000W	6	2,862.9 (415,222.2)		■	
Class 'A' Surface	AR 4000W	6	5,149.4 (746,860.2)			■

4.2.1 Repetitive Shear Strain Test-Constant Height (RSST-CH)

In the RSST-CH each mixture was characterized by the number of shearing repetitions to failure (5% permanent strain). Table 4.3 gives the results for multiple range analysis on the number of repetitions to failure for each mixture. For analysis, the Least Square Mean (LSM) was transformed to the $\log_{10}$ scale for each repetition to failure. Shown in Table 4.3 is the LSM transformed back from the $\log_{10}$ scale. The transformation was necessary to meet the assumptions behind multiple range analysis. Table 4.3 ranks the mixtures by repetitions to failure from worst performance to best. Performance of the PlusRide II® mixtures were statistically similar. The Class 'A' and ARHM-GG surface mixtures were shown to have independent performance. It was readily apparent that the ARHM-GG greatly outperformed the other mixtures with respect to repetitive shear strains. Figure 4.1 shows the average results between each mixture. This figure further demonstrates that the ARHM-GG surface mixture showed better permanent shear resistance than the other mixtures. The Class 'A' surface followed by both PlusRide II® mixtures show the same results as the ranking in Table 4.3.

For the shear strain test, it would seem that the high asphalt content and the low air voids of the PlusRide II® mixtures were the controlling factors affecting the low resistance to permanent shear strain. For the ARHM-GG mixture, despite high binder content, the higher air voids, and the different aggregate gradation, this mixture showed better shear resistance.

Table 4.3 Multiple Range Analysis for Repetitions to Failure

Mixture	Asphalt Type	Number of Observations	LS Mean (back transformed) Repetitions to Failure	Homogeneous Groups		
PlusRide II® Surface	AR 4000W	2	247.0	■		
PlusRide II® Base	AR 4000W	3	254.1	■		
Class 'A' Surface	AR 4000W	2	776.7		■	
ARHM-GG Surface	AR 2000	3	16,633.2			■

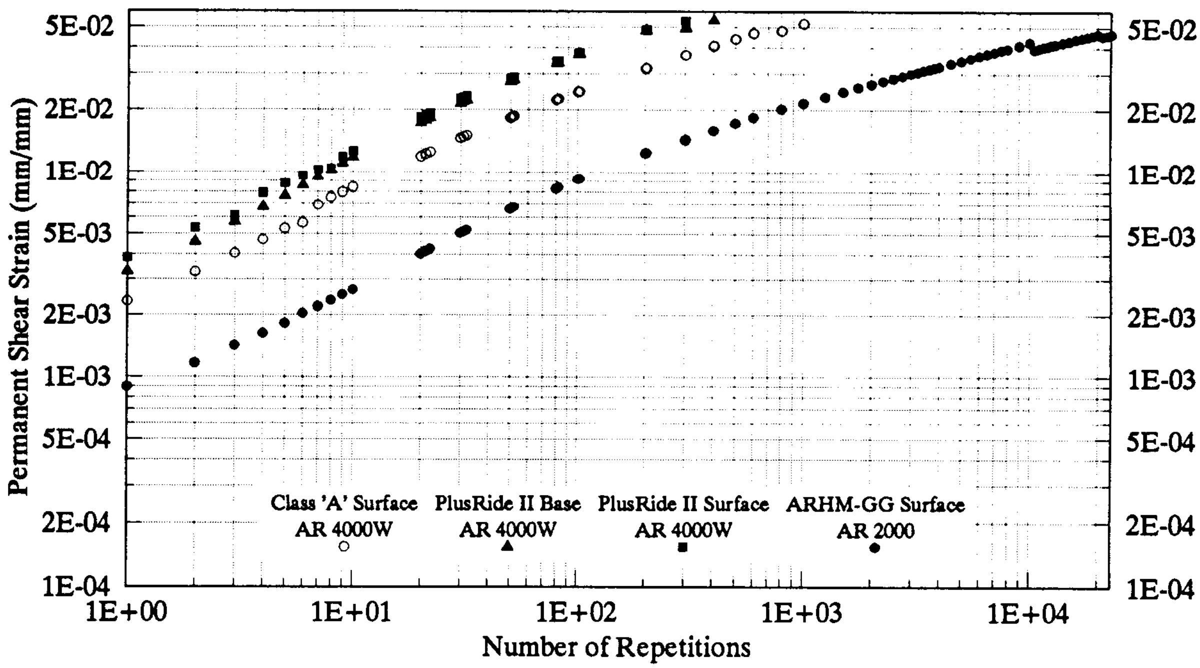


Figure 4.1 Average RSST-CH Results for Each Mixture

4.2.2 OSU Wheel Tracker

A comparison of average rut depth at 1,000 wheel passes was made using Duncan's Multiple Range Analysis. Some mixtures failed soon after 1,000 wheel passes. Therefore, comparison at 1,000 wheel passes was available for all mixtures. Table 4.4 gives the average rut depth and compares the mean values at a 95% confidence interval. The mixtures at the top of the table exhibit the least amount of rutting while the mixtures at the bottom show the highest degree of rutting.

Figure 4.2 graphically shows the average OSU Wheel Tracking results for each mixture. Observation shows little difference between the ARHM-GG and the Class 'A' surface mixtures. These mixtures maintained stable permanent deformation through all 50,000 wheel passes. However, it was evident that the PlusRide II® mixtures approached tertiary failure quite rapidly, within 1,000 wheel passes.

Table 4.4 above shows no statistical difference between the Class 'A' and ARHM-GG surface mixtures. The PlusRide II® surface (AR 4000W) mixture performed better than the other PlusRide II® mixtures. Finally, there was no statistical difference in performance between the PlusRide II® base (AC 5 and AR 4000W) and surface (AC 5) mixtures.

Results from the OSU Wheel Tracker and the RSST-CH were very similar. In a general sense, the mixture types remain in the same order with respect to the degree of permanent deformations. However, the ARHM-GG mixture outperformed the Class 'A' surface mixture in the RSST-CH test. On the other hand, the performance

Table 4.4 Multiple Range Analysis for Rut Depth at 1,000 Wheel Passes

Mixture	Asphalt Type	Number of Observations	LS Mean Rut Depth* mm (in)	Homogeneous Groups
ARHM-GG Surface	AR 4000W	2	1.13 (0.04)	
Class 'A' Surface	AR 2000	2	1.22 (0.05)	
PlusRide II® Surface	AR 4000W	2	5.43 (0.21)	
PlusRide II® Base	AR 4000W	2	9.51 (0.37)	
PlusRide II® Surface	AC 5	2	10.19 (0.40)	
PlusRide II® Base	AC 5	2	11.67 (0.46)	

*Note: At 1,000 Wheel Passes

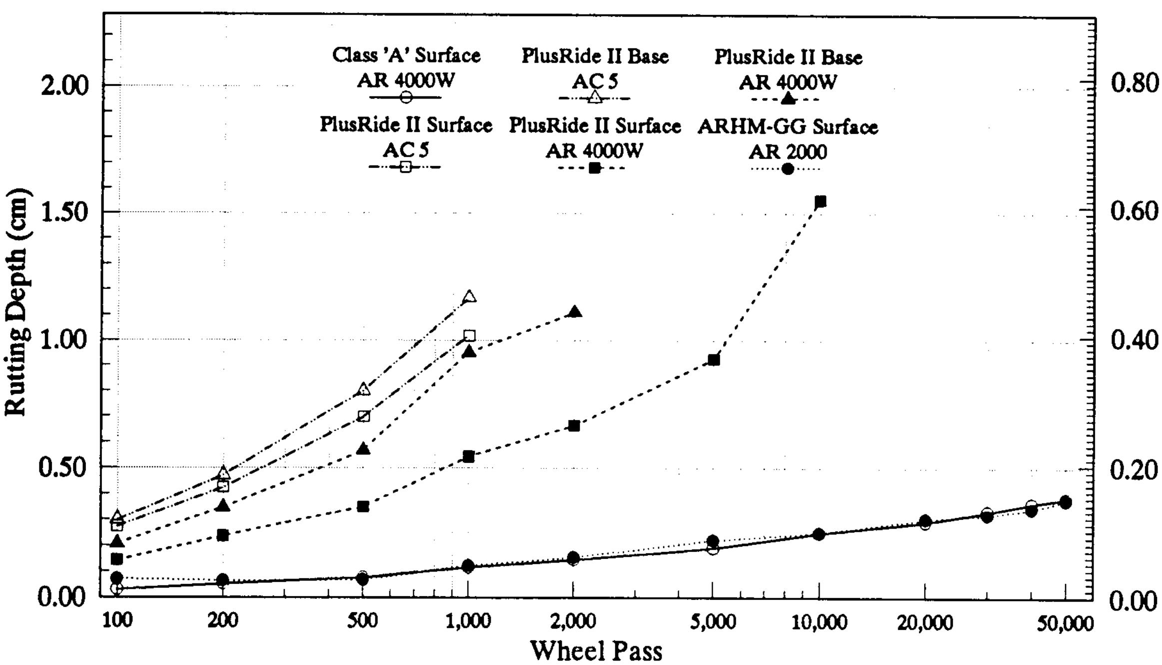


Figure 4.2 Average OSU Wheel Tracker Results for Each Mixture

between the PlusRide II® surface and base (AR 4000W) and base mixtures were shown to be statistically independent in the OSU Wheel Tracker.

4.3 Thermal Cracking

Duncan's Multiple Range Analysis was used to compare the mixtures. Three analyses were performed with respect to mixture type: fracture stress, fracture temperature, and rate of thermal stress with temperature (dS/dT). This rate of thermal stress with temperature is often defined as the second slope, the slope from the last data point down to where the data starts to curve, shown in Figures 4.3. Table 4.5 shows the average results of fracture stress along with average air void content for each mixture. Table 4.6 shows the average results for fracture temperature, and Table 4.7 shows the average results for rate of thermal stress with temperature (dS/dT).

The results from Table 4.5 show the ranking of mixture fracture stress, which causes cracks to develop at low temperatures. The Table 4.5 lists the mixtures by increasing fracture stress from top to bottom. The mixtures most susceptible to fracture stress were the PlusRide II® base and surface (AR 4000W) and ARHM-GG surface mixtures. The mixtures most resistant to fracture stresses due to low temperatures were the PlusRide II® base (AC 5) with low air voids and Class 'A' surface mixtures. It was interesting to note that the harder binder with CRM mixtures (AR 4000W) had lower fracture stress at failure. Conversely, the Class 'A' surface mixture (with AR 4000W) had higher fracture stress. Also, the low air void PlusRide II® base (AC 5) mixture was statistically similar to the Class 'A' surface mixture.

Table 4.5 Multiple Range Analysis for Thermal Fracture Stress

Mixture	Asphalt Type	Number of Observations	Mean Air Voids (%)	LS Mean MPa (psi)	Homogeneous Groups
PlusRide II® Base	AR 4000W	3	3.3	2.57 (372.7)	
PlusRide II® Surface	AR 4000W	4	2.5	2.80 (406.0)	
ARHM-GG Surface	AR 2000	4	4.9	3.19 (463.0)	
PlusRide II® Base	AC 5	3	2.8	3.28 (475.3)	
PlusRide II® Surface	AC 5	4	1.7	3.28 (476.3)	
Class 'A' Surface	AR 4000W	3	4.9	3.66 (531.3)	
PlusRide II® Base	AC 5	3	0	4.13 (599.5)	

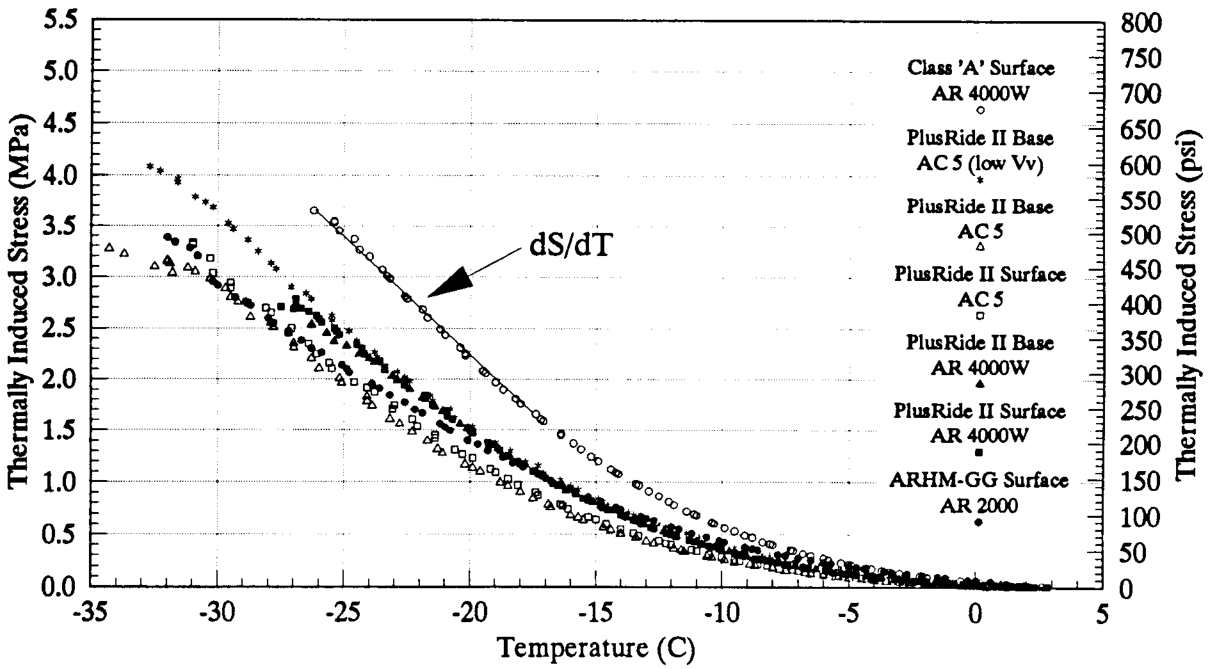


Figure 4.3 Average Thermally Induced Stress Results for Each Mixture

Table 4.6 Multiple Range Analysis for Thermal Fracture Temperature

Mixture	Asphalt Type	Number of Observations	Mean Air Voids (%)	LS Mean C (°F)	Homogeneous Groups	
PlusRide II® Base	AC 5	3	2.8	-33.0 (-27.4)	X	
PlusRide II® Base	AC 5	2	0	-33.0 (-27.3)	X	
ARHM-GG Surface	AR 2000	4	4.9	-32.3 (-26.1)	X	
PlusRide II® Surface	AC 5	4	1.7	-31.9 (-25.4)	X	
PlusRide II® Surface	AR 4000W	4	2.5	-27.6 (-17.7)		X
PlusRide II® Base	AR 4000W	3	3.3	-26.7 (-16.1)		X
Class 'A' Surface	AR 4000W	3	4.9	-26.2 (-15.2)		X

Table 4.7 Multiple Range Analysis for Rate of Thermal Stress with Temperature (dS/dT)

Mixture	Asphalt Type	Number of Observations	Mean Air Voids (%)	LS Mean MPa/C (psi/°F)	Homogeneous Groups	
Class 'A' Surface	AR 4000W	3	4.9	-0.242 (0.031300)	X	
PlusRide II® Base	AC 5	2	0	-0.224 (0.031304)	X	
PlusRide II® Base	AC 5	3	2.8	-0.188 (0.031315)		X
PlusRide II® Surface	AR 4000W	4	3.3	-0.187 (0.031315)		X
PlusRide II® Surface	AC 5	4	2.7	-0.185 (0.031316)		X
PlusRide II® Base	AR 4000W	3	1.7	-0.174 (0.031320)		X
ARHM-GG Surface	AR 2000	4	2.8	-0.164 (0.031324)		X

Table 4.6 ranks the mixture's fracture temperatures from lowest to highest, top to bottom respectively. The most noticeable correlation was mixture with binder type. All mixtures using AC 5 binder were shown to be statistically similar. Also, the ARHM-GG mixture was shown to be statistically similar to the AC 5 binder type mixtures. Finally, all mixtures with AR 4000W binder were grouped together as well.

The statistical similarity between the AR 2000 and the AC 5 mixtures may be traced to the binder properties. AR 2000 binders, by definition, have a higher viscosity and lower penetration value than the AC 5 binders. If the AC 5 binder becomes stiffer, due to reaction with the CRM, then AC 5 may show similar performance characteristics, as the AR 2000 (Heitzman, 1992). However, this does not explain why the fracture temperatures for the PlusRide II® (AR 4000W) mixtures matches the Class 'A' surface mixture. Perhaps there were more volatile components to absorb from the AC 5 binder than the AR 4000W binder, this may explain the anomaly.

Duncan analysis on the second slope (dS/dT) showed only two similar performance groups. Table 4.7 ranks the mean dS/dT from highest to lowest, top to bottom respectively. The Class 'A' surface mixture and the PlusRide II® base (AC 5) mixture with low air voids were shown to be the most susceptible the thermal changes. The PlusRide II® (AC 5 and AR 4000W) and the ARHM-GG mixture were all statistically similar with respect to the second slope.

4.4 Aging (Long Term)

A Least Significant Difference (LSD) analysis was performed on M_R values before and after LTOA. Multiple range analysis reveals how the mixtures compare initially, with lower modulus values increasing to higher modulus values, top to bottom of Table 4.8 respectively. The mixture rankings remained the same before and after aging. A ratio of these values show that there was no statistical difference, with respect to the degree of aging, between the CRM mixtures.

Figure 4.4 shows graphically the initial STOA modulus with a solid bar, and the amount of modulus increase due to LTOA with the shaded bar. Adding both bars gives the LTOA modulus. Notice only a small increase in modulus with the CRM mixtures. Conversely, there was a substantial M_R increase in the Class 'A' surface mixture.

The retained modulus ratio was used to determine the relative effect of aging between each mixture and its resistance to long term oven aging. The LSD analyses reveals the similarity of the CRM mixtures to aging. At a 95% confidence interval, all CRM mixtures exhibit the same M_R ratio ranking after LTOA. The only significantly higher M_R ratio was the Class 'A' surface mixture. It was interesting to note that the CRM mixes have the same M_R ratio, regardless of mixture. Similarly, the CRM mixes had lower initial modulus values and retained these relatively lower values after aging.

Table 4.8 Multiple Range Analysis for Age Hardening

Mixture	Asphalt Type	Mean Air Voids	Number of Observations	STOA LS Mean MPa (ksi)	STOA Homogeneous Groups				LTOA LS Mean Mpa (ksi)	LTOA Homogeneous Groups				RETAINED LS Mean	RETAINED Homogeneous Groups	
PlusRide II® Base	AC 5	3.8	3	672.2 (97.5)	■				839.1 (121.7)	■				1.23	■	
PlusRide II® Surface	AC 5	3.5	3	772.9 (112.1)	■				1,013.5 (147.0)	■				1.31	■	
PlusRide II® Base	AC 5	1.5	3	1,080.4 (156.7)		■			1,328.6 (192.7)		■			1.25	■	
PlusRide II® Base	AR 4000W	4.7	3	1,197.6 (173.7)		■			1,441.0 (209.0)		■			1.21	■	
PlusRide II® Surface	AR 4000W	5.0	3	1,227.3 (178.0)		■			1,513.4 (219.5)		■			1.17	■	
ARHM-GG Surface	AR 2000	7.7	3	1,652.7 (239.7)			■		1,994.7 (289.3)			■		1.21	■	
Class 'A' Surface	AR 4000W	8.3	3	2,266.3 (328.7)				■	4,136.9 (600.0)				■	1.83		■

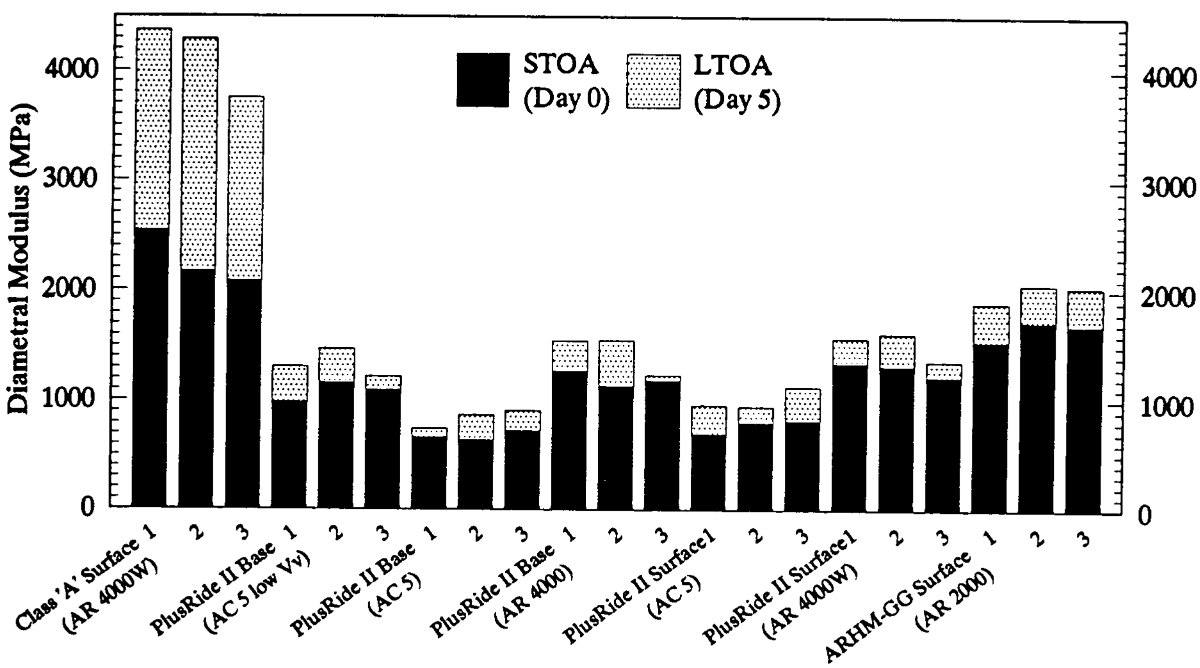


Figure 4.4 LTOA Test Results for Each Mixture

In the field, pavements with CRM have shown reduced aging effects (Heitzman, 1992). One component in crumb rubber is carbon black. Carbon black is added to rubber during the tire manufacturing process. It prevents tires from degrading due to the environment before the end of their useful life. This effect has also been noted in CRM mixtures. In some instances, carbon black is added to asphalt concrete mixtures to improve binder viscosity, and help the binder retain thicker films on the aggregate. These thicker films are said to delay the detrimental effects of oxidation (Heitzman, 1992). Previous research had indicated that higher air voids in a mixture increase the rate of mixture aging. In this study, the ARHM-GG mixtures have high air voids, almost the same as the Class 'A' surface mixture. However, the ARHM-GG aged at the same rate as the PlusRide II® mixtures, which have a much lower air void content. Also, the PlusRide II® base (AC 5) mixture, with virtually no air voids, showed the same degree of aging as the other CRM mixtures. The CRM mixtures do share one property, they have very high binder contents. Therefore, in addition to the carbon black effect, the high binder contents may be contributing to low aging susceptibility.

4.5 Water Sensitivity

Results from the ECS show each mixture has good resistance to water sensitivity. All mixtures experienced increased ECS M_R after the fourth conditioning cycle. This would suggest little or no loss of adhesion or cohesion within the mixtures. The ARHM-GG surface mixtures showed a constant increase in strength through all cycles. It was apparent that all mixtures may be experiencing stiffness gains through either mixture oxidation during water conditioning, or increased sample density due to repeated loading. The PlusRide II® mixtures did not experience any water damage. Due to the low (interconnecting) void content in the PlusRide II® mixtures, there was not enough water entering the specimens to cause water damage. This behavior was consistent with the pessimum voids concept developed by Terrel and Al-Swailmi (1993). It was previously determined that an ECS modulus ratio of 0.7 or lower was a mixture failure (Allen et al, 1993). Hence, it may be concluded that none of the mixtures failed the ECS test.

Table 4.9 shows the LSD analysis of each mixture's ECS M_R ratio with respect to cycle. The variation between cycles, from the multiple range analysis, shows the lack of moisture sensitivity. Cycle zero was removed from the analysis since, in all cases, it was 1.00. From this analysis, each result seems to vary so much that there was no defined pattern to moisture sensitivity with respect to each mixture. The results above seem to show that the ECS was not evaluating the mixtures effectively. Figure 4.5 shows the average ECS M_R with each cycle. Evaluation of these results would show the same, indefinite conclusion between mixtures.

Table 4.9 Multiple Range Analysis for ECS M_R Ratio at Each Cycle

Mixture	Asphalt Type	Mean Air Voids	Number of Observations	Cycle 1		Cycle 2		Cycle 3		Cycle 4	
				LS Mean	Homogeneous Groups	LS Mean	Homogeneous Groups	LS Mean	Homogeneous Groups	LS Mean	Homogeneous Groups
Class 'A' Surface	AR 4000W	6.2	3	1.07		1.08		0.88		1.24	
PlusRide II® Base	AC 5	1.5	3	0.92		0.90		0.93		0.95	
PlusRide II® Base	AC 5	3.6	3	1.05		1.19		1.00		1.17	
PlusRide II® Base	AR 4000W	5.2	3	1.02		1.09		1.04		1.39	
PlusRide II® Surface	AC 5	3.5	3	0.95		0.98		0.99		1.07	
PlusRide II® Surface	AR 4000W	4.4	3	0.90		1.09		0.98		1.22	
ARHM-GG Surface	AR 2000	7.1	3	1.26		1.28		1.33		1.35	

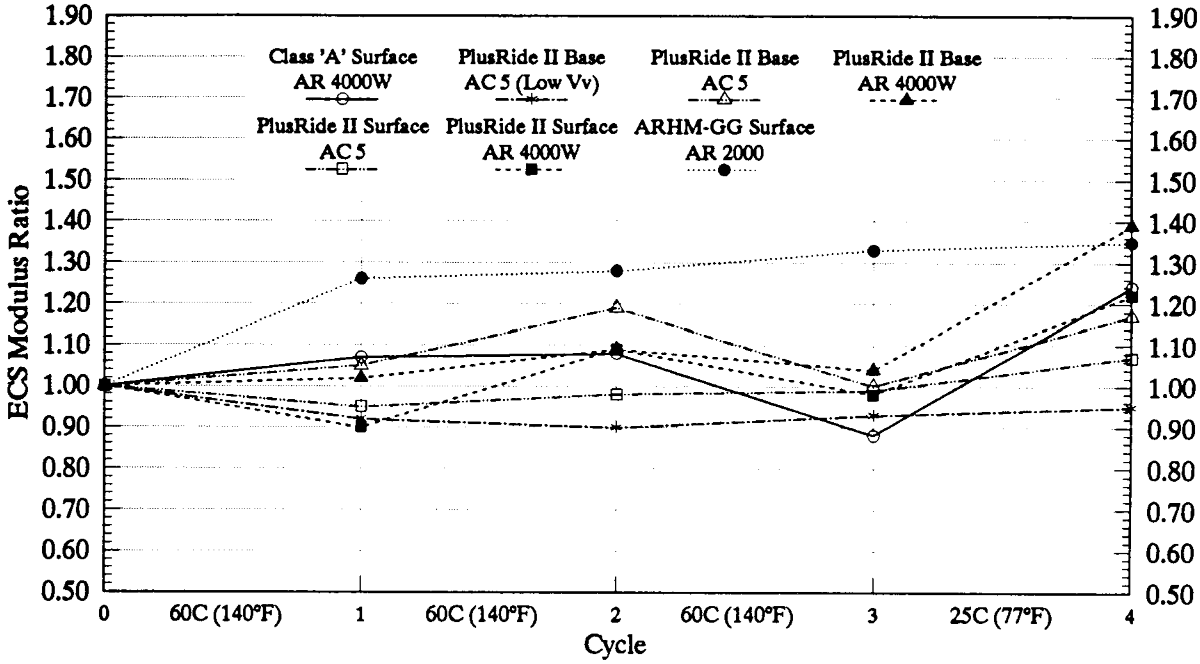


Figure 4.5 Average ECS M_R Ratio for Each Mixture

5.0 CONCLUSIONS AND RECOMMENDATIONS

Evaluation of these mixtures using performance based test procedures provides an indication of relative field performance. However, direct correlation to field performance was not possible. For now, these performance based tests accelerate the deterioration of a mixture, and allows for relative comparisons of mixtures. With these limitations in mind, the following conclusions and recommendations were appropriate.

5.1 Conclusions

Depending on the type of mixture testing conducted, the CRM mixtures demonstrated worse, the same, or better results than the conventional Class 'A' surface mixture. Although the PlusRide II® (AR 4000W) mixtures were ultimately preferred (and placed) in the field, PlusRide II® (AC 5) is included for comparison. Since the PlusRide II® base (AC 5) mixture with low air voids was only used for laboratory comparison, the results are not included with these conclusions or recommendations. Several observations and conclusions can be made with respect to each type of mixture:

ARHM-GG Surface Mixture:

- Fatigue resistance was improved by a factor of eight compared to the Class 'A' surface mixture, but the rate of cycles to failure to strain were statistically similar to the Class 'A' surface mixture.
- Rutting resistance, with respect to the simple shear test, was superior to all other mixtures. The OSU Wheel Tracker, on the other hand, showed no difference between the Class 'A' and ARHM-GG surface mixtures.
- Thermal fracture stress was statistically similar to the Class 'A' surface mixture. However, fracture temperature and rate of thermal stress with temperature was similar to the PlusRide II® mixtures.
- Long term aging resistance was improved over the Class 'A' surface mixture and statistically similar to the PlusRide II® mixtures.
- The mixture showed low susceptibility to moisture damage. An increase in ECS M_R ratio suggests the mixture may experience stiffening, similar to aging, due to moisture and temperature interaction.

PlusRide II® Base and Surface Mixtures:

- The fatigue resistance for the PlusRide II® (AR 4000W) base and surface mixtures show similar performance, eight times better than the Class 'A' surface mixture. No PlusRide II® (AC 5) mixtures were tested in fatigue.

- The resistance to permanent deformations, with respect to the RSST-CH, for both PlusRide II® (AR 4000W) surface and base mixtures was low. No PlusRide II® (AC 5) mixtures were tested in the RSST-CH. All four PlusRide II® mixtures (AR AC 5 and 4000W) were tested in the OSU Wheel Tracker. Again, poor rutting resistance was observed in all PlusRide II® mixtures.
- Thermal fracture stress of the PlusRide II® (AC 5 and AR 4000W) mixtures were less resistant than the Class 'A' surface mixture. Fracture temperatures were grouped by binder type. Rate of thermal stress with temperature (dS/dT) was the same for all PlusRide II® mixtures.
- Long term aging resistance was improved over the Class 'A' surface mixture. The PlusRide II® mixtures were relatively insensitive to aging.
- All mixtures showed low susceptibility to moisture damage. This could be attributed to low air voids and high binder contents, which prevent moisture interaction.

To gain an overall understanding Table 5.1, provides a summary of the CRM mixture performances. Each of the CRM mixtures were compared with the Class 'A' surface mixture. Each CRM mixture either performed better, the same, or worse than the Class 'A' surface mixture in each of the five performance categories.

Table 5.1 CRM Mixture Performance Summary

Mixture	Asphalt Type	Fatigue Cracking	Permanent Deformation	Thermal Cracking	Age Hardening	Water Sensitivity
PlusRide II® Base	AC 5	Not Tested	Worse	Same	Better	Same
PlusRide II® Surface	AC 5	Not Tested	Worse	Same	Better	Same
PlusRide II® Base	AR 4000W	Better	Worse	Worse	Better	Same
PlusRide II® Surface	AR 4000W	Better	Worse	Worse	Better	Same
ARHM-GG Surface	AR 2000	Better	Better/Same*	Same	Better	Same

*Note: Better for RSST-CH, Same for OSU Wheel Tracker

5.2 Recommendations

From these performance-based tests the following recommendations were made:

- The ARHM-GG surface mixture could be readily used where the Class 'A' surface mixture is now specified.
- PlusRide II® (AC 5 and AR 4000W) base and surface mixture designs should be modified such that their performance is improved with respect to permanent deformation (rutting).
- The PlusRide II® (AR 4000W) base and surface mixture designs should be modified such that their performance is improved with respect to thermal fracture susceptibility.
- Long term evaluation of these mixtures in the field should continue. Only through field validation will the mixture performance results, obtained herein, be verified.

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APPENDICES

APPENDIX A

Standard Practice for

Preparation of Test Specimens of Bituminous Mixtures by Means of Rolling Wheel Compaction

Standard Practice for

Preparation of Test Specimens of Bituminous Mixtures by Means of Rolling Wheel Compaction

SHRP Designation: M-008[1]

1. SCOPE

1.1 This method describes the mixing and compaction procedures to produce large slab specimens (approximately 101.6 mm × 762 mm × 762 mm) of bituminous concrete in the laboratory by means of a mechanical rolling wheel compactor. It also describes the procedure for determining the air void content of the specimens obtained.

1.2 The values stated in SI units are to be regarded as the standard.

1.3 *This standard may involve hazardous materials, operations and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. APPLICABLE DOCUMENTS

2.1 *AASHTO Test Methods:*

T11-85	Amount of Material Finer than 75-µm Sieve in Aggregate
T27-84	Sieve Analysis of Fine and Coarse Aggregates
T246-81	Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus

2.2 *ASTM Test Methods:*

[1]This standard is based on SHRP Product 1015.

C 117-90	Materials Finer than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing
C 136-84a	Sieve Analysis of Fine and Coarse Aggregates
D 1561-81a	Preparation of Bituminous Mix Test Specimens by Means of California Kneading Compactor
D 2041-78	Test Method for Theoretical Maximum Specific Gravity of Bituminous Paving Mixtures
D 2493-91	Standard Viscosity Temperature Chart for Asphalts

3. APPARATUS

3.1 *Rolling Wheel Compactor*—A mechanical, self-propelled rolling wheel compactor with forward/reverse control such as that shown in figure A.1 for compaction of asphalt concrete mixtures. It must weigh a minimum of 1,000 kg and possess the capability of increasing the weight to 1,500 kg. The load applied must be in the static mode.

3.2 *Mold*—A mold to hold the bituminous mix as shown in figure A.2. The mold is composed of one lift 101.6 mm thick.

3.3 *Ovens*—Forced-draft electric ovens of sufficient size, capable of maintaining a uniform temperature between 100 ± 3°C and 200 ± 3°C. It is preferable to have ovens with a capacity of 2.8×10^{-2} m^3 to 4.2×10^{-2} m^3 for asphalts and 0.7 m^3 to 0.85 m^3 for aggregates.

3.4 *Specimen Mixing Apparatus*—Suitable mechanized mixing equipment is required for mixing the aggregate and the bituminous material. It must be capable of maintaining the bituminous mixture at the selected mixing temperature, and allow the aggregate to be uniformly and completely coated with asphalt during the mixing period (approximately 4 min). It is preferable to have a mixer with a capacity of 7×10^{-2} m^3 to 8.5×10^{-2} m^3. A conventional concrete mixer fitted with infrared propane heaters has been found to be suitable.

3.5 *Coring and Saw Cutting Equipment*—Mechanized coring and saw cutting equipment capable of coring specimens 101.6 mm to 203.2 mm in diameter and beams of different sizes from an asphalt concrete slab. It is preferable to dry-cut the cores and beams.

3.6 *Balance*—Two balances are required: one with a capacity of 5 kg or more and sensitive to 1.0 g or less, and the other with a capacity between 45 and 120 kg, and sensitive to 0.5 kg or less.

3.7 *Miscellaneous Apparatus:*

3.7.1 Digital thermometers with thermocouple probe

3.7.2 Spatulas, trowels, scoops, spades, rakes

3.7.3 Heat-resistant gloves

3.7.4 Metal pans

3.7.5 Socket wrench, sockets, screw drivers, crescent wrench

3.7.6 Lubricant for mold (e.g., PAM® cooking oil or equivalent)

3.7.7 Tape measure

3.7.8 Parafilm (manufactured by American National Can Co., Greenwich, CT)

3.7.9 Pallet jack

4. MATERIAL PREPARATION

4.1 *Aggregate*—Aggregate to be used for specimen preparation should be prepared in accordance with AASHTO T11-85 and T27-84. After the aggregate has dried to a constant weight, remove the aggregate from the oven, and cool to room temperature. Then sieve into the separate size fractions necessary to accurately recombine into test mixtures that conform to specified grading requirements.

4.2 *Determine material quantities*—Calculate the quantity of material required to achieve the desired air void content. These calculations are shown in section 7.

4.3 *Mixing Temperature*—Set the oven to the mixing temperature. For mixes employing unmodified asphalt cements, the temperature of the aggregate and the asphalt at the time mixing begins shall be in accordance with the temperatures specified in AASHTO T246-82 or ASTM D 1561-81a. The temperature selected should correspond to a viscosity of 170 ± 20 mm^2/s (based on the original asphalt properties).

4.4 *Heating the asphalt*—Asphalts supplied in 19-L epoxy-coated containers must first be heated to 135°C in a forced draft oven. The container should be loosely covered with a metal lid. This first heating is to subdivide the 19-L sample into smaller containers for subsequent use. After approximately 1.5 h, remove the sample from the oven, and stir with a large spatula or metal rod. The sample should be stirred every half hour to ensure uniform heating. Typically, a 19-L sample will require approximately 5 h for the entire heating cycle.

NOTE 1.—Watch for signs of blue smoke from the asphalt. This would indicate overheating. If a noticeable quantity of smoke is observed, then the oven temperature should be reduced by 5 to 10°C.

Place paper or newsprint on the floor in a well-ventilated area. Place empty and clean 1-L containers on the paper in a sequence convenient for pouring the hot asphalt. Different-sized containers may also be used. It is important that the containers be properly labelled with self-adhesive labels or a diamond-tipped pencil prior to pouring.

Remove the 19-L container from the oven and stir the asphalt for approximately 1 minute. Fill the containers, taking care that the labels on the containers are not obliterated. After filling, close all containers tightly, and allow to cool to room temperature. Store at a temperature of 10°C. Closing the containers prior to cooling will produce a vacuum seal.

4.5 Prior to mixing, set the oven to the mixing temperature as determined in section 4.3. Place a sufficient number of 1-L cans (with a total weight greater than that calculated in section 7.8) of asphalt in the oven at least 2 h prior to mixing. Monitor the temperature of the asphalt periodically. When the temperature approaches the mixing temperature, transfer the asphalt into a large pot (e.g., an 11-L stock pot) and at the same time weigh the amount of asphalt added to the pot. Transfer enough asphalt to equal the amount calculated in section 7.8 plus an extra 80 g (to account for the quantity retained in the pot after asphalt has been added to the aggregate). Then place the pot in the oven and continue to monitor the temperature periodically.

NOTE 2.—This constitutes the second heating of the asphalt. Any asphalts that have been heated more than twice must be discarded.

4.6 *Mixing*—Preheat the mixer approximately 1 h prior to mixing. Place coarse aggregate in the mixer, followed by the fine aggregate, and then the asphalt. Mix for approximately 4 min to ensure uniform coating of the aggregate.

4.7 *Short Term Aging*—After mixing, remove the mixture from the mixer and place it in metal pans. Place the mixture in an oven set at a temperature of 135° ± 1°C for 4 h ± 1 min. Stir the mixture once an hour.

5. COMPACTION

5.1 Assemble the mold as shown in the schematic illustrated in figure A.2. Preheat the mold with a "tent" equipped with infrared heat lamps (see figure A.3).

5.2 Check the oil and fuel levels in the rolling wheel compactor and refill if necessary. Start the compactor and allow it to warm up. Spray a mild soapy solution on the rollers.

5.3 Sparingly apply a light oil (e.g., PAM® cooking oil) to the base and sides of the mold.

5.4 Remove a pan of mixture from the oven and place it in the center of the mold. Level the mixture using a rake while at the same time avoiding any segregation of the mixture (i.e., avoid any tumbling of the coarse aggregate). Repeat this process until the mold is filled with the required quantity of material to achieve the target air void content. This should be all of the pre-weighed material. Tamp the mixture to achieve as level a surface as possible.

5.5 Monitor the temperature of the mixture at the surface, at mid-depth, and at the bottom in various locations. Allow the mixture to cool until the coolest temperature corresponds to the pre-established compaction temperature (see notes 3 and 4).

NOTE 3.—The field compaction temperature should be used. As general guide, the compaction temperature to be used for most typical asphalt cements (AC-5 to AC-30) should correspond to an equiviscous temperature of 280 ± 30 mm^2/s (based on original binder properties) as described in section 4.3. If necessary, the mixture should be placed in an oven until it reaches a uniform temperature.

NOTE 4.—Lower compaction temperatures in the range between 115°C and 138°C may be necessary depending on the compactibility of the mixtures used under the rolling wheel compactor.

5.6 Compact the mixture until the rollers bear down on the compaction stops (steel channels with depths equal to slab thickness inserted in the mold as shown in figure A.2). When compacting, each pass of the roller must extend from the ramp to the platform in a continuous motion, with no stops on the mixture. After the first few passes, it may be necessary to scrape bituminous mixture off the rollers and reshape the mixture.

5.7 When compaction is complete, let the slab cool overnight (typically 15 to 16 h) before removing the mold. If the slab is still warm to the touch, do not remove the mold. Do *not* place any weights on top of the slab.

5.8 After the slab is completely cooled, remove the slab from the mold together with the removable base of the mold (constructed of particle board) before placing on a pallet jack.

5.9 The slab should be dry cored and sawn into the desired specimen shapes as soon as possible. Note that the specimens should not be taken within 5 to 6.3 cm of the outside edges of the slab. This is approximately 2 to 2.5 times the nominal top size of the aggregate used. Store approximately 3 kg of the wasted mix for the determination of the theoretical maximum specific gravity as described in section 6.

6. CALCULATE THE AIR VOID CONTENT

6.1 Weigh the dry, unwrapped, room-temperature–stabilized specimen. Record this as *Mass in Air, A*.

6.2 Wrap the specimen in Parafilm so that it is completely watertight with no air bubbles between the Parafilm and the specimen. Use the minimum amount of Parafilm necessary. Weigh the specimen in air and record this as *Mass in Air with Parafilm, B*.

6.3 Weigh the wrapped specimen suspended in water at 25°C (77°F), taking the reading as soon as the balance stabilizes. Record this as the *Mass in Water with Parafilm, C*.

6.4 Determine the specific gravity of Parafilm at 25°C, or assume a value of 0.9. Record this as D.

6.5 Calculate the bulk specific gravity of the specimen as follows:

$$G_{mb} = \left[\frac{A}{B - C - \left(\frac{B-A}{D} \right)} \right] \qquad (1)$$

where

A=Mass of dry uncoated specimen in air, g
B=Mass of Parafilm-coated specimen in air, g
C=Mass of Parafilm-coated specimen in water, g
D=Specific gravity of Parafilm at 25°C (77°F)
G_{mb}=bulk specific gravity

6.6 Determine the theoretical maximum specific gravity, G_{mm}, in accordance with ASTM D 2041-78.

6.7 Calculate the air void content as follows:

$$Air\ Voids = \left[1 - \left(\frac{G_{mb}}{G_{mm}}\right)\right] \cdot 100\% \tag{2}$$

7. CALCULATE THE QUANTITY OF BITUMINOUS MIX REQUIRED

7.1 Measure the dimensions (height, length and width) of the compaction mold that will contain the compacted slab. Record this as H, L and W in dm.

7.2 Determine the volume (V) of the mold in units of cm^3.

7.3 Determine the maximum specific gravity of the bituminous mix at the desired asphalt content in accordance with ASTM D 2041. Record this as G_{mm}.

7.4 Determine the target bulk specific gravity for the compacted slab based on the target air void content:

$$G_{mb} = G_{mm}\left[1 - \frac{\%AV}{100}\right] \tag{3}$$

where

G_{mb}=target bulk specific gravity of the compacted slab
$\%AV$=target air voids of the compacted slab (percent)

7.5 Determine the unit mass (density) of the compacted slab:

$$\rho = G_{mb}\,\rho_w \tag{4}$$

where

ρ=unit mass of the compacted slab, kg/m^3
ρ_w=unit mass of water, kg/m^3

7.6 Determine the mass, M (in kilograms) of the compacted slab:

$$M = \rho\, V$$

7.7 Determine the mass of the aggregate required for compaction as shown below in equations 5 and 6. Equation 5 uses the asphalt content based on the dry mass of the aggregate, whereas equation 6 uses the asphalt content based on total mass of the mixture.

$$M_{aggr} = \left[\frac{M}{\left(1+\frac{\%AC}{100}\right)}\right] \tag{5}$$

$$M_{aggr} = M\left[1-\frac{\%AC}{100}\right] \tag{6}$$

where

M_{aggr} =total mass of aggregate, kg
$\%AC$= asphalt content

7.8 Determine the mass of asphalt binder required for compaction as shown in equations 7 and 8 below. Equation 7 uses the asphalt content based on the dry mass of the aggregate, whereas equation 8 uses the asphalt content based on total mass of the mixture.

$$M_{AC} = M_{aggr}\left[\frac{\%AC}{100}\right] \tag{7}$$

$$M_{AC} = M\left[\frac{\%AC}{100}\right] \tag{8}$$

where

M_{AC}=mass of asphalt binder, kg

8.REPORT

8.1 The report shall include the following information:

8.1.1 *Bituminous Mixture Description*—bitumen type, bitumen content, aggregate type, aggregate gradation, and air void percentage.

8.1.2 Mix and compaction temperatures, °C.

8.1.3 Mass of specimen in air, g (*A*)

8.1.4 Mass of specimen in air with Parafilm, g (*B*)

8.1.5 Mass of specimen in water with Parafilm, g (*C*)

8.1.6 Specific gravity of Parafilm (*D*)

8.1.7 Bulk specific gravity, G_{mb}

8.1.8 Maximum specific gravity, G_{mm}

8.1.9 Air void content of specimen, percent

8.1.10 Dimensions of mold, cm

8.1.11 Volume of mold, cm^3

8.1.12 Unit mass of compacted slab, kg/cm^3

8.1.13 Mass of mix required for compaction, kg

8.1.14 Mass of aggregate required for compaction, M_{aggr} (kg)

8.1.15 Weight of asphalt required for compaction, M_{AC} (kg)

8.1.16 Time of mixing, minutes

8.1.17 Time of compaction, minutes

9. PRECISION

9.1 A precision statement has not yet been developed for this test method.

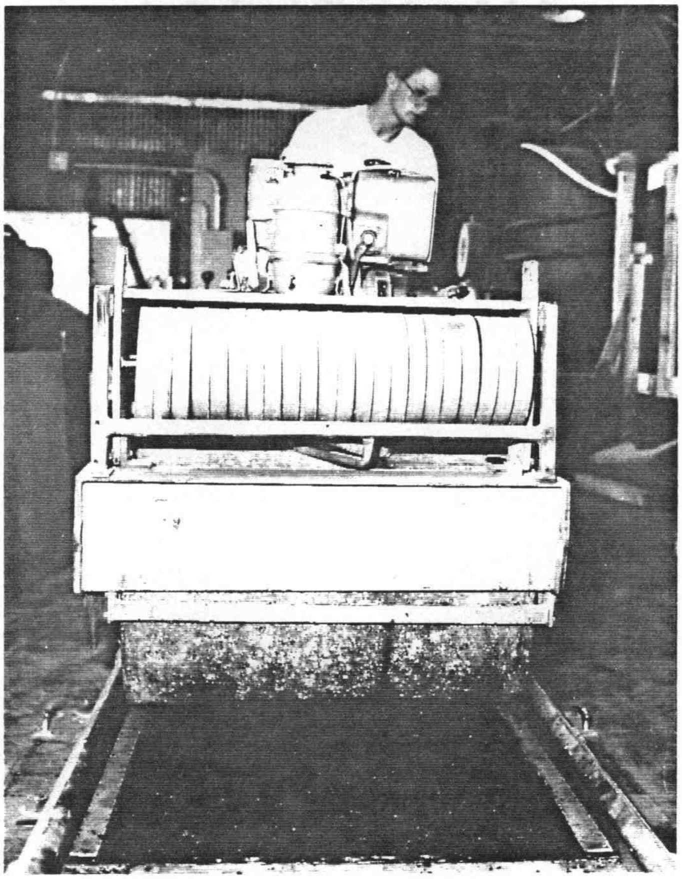

Figure A.1 Rolling Wheel Compactor

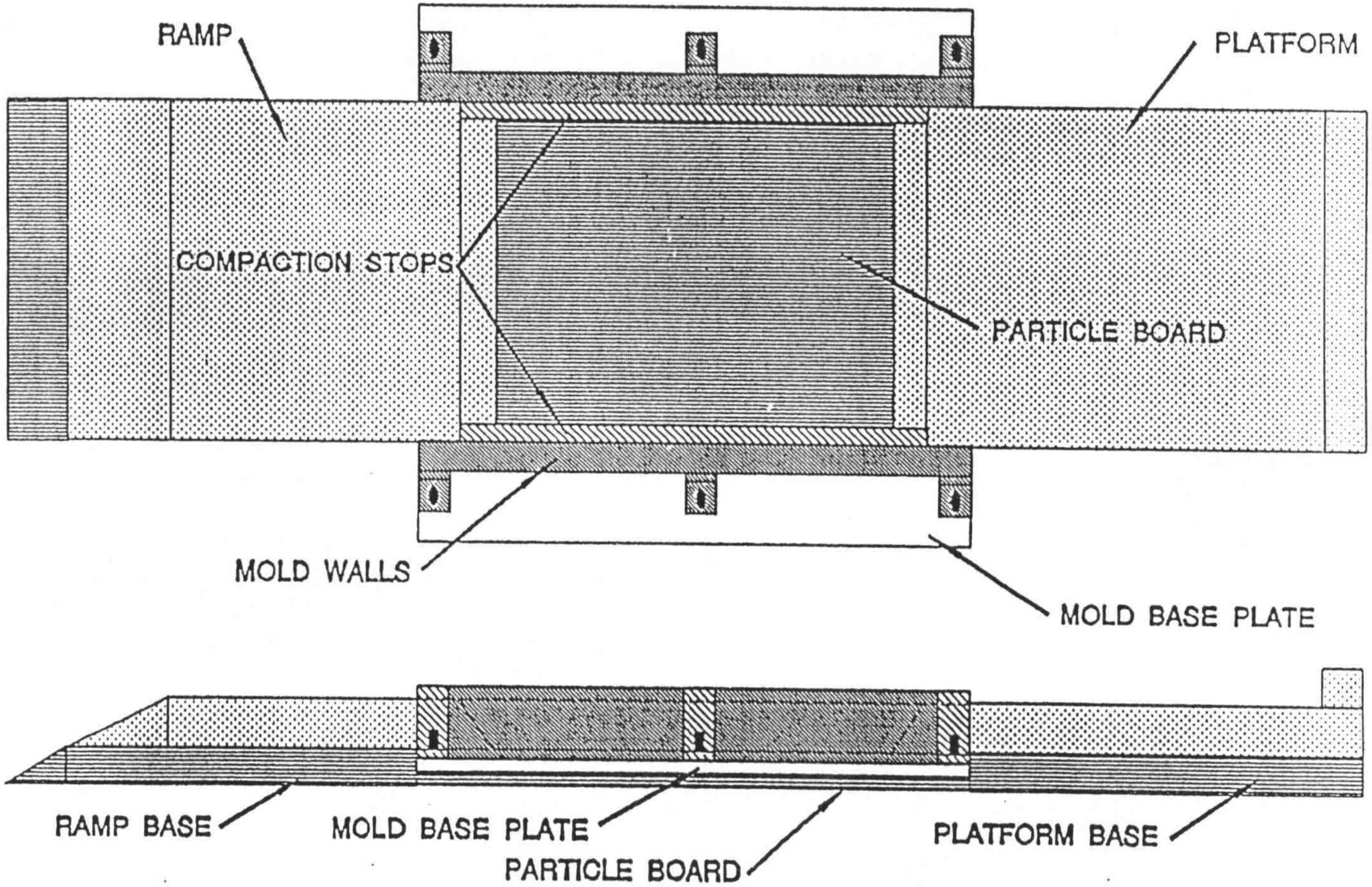

Figure A.2 Schematic of Mold for Slab

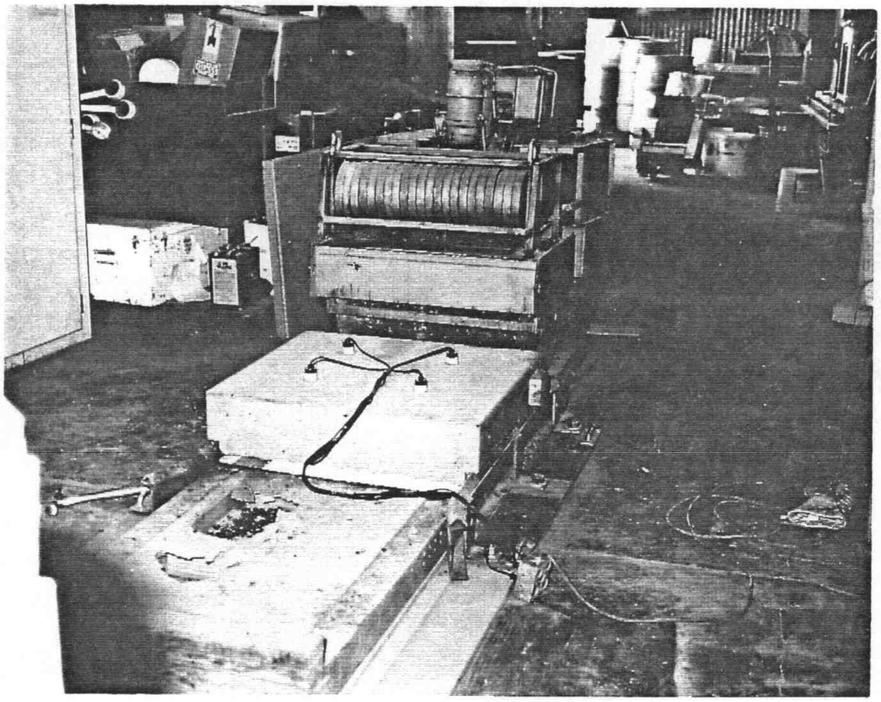

Figure A.3 Preheating the Mold

APPENDIX B

Standard Method of Test for

Short- and Long-Term Aging of Bituminous Mixes

Standard Method of Test for

Short- and Long-Term Aging of Bituminous Mixes

SHRP Designation: M-007[2]

1. SCOPE

1.1 This method describes the short- and long-term aging procedures for compacted and uncompacted bituminous mixtures. Two types of aging are described: 1) short-term aging of uncompacted mixtures to simulate the precompaction phase of the construction phase, and 2) long-term aging of compacted mixtures to simulate the aging that occurs over the service life of a pavement. The long-term aging procedures should be preceded by the short-term aging procedure. Evaluation of the extent of aging should be performed using a resilient modulus test (ASTM D 4123-82), dynamic modulus test (ASTM D 3497-79) or other approved test.

1.2 *This standard may involve hazardous materials, operations and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

1.3 The values stated in SI units are to be regarded as the standard. The values in parentheses are for information only.

2. REFERENCED DOCUMENTS

2.1 *AASHTO Documents:*

MP1	Test Method for Performance-Graded Asphalt Binder
R 11	Practice for Indicating Which Places of Figures are to be Considered Significant in Specifying Limiting Values
T2	Methods of Sampling Stone, Slag, Gravel, Sand, and Stone Block for Use as Highway Materials

[2]This standard is based on SHRP Products 1025 and 1030.

T27	Method for Sieve Analysis of Fine and Coarse Aggregates
T40	Method of Sampling Bituminous Materials
T164	Methods of Test for Quantitative Extraction of Bitumen from Bituminous Paving Material
T168	Methods of Sampling Bituminous Paving Mixtures
T201	Method of Test for Kinematic Viscosity of Asphalts
T269	Method for Percent Air Voids in Compacted Dense and Open Bituminous Paving Mixtures
M-002	Preparation of Compacted Specimens of Modified and Unmodified Hot Mix Asphalt by Means of the SHRP Gyratory Compactor
M-008	Preparation of Test Specimens of Bituminous Mixtures by Means of Rolling Wheel Compaction

2.2 *ASTM Documents:*

D 8	Standard Definitions of Terms Relating to Materials for Roads and Pavements
D 3497	Standard Test Methods for Dynamic Modulus of Asphalt Mixtures
D 3549	Method for Thickness or Height of Compacted Bituminous Paving Mixture Specimens
D 4123	Method for Indirect Tension Test for Resilient Modulus of Bituminous Mixes
E 1	Specification for Thermometers

3. TERMINOLOGY

3.1 *Desired Mixing Temperature*—the target temperature for mixing asphalt binder and aggregate in the laboratory. The desired mixing selected should be equivalent to the anticipated field plant mixing temperature. If field mixing temperatures are unknown, select a temperature which corresponds to a kinematic viscosity of 170 ± 20 mm^2/s for the asphalt binder.

3.2 *Desired Mixing Temperature*—the target temperature for mixing asphalt binder and aggregate in the laboratory. The desired mixing temperature should be equivalent to the anticipated field plant mixing temperature. If field mixing temperatures are unknown, select a temperature which corresponds to a kinematic viscosity of 170 ± 20 mm^2/s for the asphalt binder which is used.

3.3 Definitions for many terms common to asphalt are found in the following documents:

3.3.1 ASTM D 8 Standard Definitions

3.3.2 AASHTO MP1 Performance-Graded Asphalt Binder

3.3.3 AASHTO T201 Kinematic Viscosity of Asphalts

4. SUMMARY OF PRACTICE

4.1 For short-term aging, a mixture of aggregate and asphalt binder is aged in a forced draft oven for 4 hours at 135°C. The oven aging is designed to simulate the aging the mixture would undergo during plant mixing and construction.

4.2 For long-term aging, a compacted mixture of aggregate and asphalt binder is aged in a forced draft oven for 5 days at 85°C. The oven aging is designed to simulate the total aging that the compacted mixture will undergo during 7 to 10 years of service.

5. SIGNIFICANCE AND USE

5.1 The short-term aging practice simulates the aging that asphalt concrete mixtures undergo during field plant mixing operations. The long-term aging practice simulates the in-service aging of asphalt concrete mixtures after field placement and compaction.

5.2 The properties and performance of asphalt concrete mixtures may be more accurately predicted by using aged test samples.

6. APPARATUS

6.1 *Aging Test System*—A system that consists of a forced draft oven which possesses the requirements specified in table B.1.

Table B.1 Minimum Aging Test System Requirements

	Range (°C)	Resolution (°C)	Accuracy (°C)
Temperature Measurement	10–260	<1	±1
Temperature Control	25–250	<0.1	±0.1

6.2 *Oven*—Any oven which is thermostatically controlled and capable of being set to maintain any desired temperature from room temperature to 160°C. The oven shall be used for heating aggregates, asphalt binders, or laboratory equipment.

6.3 *Mixing Apparatus*—Any type of mechanical mixer that: 1) can be maintained at the required mixing temperatures; 2) will provide a well-coated, homogenous mixture of the required amount of asphalt concrete in the allowable time; and 3) allows essentially all of the mixture to be recovered.

6.4 *Miscellaneous Apparatus*

6.4.1 One metal oven pan for heating aggregates

6.4.2 One shallow metal oven pan for heating uncompacted asphalt concrete mixtures

6.4.3 Thermometers that have a range of 50 to 260°C and conform to the requirements prescribed in ASTM Document E 1

6.4.4 One metal spatula or spoon

6.4.5 Oven gloves

7. HAZARDS

7.1 This test method involves the handling of hot asphalt binder, aggregate, and asphalt concrete mixtures. These materials can cause severe burns if allowed to contact skin. Proper precautions must be taken to avoid burns.

8. SAMPLING

8.1 The asphalt binder shall be sampled in accordance with T40.

8.2 The aggregate shall be sampled and tested in accordance with T2 and T27, respectively.

9. SPECIMEN PREPARATION

9.1 Preheat the aggregate for a minimum of 2 h at the desired mixing temperature. The amount of aggregate preheated shall be of sufficient size to obtain a mixture specimen of the desired size.

9.2 Preheat the asphalt binder to the desired mixing temperature. The amount of asphalt binder preheated shall be of sufficient size to obtain the desired asphalt binder content to be tested.

NOTE 1.—Asphalt binders held for more than 2 h at the desired mixing temperature should be discarded.

9.3 Mix the heated aggregate and asphalt binder at the desired asphalt content.

10. PROCEDURE

10.1 Place the mixture on the baking pan and spread it to an even thickness of approximately 21 to 22 kg/m^2. Place the mixture and pan in the forced draft oven for 4 h ± 5 min at a temperature of 135°C ± 1°C.

10.2 Stir the mixture every hour to maintain uniform aging.

10.3 After 4 h, remove the mixture from the forced draft oven. The aged mixture is now ready for further conditioning or testing as required. Proceed to section 11 if the specimens are *not* conditioned for the effects of *long-term* aging.

10.4 *Sampling*

10.4.1 Plant-mixed asphalt concrete mixtures shall be sampled in accordance with T164.

10.4.2 Laboratory-mixed asphalt concrete mixtures shall be sampled, prepared and aged in accordance with T164.

10.4.3 Compacted roadway samples shall have a cut test specimen size that is 102 ± 6 mm in diameter by 152 ± 6 mm in height.

10.5 Heat the asphalt concrete to the desired compaction temperature.

10.6 Compact the sample in accordance with M-002 or M-008.

NOTE 2.—Compact a sufficient amount of material to ensure that the final test specimen size is 102 ± 6 mm in diameter by 152 ± 6 mm in height.

10.7 Cool the compacted test specimen to 60°C ± 1°C in an oven set at 60°C.

NOTE 3.—Cooling to 60°C will take approximately 2 h for the test specimen size stated in note 2.

10.8 After cooling the test specimen to 60°C, level the specimen ends by applying a static load to the specimen at a rate of 72.00 ± .05 kN/min. Release the load at the same rate when the specimen ends are level or when the load applied reaches a maximum of 56 kN.

10.9 After cooling the test specimen at room temperature overnight, extrude the specimen from the compaction mold.

10.10 Place the compacted test specimen on a rack in the forced draft oven for 120 ± 0.5 h at a temperature of 85°C ± 1°C.

10.11 After 120 h, turn the oven off, open the doors, and allow the test specimen to cool to room temperature. Do not touch or remove the specimen until it has cooled to room temperature.

NOTE 4.—Cooling to room temperature will take approximately overnight for the test specimen size stated in note 21.

10.12 After cooling to room temperature, remove the test specimen from the oven. The aged specimen is now ready for testing as required.

11. REPORT

11.1 Report the following information:

11.1.1 *Asphalt Binder Grade*

11.1.2 *Asphalt Binder Content*—in percent to the nearest 0.1%

11.1.3 *Aggregate Type and Gradation*

11.1.4 *Short-Term Aging Conditions*—the following information as applicable:

11.1.4.1 *Plant-Mixing Temperature*—in degrees Celsius to the nearest 1°C

11.1.4.2 *Laboratory-Mixing Temperature*—in degrees Celsius to the nearest 1°C

11.1.4.3 *Short-Term Aging Temperature in Laboratory*—in degrees Celsius to the nearest 1°C

11.1.4.4 *Short-Term Aging Duration in Laboratory*—in minutes to the nearest 1 min

11.1.5 *Long-Term Aging Conditions*

11.1.5.1 *Compaction Temperature*—in degrees Celsius to the nearest 1°C

11.1.5.2 *Compacted Specimen Height*—in millimeters to the nearest 1 mm

11.1.5.3 *Compacted Specimen Diameter*—in millimeters to the nearest 1 mm

11.1.5.4 *Compacted Specimen Density*—in kilograms per square meter to the nearest 1 kg/m^2

11.1.5.5 *Compacted Specimen Air Voids*—in percent to the nearest 0.1%

11.1.5.6 *Long-Term Aging Temperature*—in degrees Celsius to the nearest 1°C

11.1.5.7 *Long-Term Aging Duration*—in minutes to the nearest 1 min

12. KEY WORDS

12.1 Aging, asphalt concrete, asphalt concrete aging, bituminous mixtures, bituminous paving mixtures, short-term aging.

APPENDIX C

Test Data*

- Table C.1 Flexural Beam Fatigue Test--Controlled Strain (FBFT-CS) Results
- Table C.2 Repetitive Shear Strain Test--Constant Height (RSST-CH) Results
- Table C.3 Rutting Resistance Test (OSU Wheel Tracker) Results
- Table C.4 Thermal Stress Restrained Specimen Test (TSRST) Results
- Table C.5 Long Term Oven Aging (LTOA) Results
- Table C.6 Environmental Conditioning System (ECS) Results

*Note: The data presented in this Appendix is not converted to SI units.
The values are presented in the original units reported.

Table C.1 Flexural Beam Fatigue Test--Controlled Strain (FBFT-CS) Results

Mix Type	Sample ID	Asphalt Type	Air Voids	Dynamic Modulus		Accumulated Energy	Mean Strain	Cycles to Failure	Regression Equation ln(Nf)=ln(C)+P·ln(strain)		
				N50 cycles	Nf cycles						
			(%)	(psi)	(psi)	(psi)		Nf (cycles)	C	P	R^2 %
Class 'A'	CWC-C3-A	AR 4000W	6.9	707,109.4	353,554.7	2,992.05	3.95E-04	89,999.2			
	CWC-C3-B		6.0	745,501.6	372,750.8	3,467.86	2.48E-04	222,143.7			
	CWC-C3-C		6.3	679,245.5	339,622.8	1,822.82	3.96E-04	45,387.0			
	CWC-C3-E		5.5	827,440.1	413,720.0	56,241.03	1.99E-04	600,000.0			
	CWC-C3-F		6.0	738,999.6	369,499.8	1,659.67	3.96E-04	41,897.7			
	CWC-C3-G		5.8	782,865.2	391,432.6	5,663.14	2.48E-04	407,316.5			
	Average		6.1	746,860.2	373,430.1	11,974.43		Constants	-5.0164	-0.2599	91.6
	Standard Deviation		0.5	52,924.5	26,462.2	21,734.11		Standard Error	0.4682	0.0392	
PlusRide II Base	CWC-4PB1-A	AR 4000W	3.3	408,693.4	204,346.7	3,146.38	1.02E-03	21,719.7			
	CWC-4PB1-B		3.7	400,656.2	200,328.1	9,484.58	7.50E-04	135,381.5			
	CWC-4PB1-C		3.4	401,727.6	200,863.8	25,010.32	5.94E-04	600,000.0			
	CWC-4PB1-A1		2.5	460,111.2	230,055.6	13,423.05	5.91E-04	244,742.5			
	CWC-4PB1-B1		2.9	420,550.7	210,275.4	3,543.88	1.01E-03	24,459.9			
	CWC-4PB1-C1		2.4	399,594.5	199,797.3	2,679.39	1.02E-03	19,234.2			
	Average		3.0	415,222.3	207,611.2	9,547.93		Constants	-5.1171	-0.1783	95.5
	Standard Deviation		0.5	23,343.3	11,671.6	8,698.41		Standard Error	0.2191	0.0193	
PlusRide II Surface	CWC-4PS1-A	AR 4000W	4.6	392,243.7	196,121.8	6,718.50	7.50E-04	81,996.6			
	CWC-4PS1-B		4.5	374,886.3	187,443.1	13,676.13	4.94E-04	444,652.2			
	CWC-4PS1-C		3.5	359,264.3	179,632.2	2,430.49	1.02E-03	18,452.1			
	CWC-4PS1-D		3.9	425,275.0	212,637.5	2,810.87	1.01E-03	18,055.3			
	CWC-4PS1-E		3.9	369,497.0	184,748.5	2,230.02	1.02E-03	17,107.0			
	CWC-4PS1-F		3.8	391,468.2	195,734.1	10,056.98	4.96E-04	300,000.0			
	Average		4.0	384,233.3	192,116.6	5,573.20		Constants	-4.5803	-0.2354	98.9
	Standard Deviation		0.4	25,867.8	12,933.9	4,890.12		Standard Error	0.1403	0.0126	
ARHM-GG Surface	CWC-WS1-A	AR 2000	6.0	No Failure	No Failure	No Failure	No Failure	No Failure			
	CWC-WS1-B		6.3	353,422.7	176,711.4	4,045.98	6.52E-04	87,836.4			
	CWC-WS1-C		6.5	213,498.6	Damaged	Damaged	Damaged	Damaged			
	CWC-WS1-D		5.9	342,403.8	171,201.9	6,242.05	4.95E-04	249,939.7			
	CWC-WS1-E		5.5	299,989.6	149,994.8	4,815.69	6.53E-04	125,000.0			
	CWC-WS1-F		5.6	345,450.0	172,725.0	5,471.53	4.73E-04	251,153.9			
	Average		6.0	259,127.5	111,772.2	3,429.21		Constants	-3.6839	-0.3168	91.3
	Standard Deviation		0.4	137,242.0	87,076.1	2,753.36		Standard Error	2.2986	0.0693	

Table C.2 Repetitive Shear Strain Test -- Constant Height (RSST-CH) Results

Mix Type	Sample ID	Asphalt Type	Air Voids (Parafilm) (%)	Air Voids (SSD) (%)	Test Temperature (C)	Permanent Shear Strain (μ Strain)	Permanent Shear Strain Cycles	Notes
Class 'A'	CWC-C3-S1	AR 4000W	7.1	6.3	40	34,052	5,502	Note: lower temperature
	CWC-C3-S2		6.1	4.7	50	50,260	602	
	CWC-C3-S3		6.8	5.3	50	52,079	1,002	
	Average		6.7	5.4		51,170	802	Average of permanent shear strains at 50 C
	Standard Deviation		0.5	0.8		1,286	283	Standard deviation of permanent shear strains at 50 C
PlusRide II Base	CWC-4PB1-S1	AR 4000W	2.6	1.3	50	48,737	202	
	CWC-4PB1-S2		2.3	1.2	50	52,033	202	
	CWC-4PB1-S3		3.3	1.3	50	54,170	402	
	Average		2.7	1.3		51,647	269	
	Standard Deviation		0.5	0.1		2,737	115	
PlusRide II Surface	CWC-4PS1-S1	AR 4000W	1.6	0.3	50	53,511	302	
	CWC-4PS1-S2		2.7	1.2	40	51,442	3,000	Note: lower temperature
	CWC-4PS1-S3		2.8	0.9	50	51,124	202	
	Average		2.4	0.8		52,318	252	Average of permanent shear strains at 50 C
	Standard Deviation		0.7	0.5		1,688	71	Standard deviation of permanent shear strains at 50 C
ARHM-GG Surface	CWC-WS1-S1	AR 2000	6.5	5.4	50	49,510	10,002	
	CWC-WS1-S2		6.3	5.1	50	46,464	23,002	
	CWC-WS1-S3		6.5	5.4	50	48,191	20,002	
	Average		6.4	5.3		48,055	17,669	
	Standard Deviation		0.1	0.2		1,528	6,807	

Table C.3 Rutting Resistance Test (OSU Wheel Tracker) Results

Mix Type	Specimen ID	Asphalt Type	Air Voids Parafilm (%)	Air Voids SSD (%)	WHEEL PASS RUTDEPTH 0 (in)	100 (in)	200 (in)	500 (in)	1,000 (in)	2,000 (in)	5,000 (in)	10,000 (in)	20,000 (in)	30,000 (in)	40,000 (in)	50,000 (in)
Class 'A'	CWC-C2-R1	AR 4000W	7.5	6.2	0	0.0037	0.0185	0.0234	0.0342	0.0545	0.0703	0.0938	0.1155	0.1276	0.1460	0.1487
	CWC-C2-R2		6.3	4.8	0	0.0204	0.0230	0.0363	0.0550	0.0593	0.0791	0.1010	0.1128	0.1329	0.1393	0.1519
	Average		6.9	5.5	0	0.0120	0.0207	0.0298	0.0446	0.0569	0.0747	0.0974	0.1142	0.1302	0.1426	0.1503
	Standard Deviation		0.6	0.7	0	0.0083	0.0022	0.0064	0.0104	0.0024	0.0044	0.0036	0.0014	0.0026	0.0034	0.0016
PlusRide II Base	CWC-PB3-R1	AC 5	4.6	1.5	0	0.1293	0.2031	0.3305	0.4878							
	CWC-PB3-R2		3.0	1.4	0	0.1056	0.1680	0.2990	0.4313							
	Average		3.8	1.4	0	0.1175	0.1856	0.3148	0.4596							
	Standard Deviation		0.8	0.1	0	0.0118	0.0176	0.0158	0.0283							
PlusRide II Base	CWC-4PB2-R1	AR 4000W	5.2	3.1	0	0.0792	0.1405	0.2348	0.3398	0.4794						
	CWC-4PB2-R2		5.3	3.6	0	0.0825	0.1330	0.2120	0.4087	0.3946						
	Average		5.2	3.3	0	0.0809	0.1367	0.2234	0.3742	0.4370						
	Standard Deviation		0.0	0.2	0	0.0016	0.0037	0.0114	0.0344	0.0424						
PlusRide II Surface	CWC-PS1-R1	AC 5	3.6	2.0	0	0.1182	0.1771	0.2986	0.4488							
	CWC-PS1-R2		4.2	2.5	0	0.0964	0.1569	0.2501	0.3537							
	Average		3.9	2.2	0	0.1073	0.1670	0.2744	0.4012							
	Standard Deviation		0.3	0.3	0	0.0109	0.0101	0.0242	0.0476							
PlusRide II Surface	CWC-4PS2-R1	AR 4000W	5.0	Not Tested	0	0.0498	0.0881	0.1237	0.1973	0.2297	0.3107	0.4075	0.5515			
	CWC-4PS2-R2		5.1	Not Tested	0	0.0625	0.0979	0.1508	0.2306	0.2921	0.4177	0.8150				
	Average		5.1	---	0	0.0561	0.0930	0.1372	0.2139	0.2609	0.3642	0.6112	0.5515			
	Standard Deviation		0.1	---	0	0.0063	0.0049	0.0136	0.0166	0.0312	0.0535	0.2037	0.0000			
ARHM-GG Surface	CWC-WS2-R1	AR 2000	8.6	6.4	0	0.0289	0.0312	0.0289	0.0501	0.0620	0.0847	0.0891	0.1079	0.1131	0.1216	0.1403
	CWC-WS2-R2		8.2	5.8	0	0.0269	0.0199	0.0244	0.0463	0.0592	0.0886	0.1066	0.1303	0.1367	0.1468	0.1546
	Average		8.4	6.1	0	0.0279	0.0256	0.0266	0.0482	0.0606	0.0866	0.0978	0.1191	0.1249	0.1342	0.1475
	Standard Deviation		0.2	0.3	0	0.0010	0.0057	0.0023	0.0019	0.0014	0.0020	0.0088	0.0112	0.0118	0.0126	0.0072

Table C.4 Thermal Stress Restrained Specimen Test (TSRST) Results

Mix Type	Specimen ID	Asphalt Type	Air Voids (%)	Cross Sectional Area (in²)	Number of Data Points (ea)	Cooling Rate (°C/hr)	Cooling Rate R^2	Fracture Stress (psi)	Fracture Temperature (°C)	First ds/dt (psi/°C)	First ds/dt R^2	Second ds/dt (psi/°C)	Second ds/dt R^2	Tangent Transition Temperature (°C)	Bisector Transition Temperature (°C)	Notes
Class 'A'	CWC-C2-T1	AR 4000W	5.6	3.97	354	-9.90	1.000	484	-25.9	-4.07	0.943	-33.98	0.990	-12.9	-16.9	
	CWC-C2-T2		5.2	3.97	345	-10.26	1.000	522	-25.9	-4.20	0.953	-34.75	0.998	-12.5	-16.5	
	CWC-C2-T3		4.0	3.97	365	-10.56	1.000	588	-26.9	-4.04	0.954	-36.86	0.993	-12.4	-16.7	
	Average		4.9	3.97	355	-10.24		531	-26.2	-4.10		-35.20		-12.6	-16.7	
	Standard Deviation		0.8	0.00	10	0.33		53	0.6	0.09		1.49		0.3	0.2	
PlusRide II Base	CWC-PB1-T1	AC 5	-0.3	3.97	444	-10.35	1.000	604	-33.2	-2.96	0.958	-32.67	0.973	-15.3	-20.1	
	CWC-PB1-T2		0.3	3.97	443	-9.41	1.000	595	-32.7	-3.31	0.965	-32.25	0.991	-15.9	-21.9	
	CWC-PB1-T3		1.0	3.97	418	-10.18	1.000	787	-13.9	-2.93	0.965	0.02	0.088	2.9	-14.8	Premature fracture due to nonhomogeneous sample.
	Average		0.0	3.97	444	-9.88		600	-33.0	-3.14		-32.46		-15.6	-21.0	CWC-PB1-T3 not included in average.
	Standard Deviation		0.4	0.00	1	0.66		6	0.4	0.25		0.30		0.4	1.3	CWC-PB1-T3 not included in standard deviation
PlusRide II Base	CWC-PB3-T1	AC 5	2.5	3.97	472	-10.44	1.000	498	-32.4	-2.09	0.936	-27.71	0.985	-14.3	-18.0	Air voids measured after TSRST. Previously at 1.5%.
	CWC-PB3-T2		3.1	3.97	429	-9.97	1.000	452	-32.4	-1.71	0.951	-27.10	0.997	-15.7	-20.4	
	CWC-PB3-T3		2.8	3.97	461	-10.02	1.000	476	-34.3	-2.10	0.949	-26.78	0.878	-17.1	-22.1	
	Average		2.8	3.97	454	-10.14		475	-33.0	-1.97		-27.20		-15.7	-20.2	
	Standard Deviation		0.3	0.00	22	0.26		23	1.1	0.22		0.47		1.4	2.1	
PlusRide II Base	CWC-4PB2-T1	AR 4000W	3.2	3.97	359	-10.51	1.000	374	-27.2	-2.72	0.950	-24.62	0.997	-13.2	-17.2	Air voids measured after TSRST. Previously at -2.7%.
	CWC-4PB2-T2		3.7	3.97	333	-10.02	1.000	337	-24.5	-3.06	0.950	-24.73	0.998	-12.4	-16.3	Air voids measured after TSRST. Previously at 1.8%.
	CWC-4PB2-T3		2.9	3.97	368	-10.65	0.999	407	-28.3	-2.58	0.928	-26.56	0.999	-13.4	-17.9	Air voids measured after TSRST. Previously at 0.1%.
	Average		3.3	3.97	353	-10.39		373	-26.7	-2.79		-25.30		-13.0	-17.1	
	Standard Deviation		0.4	0.00	18	0.33		35	2.0	0.25		1.09		0.5	0.8	
PlusRide II Surface	CWC-PS1-T1	AC 5	2.5	3.97	410	-10.20	1.000	417	-30.5	-1.94	0.942	-23.39	0.973	-14.4	-18.5	
	CWC-PS1-T2		1.1	3.97	439	-10.26	0.999	512	-32.4	-2.29	0.946	-28.59	0.972	-15.1	-19.7	
	CWC-PS1-T3		1.3	3.97	426	-10.28	0.999	467	-31.8	-2.02	0.928	-26.71	0.980	-15.4	-20.4	
	CWC-PS1-T4		1.9	3.97	419	-9.98	1.000	509	-32.9	-2.77	0.963	-28.49	0.981	-15.5	-19.8	
	Average		1.7	3.97	424	-10.18		476	-31.9	-2.26		-26.80		-15.1	-19.6	
	Standard Deviation		0.6	0.00	12	0.14		45	1.0	0.37		2.43		0.5	0.8	
PlsuRide II Surface	CWC-4PS3-T1	AR 4000W	2.5	3.97	372	-9.71	1.000	388	-27.6	-2.54	0.958	-26.83	0.992	-13.5	-17.7	Air voids measured after TSRST. Previously at 6.7%.
	CWC-4PS3-T2		2.8	3.97	373	-10.44	0.999	414	-28.5	-2.44	0.934	-26.50	0.990	-13.5	-18.1	
	CWC-4PS3-T3		2.6	3.97	391	-9.99	1.000	385	-26.9	-2.78	0.946	-26.28	0.997	-13.5	-17.8	Air voids measured after TSRST. Previously at 4.9%.
	CWC-4PS3-T4		2.0	3.97	420	-9.77	1.000	437	-27.2	-2.39	0.951	-28.82	0.971	-13.3	-17.8	
	Average		2.5	3.97	389	-9.98		406	-27.6	-2.54		-27.11		-13.5	-17.9	
	Standard Deviation		0.3	0.00	22	0.33		24	0.7	0.17		1.16		0.1	0.2	
ARHM-GG Surface	CWC-WS2-T1	AR 2000	4.8	3.97	445	-10.21	1.000	502	-33.0	-3.52	0.979	-25.27	0.994	-14.9	-20.9	
	CWC-WS2-T2		5.7	3.97	419	-11.26	0.999	359	-30.3	-3.76	0.986	-18.12	0.996	-12.9	-16.8	
	CWC-WS2-T3		4.6	3.97	418	-10.31	0.999	484	-31.7	-4.52	0.984	-25.54	0.995	-15.5	-20.8	
	CWC-WS2-T4		4.6	3.97	428	-10.59	0.999	507	-34.1	-4.17	0.981	-26.23	0.999	-15.8	-21.6	
	Average		4.9	3.97	428	-10.59		463	-32.3	-3.99		-23.79		-14.8	-20.0	
	Standard Deviation		0.5	0.00	13	0.47		70	1.6	0.44		3.80		1.3	2.2	

Table C.5 Long-Term Oven Aging (LTOA) Results

Mix Type	Specimen ID	Asphalt Type	Air Voids (%)	Initial MTS Mr (ksi)	Final MTS Mr (ksi)	Mr Ratio	Notes
Class 'A'	CWC-C2-L1	AR 4000W	6.9	Damaged	Damaged	--	
	CWC-C2-L2		6.4	Damaged	Damaged	--	
	CWC-C2-L3		8.6	369	634	1.7	
	CWC-C2-L4		7.3	Not Tested	Not Tested	--	
	CWC-C2-L5		8.2	315	622	2.0	
	CWC-C2-L6		8.0	302	544	1.8	
	Average		8.3	329	600	1.8	Average of three samples tested.
	Standard Deviation		0.3	36	49	0.1	Standard deviation of three samples tested.
PlusRide II Base	CWC-PB1-L1	AC 5	1.2	Damaged	Damaged	--	
	CWC-PB1-L2		2.1	143	189	1.3	
	CWC-PB1-L3		0.6	168	213	1.3	
	CWC-PB1-L4		1.7	159	177	1.1	
	Average		1.5	157	193	1.2	Average of three samples tested.
	Standard Deviation		0.8	13	18	0.1	Standard deviation of three samples tested.
PlusRide II Base	CWC-PB3-L1	AC 5	4.4	96	108	1.1	
	CWC-PB3-L2		3.8	93	126	1.4	
	CWC-PB3-L3		3.3	105	132	1.3	
	Average		3.8	98	122	1.2	
	Standard Deviation		0.6	6	13	0.1	
PlusRide II Base	CWC-4PB2-L1	AR 4000W	4.6	184	224	1.2	
	CWC-4PB2-L2		4.9	165	225	1.4	
	CWC-4PB2-L3		4.7	172	179	1.0	
	Average		4.7	174	209	1.2	
	Standard Deviation		0.2	10	26	0.2	
PlusRide II Surface	CWC-PS1-L1	AC 5	3.3	101	139	1.4	
	CWC-PS1-L2		3.8	116	138	1.2	
	CWC-PS1-L3		3.3	119	164	1.4	
	Average		3.5	112	147	1.3	
	Standard Deviation		0.3	9	15	0.1	
PlusRide II Surface	CWC-4PS2-L1	AR 4000W	5.3	195	228	1.2	
	CWC-4PS2-L2		4.9	192	234	1.2	
	CWC-4PS2-L3		4.9	177	198	1.1	
	Average		5.0	188	220	1.2	
	Standard Deviation		0.2	10	19	0.1	
ARHM-GG Surface	CWC-WS2-L1	AR 2000	7.0	224	275	1.2	
	CWC-WS2-L2		7.3	250	299	1.2	
	CWC-WS2-L3		8.7	245	295	1.2	
	Average		7.7	240	289	1.2	
	Standard Deviation		0.9	14	13	0.0	

Table C.6 Environmental Conditioning System (ECS) Results

Mix Type	Specimen ID	Asphalt Type	Asphalt Content (%)	Air Voids (%)	Air Perm. (in/sec)	MTS Mr (ksi)	Condition Cycle (number)	ECS Stress (psi)	ECS Strain (μstrain)	ECS Mr (ksi)	Mr Ratio	Water Perm. (in/sec)	Visual Stripping (%)	Binder Migration (%)	Notes
Class 'A'	CWC-C2-E1	AR 4000W	4.7	6.4	1.59E-02	364	0	52.6	100.4	524	1.0	8.82E-06	5	1 - 10	Standard ECS test
							1	51.7	100.1	517	1.0	3.62E-05			
							2	56.3	100.1	563	1.1	3.85E-05			
							3	49.2	100.0	493	0.9	3.04E-05			
							4	67.2	100.4	669	1.3	2.87E-05			
	CWC-C2-E2		4.7	5.9	1.09E-02	375	0	59.4	100.2	593.2	1.0	4.53E-06	5	10 - 20	Standard ECS test
							1	49.4	100.5	491.9	0.8	2.73E-05			
							2	53.4	100.6	530.9	0.9	2.08E-05			
							3	34.1	100.3	339.9	0.6	1.21E-05			
							4	53.7	100.7	533.3	0.9	1.32E-05			
	CWC-C2-E3		4.7	6.4	5.48E-02	350	0	37.9	99.8	380.0	1.0	1.43E-05	5	< 1 - 10	Sandard ECS test
							1	53.3	99.9	533.9	1.4	3.30E-05			
							2	49.2	101.0	487.1	1.3	2.91E-05			
							3	43.3	100.6	430.5	1.1	2.08E-05			
							4	58.3	100.2	581.7	1.5	2.78E-05			
PlusRide II Base	CWC-PB1-E1	AC 5	7.5	1.8	0.00E+00	146	0	25.5	100.5	253.8	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading
							1	26.6	101.3	262.5	1.0	0.00E+00			
							2	26.7	100.6	265.5	1.0	0.00E+00			
							3	25.4	100.7	251.7	1.0	0.00E+00			
							4	27.0	100.8	268.4	1.1	0.00E+00			
	CWC-PB1-E2		7.5	0.8	0.00E+00	152	0	28.9	101.1	285.7	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading
							1	30.4	100.0	304.0	1.1	0.00E+00			
							2	26.8	99.6	269.0	0.9	0.00E+00			
							3	32.1	101.1	317.3	1.1	0.00E+00			
							4	32.3	101.8	317.5	1.1	0.00E+00			
	CWC-PB1-E3		7.5	2.0	0.00E+00	148	0	37.8	100.6	375.7	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading
							1	24.8	99.8	248.9	0.7	0.00E+00			
							2	26.9	100.8	266.7	0.7	0.00E+00			
							3	26.2	99.8	262.5	0.7	0.00E+00			
							4	26.1	100.2	260.9	0.7	0.00E+00			

Table C.6 Environmental Conditioning System (ECS) Results, Continued

Mix Type	Specimen ID	Asphalt Type	Asphalt Content (%)	Air Voids (%)	Air Perm. (in/sec)	MTS Mr (ksi)	Condition Cycle (number)	ECS Stress (psi)	ECS Strain (μstrain)	ECS Mr (ksi)	Mr Ratio	Water Perm. (in/sec)	Visual Stripping (%)	Binder Migration (%)	Notes
PlusRide II Base	CWC-PB3-E1	AC 5	7.5	3.4	0.00E+00	102	0	16.2	100.9	161.1	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading.
							1	16.3	101.4	160.6	1.0	0.00E+00			
							2	20.6	100.3	205.4	1.3	0.00E+00			
							3	16.7	99.9	166.5	1.0	0.00E+00			
							4	17.4	100.4	172.9	1.1	0.00E+00			
	CWC-PB3-E2		7.5	3.9	0.00E+00	88	0	18.4	100.1	184.0	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading.
							1	21.2	101.2	209.6	1.1	0.00E+00			
							2	23.0	100.4	228.7	1.2	0.00E+00			
							3	18.6	100.7	184.2	1.0	0.00E+00			
							4	25.7	100.6	254.9	1.4	0.00E+00			
	CWC-PB3-E3		7.5	3.6	0.00E+00	83	0	16.4	100.8	163.0	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading.
							1	16.4	99.7	164.2	1.0	0.00E+00			
							2	17.3	100.2	172.3	1.1	0.00E+00			
							3	15.7	100.3	156.1	1.0	0.00E+00			
							4	17.4	100.3	173.4	1.1	0.00E+00			
PlusRide II Base	CWC-4PB2-E1	AR 4000W	7.5	5.3	0.00E+00	180	0	21.0	100.4	208.8	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading.
							1	23.1	100.3	230.0	1.1	0.00E+00			
							2	23.2	100.1	231.7	1.1	0.00E+00			
							3	22.5	100.0	224.6	1.1	0.00E+00			
							4	30.8	101.5	303.6	1.5	0.00E+00			
	CWC-4PB2-E2		7.5	5.9	0.00E+00	171	0	28.5	100.8	282.9	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading.
							1	27.8	101.0	275.9	1.0	0.00E+00			
							2	29.2	100.4	290.6	1.0	0.00E+00			
							3	26.6	101.4	262.4	0.9	0.00E+00			
							4	44.2	100.3	441.0	1.6	0.00E+00			
	CWC-4PB2-E3		7.5	4.5	0.00E+00	193	0	33.8	100.8	334.8	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading.
							1	33.0	100.6	327.6	1.0	0.00E+00			
							2	37.9	100.8	377.3	1.1	0.00E+00			
							3	37.1	100.3	369.8	1.1	0.00E+00			
							4	38.8	100.5	386.1	1.2	0.00E+00			

Table C.6 Environmental Conditioning System (ECS) Results, Continued

Mix Type	Specimen ID	Asphalt Type	Asphalt Content (%)	Air Voids (%)	Air Perm. (in/sec)	MTS Mr (ksi)	Condition Cycle (number)	ECS Stress (psi)	ECS Strain (μstrain)	ECS Mr (ksi)	Mr Ratio	Water Perm. (in/sec)	Visual Stripping (%)	Binder Migration (%)	Notes
PlusRide II Surface	CWC-PS1-E1	AC 5	7.5	3.6	0.00E+00	103	0	26.4	100.7	262.3	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading.
							1	21.8	100.8	215.7	0.8	0.00E+00			
							2	22.9	101.4	225.4	0.9	0.00E+00			
							3	24.4	100.0	243.7	0.9	0.00E+00			
							4	26.3	100.9	260.6	1.0	0.00E+00			
	CWC-PS1-E2		7.5	3.2	0.00E+00	108	0	27.9	100.6	277.4	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading.
							1	24.7	99.9	247.8	0.9	0.00E+00			
							2	24.3	100.9	240.7	0.9	0.00E+00			
							3	25.9	100.3	258.3	0.9	0.00E+00			
							4	27.4	100.8	271.3	1.0	0.00E+00			
	CWC-PS1-E3		7.5	3.6	0.00E+00	104	0	21.9	99.5	220.4	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading.
							1	25.1	99.7	251.7	1.1	0.00E+00			
							2	26.5	100.0	265.1	1.2	0.00E+00			
							3	24.9	100.4	247.3	1.1	0.00E+00			
							4	27.1	99.5	272.3	1.2	0.00E+00			
PlusRide II Surface	CWC-4PS2-E1	AR 4000W	7.5	4.7	0.00E+00	191	0	31.7	102.1	310.5	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading.
							1	37.6	99.5	377.9	1.2	0.00E+00			
							2	44.2	100.2	441.3	1.4	0.00E+00			
							3	43.4	101.5	427.5	1.4	0.00E+00			
							4	49.0	100.6	487.6	1.6	0.00E+00			
	CWC-4PS2-E2		7.5	4.0	0.00E+00	193	0	29.4	101.3	291.1	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading.
							1	25.0	100.4	249.2	0.9	0.00E+00			
							2	29.9	100.2	298.5	1.0	0.00E+00			
							3	24.2	100.5	240.6	0.8	0.00E+00			
							4	36.9	100.8	366.0	1.3	0.00E+00			
	CWC-4PS2-E3		7.5	4.4	0.00E+00	190	0	37.1	100.6	369.2	1.0	0.00E+00	< 5	< 1 - 10	ECS test with no repeated loading.
							1	22.7	99.7	227.8	0.6	0.00E+00			
							2	30.5	100.5	303.5	0.8	0.00E+00			
							3	27.1	100.6	269.3	0.7	0.00E+00			
							4	31.1	100.8	308.3	0.8	0.00E+00			

Table C.6 Environmental Conditioning System (ECS) Results, Continued

Mix Type	Specimen ID	Asphalt Type	Asphalt Content (%)	Air Voids (%)	Air Perm. (in/sec)	MTS Mr (ksi)	Condition Cycle (number)	ECS Stress (psi)	ECS Strain (μstrain)	ECS Mr (ksi)	Mr Ratio	Water Perm. (in/sec)	Visual Stripping (%)	Binder Migration (%)	Notes
							0	42.8	100.9	424.6	1.0	0.00E+00			Standard ECS test
							1	49.6	100.2	494.6	1.2	0.00E+00			
	CWC-WS2-E1		8.0	7.2	0.00E+00	240	2	46.7	100.2	466.3	1.1	3.26E-06	< 5	1 - 10	
							3	44.7	100.9	443.1	1.0	4.06E-06			
							4	50.9	100.7	505.2	1.2	3.81E-06			
							0	37.1	100.2	370.8	1.0	0.00E+00			Standard ECS test
							1	35.9	100.7	356.5	1.0	1.44E-05			
ARHM-GG Surface	CWC-WS2-E2	AR 2000	8.0	6.6	0.00E+00	244	2	40.1	99.8	401.8	1.1	1.94E-05	< 5	1 - 10	
							3	42.7	100.5	425.2	1.1	4.76E-05			
							4	41.9	99.8	419.3	1.1	3.38E-05			
							0	28.2	100.5	280.6	1.0	0.00E+00			Standard ECS test
							1	46.3	100.6	460.3	1.6	5.12E-05			
	CWC-WS2-E3		8.0	7.4	0.00E+00	259	2	47.1	102.0	461.3	1.6	1.22E-04	< 5	10 - 20	
							3	51.4	101.4	507.3	1.8	7.48E-05			
							4	48.7	99.8	487.7	1.7	7.72E-05			