PB98-157480

MIXTURE DESIGN AND PERFORMANCE PREDICTION OF RUBBER-MODIFIED ASPHALT IN OHIO

By

Robert Y. Liang

FINAL REPORT

Prepared in Cooperation with the Ohio Department of Transportation and

the U. S. Department of Transportation, Federal Highway Administration

Department of Civil Engineering
The University of Akron
Akron, OH 44325-3905

June, 1998

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/OH-98/003		

4. Title and Subtitle	5. Report Date
MIXTURE DESIGN AND PERFORMANCE PREDICTION OF RUBBER-MODIFIED ASPHALT IN OHIO	August, 1997
	6. Performing Organization Code
	8. Performing Organization Report No.
7. Author(s) Robert Y. Liang	
	10. Work Unit No. (TRAIS)
9. Performing Organization Name and Address The University of Akron Department of Civil Engineering Akron, OH 44325	11. Contract or Grant No. State Job No. 14572(0)
	13. Type of Report and Period Covered Final Report
12. Sponsoring Agency Name and Address Ohio Department of Transportation 25 S. Front St. Columbus, OH 43215	14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration

16. Abstract The mixture design and performance characteristics of crumb rubber modified asphalt concretes were investigated in this research project to meet the requirements of the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991, which has required each State to incorporate scrap tire rubber into its asphalt paving materials. Specifically, the objectives of this research encompass the following: (i) investigation of the rheological properties of asphalt-rubber binder to determine optimum content of crumb rubber, (ii) development of optimum mix design for various applications, including both wet and dry mix processes, (iii) characterization of mechanical properties of recommended paving mixtures, including resilient modulus, fatigue cracking behavior, low-temperature thermal cracking resistance, water sensitivity test, incremental creep test and loaded wheel track test, and (iv) comparison of performance of selected paving mixes. The KENLAYER computer program developed by the University of Kentucky was used to evaluate the life performance of the selected paving mixtures.

The Marshall testing procedure, with small modifications of selection criteria, was used to determine the optimum rubber-aggregate-asphalt mix design, including five wet processes and two dry processes. In addition, the unmodified conventional hot mix was used as the control mix for a comparison purpose.

Based on the recommended mix design, mechanical properties characterizations, computer analysis results and life cycle cost analysis, it can be concluded that the rubber modified asphalt mixtures, produced by the wet process, can be a viable asphalt paving material for highway pavement in terms of resistance to fatigue, low-temperature thermal cracking and rutting. However, the durability of the rubber modified asphalt concretes in terms of TSR (tensile strength ratio) from the water stripping test has showed potential problem, needing further investigation. From limited life cycle cost analysis, crumb rubber modified asphalt concretes have not shown advantage over the conventional asphalt concrete.

17. Key Words	18. Distribution Statement
crumb rubber modified asphalt concrete, mixture design, characterization of mechanical properties, life cycle cost analysis	No Restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified		

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

MIXTURE DESIGH AND PERFORMANCE PREDICTION OF RUBBER-MODIFIED ASPHALT IN OHIO

By

Robert Y. Liang

FINAL REPORT

Prepared in Cooperation with the Ohio Department of Transportation and the U. S. Department of Transportation, Federal Highway Administration

Department of Civil Engineering
The University of Akron
Akron, OH 44325-3905

June 1998

EXECUTIVE SUMMARY

Appropriate disposal of scrap tires has been a major environmental concern over the years, mainly due to potential fire and health hazards associated with uncontrolled stockpiling. Primarily driven by this environmental concern, the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 has required each State to begin incorporating scrap tire rubber into its asphalt paving materials. Although in the revision of the original ISTEA, the mandate has been eliminated, there remains a language of encouraging the use of crumb rubbers in asphalt paving materials. Ohio Department of Transportation (ODOT) desires to develop the mix design procedure, construction practice, and performance specifications for crumb rubber modified asphalt paving materials. This research was conducted to develop the needed design and construction guidance for meeting the ODOT anticipated needs. Specially, the objectives of this research encompass the following scope: (i) investigation of the rheological properties of asphalt-rubber binder to determine optimum content of crumb rubber, (ii) development of optimum mix design for various applications, including both wet and dry mix processes, (iii) characterization of mechanical properties of recommended paving mixtures, including resilient modulus, fatigue cracking behavior, low-temperature thermal cracking resistance, water sensitivity test, incremental creep test and loaded wheel track test, and (iv) comparison of performance of selected paving mixes.

A suite of recently developed SHRP (Strategic Highway Research Program) binder testing methods were used in characterizing rubber modified asphalt binder. These tests included Brookfield rotational viscosity test and dynamic shear rheometer test. Short-term aging of

binders was accomplished by thin film oven test. Based on these test results, optimum crumb rubber contents for various applications were recommended in this report.

The Marshall testing procedure, with small modifications of selection criteria, were used to determine optimum rubber-aggregate-asphalt mix design, including (1) wet process, dense-graded with 15% of minus 30 mesh crumb rubber modifier (CRM), (2) wet process dense-graded with 10% of minus 30 mesh crumb rubber modifier (CRM), (3) dense-graded with Ecoflex, (4) wet process, dense-graded with 10% of Goodyear Ultrafine CRM, (5) wet process, gap-graded with 15% of minus 30 mesh CRM, (6) dry process, dense-graded with 2% CRM, and (7) dry process, gap-graded with 2% and 3% CRM, respectively. In addition the unmodified conventional hot mix was used as the control mix for a comparison purpose.

The mechanical properties of the recommended mixes were determined by using indirect tensile test, resilient modulus test, fatigue test on third-point bending specimens, TSRST (Thermal Stress Restrained Specimen Test), incremental creep test, loaded wheel track test, and water sensitivity test. The test results were used to determine pertinent parameters for input in the KENLAYER computer program to evaluate performances of each mix under given traffic load.

Based on the recommended mix design, mechanical properties characterizations, computer analysis results and life cycle analysis, it can be concluded that the rubber modified asphalt mixtures, produced by the wet process, can be a viable asphalt paving material for highway pavement, due to their superior resistance to fatigue, low-temperature thermal cracking and rutting. Specifically, the AC10 based, wet process, dense-graded asphalt-rubber mixture (AC10+10%WRF30) showed the best performance. However, the durability of the rubber modified asphalt concrete in terms of TSR (tensile strength ratio) from the water stripping test has potential problem, indicating the needs for further investigation.

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

LIST OF TABLES

LIST OF TABLES (Continued)

LIST OF TABLES (Continued)

LIST OF TABLES (Continued)

LIST OF TABLES (Continued)

LIST OF FIGURES

LIST OF FIGURES (Continued)

Figure	Title	Page

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

CHAPTER I

INTRODUCTION

1.1 Statement of Problem

In a recent survey (EPA, 1991), it was observed that each year about 285 million used tires were generated in the U.S. Among them, 33 million were retreaded, 22 million reused (resold), and 42 million diverted to various other alternative uses. However, the majority of them (about 188 million) were added to the stockpiles and landfills. The U.S. Environmental Protection Agency (EPA) estimates that currently 2 to 3 billion scrap tires exist (EPA, 1991).

The markets for retread, reuse, and processed tires are saturated. Of the available expanding markets for scrap tires, several have shown the potential to use a significant amount of scrap tires. Two of these are fuel for combustion and Crumb Rubber Modifier (CRM) for asphalt paving (Scrap Tire Management Council, 1990). One estimate is that CRM technology can recycle 10 million scrap tires annually as CRM, if 2 to 5 million tons of Hot Mix Asphalt (HMA) material is used.

The Intermodal Surface Transportation Efficiency Act of 1991, commonly referred to as ISTEA, initiated major changes in the Federal-aid highway program. Section 1038 contains provisions for each State to begin incorporating scrap tire rubber into its asphalt paving materials. Specifically, beginning on January 1, 1995, and annually thereafter, each state shall certify that it has satisfied the minimum utilization requirement for asphalt pavement containing recycled rubber as established by the regulation. This minimum utilization requirement begins at 5 percent for the year 1994, and increases 5 percent every year until it reaches to 20 percent in the year 1997. However, it is noted that the ISTEA of 1991 was modified later to soften the language on the mandate use of CRM in asphalt paving materials. With the elimination of the mandate on the use of CRM, it becomes clear that any research on CRM should gear toward economic benifits on CRM improved asphalt pavement performance.

To gain a better understanding of CRM modified asphalt concrete, on the national level concerted research efforts are taking place to resolve several principal issues: the ability to recycle asphalt paving mixes containing CRM, the development of standards for testing materials, and assessing the environmental impact of CRM mixtures. On the state level, the principal research issue is the establishment of mix designs and the expected cost/performance of the various types of CRM paving methods. Before state highway agencies use any paving applications of CRM as routine construction production, these issues should be carefully studied and resolved.

The importance of the research issues at the state level can not be overemphasized for at least two reasons. First, the details of material compatibility, mix design, and construction vary from state to state. Thus, each state should develop its own mix designs and specifications. Second, each state has a need to develop its own set of performance criteria. This requires sufficient documentation through field evaluation programs to record the adequacy of the design procedures and construction practices.

1.2 Objectives of Study

The primary objective of this study is to evaluate the feasibility of asphalt-rubber binder in road construction with particular emphasis on characterization of performance-related properties of asphalt-rubber concrete and development of pertinent guidelines for design purposes. The specific objectives are as follows:

- Evaluate the extent and rate of swelling of crumb rubber tires due to chemical reactions between asphalt cement and rubber.
- Characterize the rheological properties of asphalt-rubber binder, including swelling, viscosity, dynamic shear modulus, phase angle, and aging effects.
- Evaluate and establish optimum mixture designs for asphalt-rubber mixture, including both wet and dry processes.
- Characterize pertinent mechanical properties of the developed mixes of asphalt-rubber concrete, including indirect tensile strength, resilient modulus, fatigue, low-temperature cracking resistance as characterized by the TSRST (Thermal Stress Restrained Specimen Test), incremental creep, water sensitivity, and loaded wheel track test.
- Predict the performance of the selected asphalt-rubber concrete pavement using the computer program KENLAYER and conduct life cycle cost analysis.
- Develop mixture design procedures and design guidelines for the use of these materials in the field.

1.3 Outline of the Report

Presented in Chapter II are the literature review of the pertinent previous studies pertaining to the asphalt-rubber binder properties and their test methods, existing CRM technologies (both wet and dry process), and summary of past field performance of CRM asphalt paving Materials. In addition, SHRP(Strategic Highway Research Program) equipments and test procedures for characterizing asphalt binder are briefly reviewed.

Provided in Chapter III are the binder test results of CRM asphalt binder. The swelling behavior of rubber in asphalt cement, both in terms of wet process and dry process, is reported. The viscosity of various CRM asphalt binders as measured by the rotational Brookfield viscosity apparatus is summarized for both aged and unaged specimens at a wide range of temperatures. Short-term aging of CRM asphalt binders is accomplished through thin film oven test. Dynamic shear rheometer test was conducted on the CRM asphalt binders to evaluate the viscoelastic properties of various CRM asphalt binders as well. Preliminary screening and selection of candidate CRM asphalt binders was reported at the end of the chapter based on the recommended viscosity range.

Presented in Chapter IV are the results of mix design. Marshall method is used for both controlled conventional hot mix and rubber modified asphalt concrete mixes. Open graded friction course incorporating asphalt-rubber binder is also studied and presented in this chapter.

Presented in Chapter V are the test results of mechanical properties of rubber modified asphalt concrete mixtures, including resilient modulus, indirect tensile strength, fatigue, TSRST, incremental creep, water sensitivity, and rutting potential using the loaded wheel track tester.

Presented in Chapter VI is the performance predictions for asphalt-rubber concrete pavement. Predicted life of the asphalt-rubber concrete pavement is obtained by using a well documented computer program, KENLAYER, developed by the University of Kentucky. Life cycle cost analysis is followed based on the predicted life.

Finally, chapter VII provides summary and conclusions of the study and recommendations for future research.

CHAPTER II

BACKGROUND

This chapter presents a literature review pertinent to the following aspects of crumb rubber modified asphalt concrete:

- Historical development and existing technologies, including both wet and dry processes.
- Absorption and swelling mechanisms of crumb rubber in asphalt binders.
- Existing knowledge on mix design for crumb rubber modified asphalt concrete mixtures, including both wet and dry processes.
- Published data on field performance.
- Recent development of performance based test methods for evaluation of asphalt concrete material properties.

2.1 Crumb Rubber Modifier (CRM)

Tire rubber has been used as an additive to asphalt cement in various highway pavement applications for 40 years. Usually, tire rubbers were ground into crumbs prior to their use. CRM is a general term used for scrap tire rubber that is reduced in size and is used as a modifier in asphalt paving materials. The principal source of raw material for producing CRM is scrap tire rubber. Scrap tire rubber can be delivered to the processing plant as whole tires, cut tires, shredded tires, or retread buffing waste. In addition to mechanical size reduction of scrap tire rubber, fibers and soils are removed during manufacturing process to obtain high quality CRM.

CRM technology is a general term to identify a group of concepts which incorporate scrap tire rubber into asphalt paving materials. Based on the methods used to add the crumb rubber into an asphalt paving material, there are two basic processes: wet process and dry process. The wet process defines any method that blends the crumb rubber with asphalt cement prior to combining the binder with the aggregate. The dry process defines those methods that mix the crumb rubber with the aggregate before the aggregate/crumb rubber is combined with the asphalt binder.

Processes for Producing CRM

There are basically four methods of processing scrap tire into CRM; namely, crackmill, granulator process, micro-mill process, and cryogenic process. Among them, the crackmill (ambient grinding) process is the most commonly used method. The crackmill process involves reducing the size of the scrap tire rubber by passing the material between rotating, corrugated drums. The process is performed at ambient temperature and requires that the scrap tires be pre-processed by shredding.

The granulator (ambient granulating) process involves shearing apart the scrap tire rubber by cutting the rubber with revolving steel plates that pass at close tolerance. The granulator process is performed at ambient temperatures and can accommodate any form of scrap tire rubber, including whole tires.

The micro-mill (wet grinding) process further reduces a crumb rubber to a very fine ground particle. Usually the micro-mill process mixes crumb rubber with water to make a rubber slurry. The slurry is forced between rotating abrasive discs which reduce the rubber into minute particles.

The cryogenic process involves embrittling the scrap tire rubbers by submerging them in liquid nitrogen. The embrittled rubber is then crushed to the desired particle size. Although this technique has been successfully demonstrated, it is too costly for full scale production at this time.

Chemical Composition

Identification of chemical components in recycled, over-the-road, tire rubber and other types of CRM involves the application of testing methods described in ASTM D 297,"Standard test Methods for Rubber Products - Chemical Analysis" and ASTM D 3677, "Standard Test Methods for Rubber - Identification by Infrared Spectrophotometry." These methods are designed to identify chemical components in a rubber compound, and are widely used in recycled rubber analysis. It is a common practice to determine the content of acetone extract, ash, carbon black, rubber hydrocarbon, and natural rubber. These chemical components are expressed as the weight percentage of the test sample.

Tire rubber is primarily a composite of a number of blends of natural rubber, synthetic rubber, carbon black, and other additives. Various parts of the tire construction require specific rubber properties; e.g., flexible side walls, abrasion resistant tread, etc. These various parts of the tire contain rubber with different amounts of natural and synthetic components. Natural rubber provides elastic properties while synthetic rubbers improve the compound's thermal stability. As an example to show general chemical composition of tire rubber, a statistical analysis performed by Baker Rubber, Inc. on their crumb rubber is reproduced in Table 2-1.

Table 2-1 Chemical composition for passenger/light truck tread rubber

Composition	Mean by weight (%)	Standard Deviation	Min (%)	Max (%)
Acetone Extract	17.2	1.12	15.5	19.1
Ash	4.81	0.51	3.9	5.4
Carbon Black	32.7	1.72	30.4	35.5
Rubber Hydrocarbon	42.9	1.45	41.5	44.4

2.2 Use of CRM in Asphalt Mixtures

The Federal Highway Administration (FHWA) defines asphalt-rubber as "asphalt cement modified by CRM". The FHWA definition does not specify ranges of applicable rubber contents and, therefore, can be applied to any blend of asphalt and CRM. The ASTM definition of asphalt-rubber requires a minimum of 15% rubber which will achieve a binder with modified properties.

Addition of CRM to asphalt paving products can be categorized into two different concepts: as a binder modifier which modifies the physical and chemical properties of the binder in HMA, or as a rubber aggregate which replaces a portion of the mineral aggregate. Other major uses of crumb rubber are surface treatments (stress absorbing membranes (SAM) and stress absorbing membrane interlayers (SAMI) and crack/joint sealants.

2.2.1 Binder Modifier

There are numerous ways that one could classify the various additives and modifiers for asphalt. National Asphalt Pavement Association (NAPA) Quality Improvement Committee has adopted an overall classification method (Terrel and Walter, 1988) as shown in Table 2-2. As can be seen, reclaimed rubber is grouped in the same category as other synthetic rubbers.

Table 2-2 Generic classification of asphalt modifiers

Type	Examples
1. Filler	Mineral Filler: crush fines, lime, Portland cement Carbon Black and Sulfur
2. Extender	Sulfur and Lignin
3. Rubber a. Natural Latex b. Syntax Latex c. Block Copolymer d. Reclaimed Rubber	 a. Natural Rubber b. Styrene-Butadiene, SBR c. Styrene-Butadiene-Styrene d. Recycled Rubber or Crumb Rubber
4. Plastic	Polyethylene, Polypropylene Ethyl-Vinyl-Acetate (EVA) Polyvinyl Chloride (PVC)
5. Combination	Blend of polymers in 3 & 4
6. Fiber	Natural: asbestos, rock wool Man-made: Polypropylene, Polyester, Fiberglass
7. Oxidant	Manganese salts
8. Antioxidant	Lead compound, Carbon , Calcium salts
9. Hydrocarbon	Recycling and rejuvenating oils, Hardening and Natural asphalt
10. Antistrip	Aimes and Lime

2.2.1.1 Characteristics of Asphalt-Rubber

Physical properties of asphalt-rubber have been shown in numerous studies to be substantially different from unmodified asphalt cement. Many of these studies investigated the use of standard asphalt cement testing procedures such as penetration, absolute viscosity, ring and ball softening point, etc., as well as a variety of procedures used for other materials, including research-type procedures, such as the Schweyer Rheometer (Pavlovich, et al., 1979; Rosner et al., 1981), sliding plate viscometer (Rosner et al., 1981), force ductility, torque fork viscosity, mechanical spectrograph or dynamic mechanical analysis (Green et al., 1977), and several others. Each of these procedures provides an indication of certain characteristics of asphalt-rubber binders. For any application that an asphalt-rubber will be used in, there are several general characteristics of the blend that should be considered.

- Pumping consistency at the placement temperature.
- Consistency at the high range of the in-use temperature that the material will be subjected to.
- Consistency at moderate in-use temperature.
- Elastic characteristics.
- Elongation properties.
- Stiffness and fracture characteristics at the low temperature range.

Table 2-3 (Heitzman, 1992) shows the proposed physical property limits for asphalt-rubber binders for usage in HMA.

Table 2-3 Proposed specification for AR binder in HMA (Heitzman, 1992)

Test Parameter	Climate usage		
	Hot	Moderate	Cold
Apparent Viscosity © @175°C, spindle #3, 12 ram (ASTM D 2669)	min 1000 max 4000	min 1000 max 4000	min 1000 max 4000
Penetration, 25°C, 100 g 5 sec, (ASTM D)	mim 25 max 75	min 50 max 100	min 75 max 150
Penetration, 4°C, 60 sec, (ASTM D)	min 15	min 25	min 40
Softening point (ASTM D36)	min 130	min 120	min 110
Resilience, 25°C, % (ASTM D3407)	min 20	min 10	min 0
Ductility, 4°C, 1 cm/min (ASTM D113)	min 5	min 10	min 15
Thin-film residue (ASTM D 1754) Penetration retention, 4°C, % of original	min 75	min 75	min 75
Ductility retention, 4°C % of original	min 50	min 50	min 50

However, this specification does not conform to the newly developed Superpave (*Su*perior *Per*forming Asphalt *Pave*ments) specification. The 5-year Strategic Highway Research Program (SHRP) did not develop an exclusive specification for the CRM binders or mixtures. Modifications on its resulting system, the Superpave, are supposed to be made for the CRM binders. Kenneth Troy et al. (1996) did some work in evaluating the SHRP grading system for application in CRM binders. Six CRM binders containing either coarse (larger than No.20) or ultrafine (smaller than No.200) rubber particles were investigated. The CRM binders containing rubber particles between No.20 and No.200 were not included in their research. They found that the dynamic shear rheometer with the parallel plate configuration in the SHRP system has some limitations in grading the CRM binders. These limitations become significant when dealing with the CRM binders containing rubber particles larger than No. 20. This comes from the principle of keeping a minimum ratio of 4 between the sample size and the maximum particle size. In the case of CRM binders containing No. 20 particles, this ratio drops to 1.2 and 2.4 for the 1-mm and 2-mm gaps, respectively. Therefore, both 1-mm and 2-mm gap sizes violate the standard ratio, which is also emphasized in the AASHTO provisional test method TP5-93 for DSR testing of binders. The DSR with the plate and cup configuration in conjunction with the BBR was recommended by Kenneth Troy et al. to grade the CRM binders containing rubber particles larger than No. 200.

2.2.1.2 Superpave Asphalt Binder Tests and Specification

Currently CRM binder design follows the Superpave testing system and specification. In the Superpave specification, the physical properties remain constant for all performance grades (PG), but the temperature at which these properties must be achieved varies form grade to grade depending on the climate in which the asphalt binder is expected to perform. Table 2-4 gives the list of testing equipment to conduct various Superpave physical tests, the related purpose for testing, and the related performance parameters being partially influenced by the asphalt binder. For a detailed Superpave asphalt binder specification, please refer to AASHTO MP1-93.

Table 2-4 Superpave Asphalt Binder Testing Equipment and Purpose

Equipment	Purpose	Performance Parameter
Rolling Thin Film Oven (RTFO)	Simulate Binder aging during HMA production and construction	Resistance to aging during service life
Pressure Aging Vessel (PAV)	Simulate binder aging during HMA service life	Resistance to aging during service life
Rotational Viscometer (RV)	Measure binder properties at high construction temperatures	Handling and pumping
Dynamic Shear Rheometer (DSR)	Measure binder properties at high and intermediate service temperatures	Resistance to permanent deformation (rutting)and fatigue cracking
Bending Beam Rheometer (BBR)	Measure binder properties at low service temperatures	Resistance to thermal cracking
Direct Tension Tester (DTT)	Measure binder properties at low service temperatures	Resistance to thermal cracking

The Superpave test and specification have the following features:

- Tests and specifications are intended for asphalt “binders” which include both modified and unmodified asphalt cements.
- The specified criteria remain constant. However the temperature at which the criteria must be met changes in consideration of the binder grade selected for the prevalent climatic conditions.
- The physical properties measured by Superpave binder tests are directly related to field performance by engineering principles.
- The Superpave binder specification requires the asphalt binder to be tested after simulating its three critical stages: (a) the first stage is represented by original

asphalt binder which has to be transported, stored, and handled prior to mixing with aggregates, (b) the second stage is represented by the aged asphalt binder after HMA production and construction (short-term aging), and (c) the third stage is represented by the asphalt binder which undergoes further aging during a long period of time in service.

- The entire range of pavement temperatures experienced at the project site is considered.
- Tests and specifications are designed to eliminate or control three specific types of HMA pavement distresses: rutting, fatigue cracking, and thermal cracking. Rutting typically occurs at high temperatures, fatigue cracking at intermediate temperatures, and thermal cracking at low temperatures.
- The Superpave asphalt binder test procedures and specification were developed in SI units which will be used in this section without English units.

2.2.1.3 Rheological Properties of Asphalt-rubber Binders

The Brookfield rotational viscometer with thermosel apparatus is recommended by Superpave for measuring the viscosity of asphalt binder as a fluid at high temperatures. Asphalt-rubber binder exhibits non-Newtonian behavior; i.e., the viscosity of asphalt-rubber binder varies with the shear rate. It has been found that asphalt-rubber binder is a pseudoplastic material whose viscosity decreases with the increase of the shear rate. This behavior is usually called shear-thinning. Steve Lalwani et al. (1982) showed the non-Newtonian behavior of asphalt-rubber with 30 percent of Goodyear truck tire rubber.

The bending beam rheometer device was developed to measure the rheological properties of the binder at low temperatures in terms of the creep stiffness, S(t), and the slope, m, of the creep stiffness versus loading-time curve. In the Superpave binder specifications, S(t) is related to low temperature cracking; m is related to both low temperature cracking and fatigue cracking. The dynamic shear rheometer test was developed to measure the rheological properties of the binder at medium temperatures in terms of complex shear modulus (stiffness), G^*, and phase angle, δ. In the Superpave binder specifications, the parameter $G^* \sin \delta$ relates to fatigue cracking and $G^*/\sin \delta$ relates to permanent deformation. The direct tension test device is used for measuring the failure properties of asphalt cement when the stiffness S(t) obtained from the bending beam rheometer test is between 300 and 600 MPa.

Gary V. Gowda et al. (1996) reported Arkansas experience with crumb rubber modified mixes using Marshall and SHRP Level I design methods. The asphalt used in their study was AC30. The crumb rubber had a mean particle size of 74 microns (75 microns = No.200). The binder test results are shown in Table 2-5

Table 2-5 Performance Grade Classification of Binders (Gary V. Gowda et al. 1996)

PG Classification Criteria	AC30	A-R 5%[a]	A-R 10%	A-R 15%
Brookfield Viscosity 20 rpm, 135°C, 3 Pa-s (Max)	0.42 Pa-s	0.75 Pa-s	1.66 Pa-s	3.1 Pa-s
Dynamic Shear Rheometer Original TFO Residue PAV Aged	 64 C 64 C 25 C	 70 C 70 C 25 C	 80 C[b] 80 C[b] 25 C	 80 C[b] 80 C[b] 22 C
Bending Beam Rheometer	-12 C	-18 C	-18 C	-24 C
PG Grade	PG 64-22	PG 70-28	PG 80-28	PG 80-34

[a]Indicates that the blend was constituted with 5%CRM by weight of asphalt.
[b]Indicates that it was not possible to test the binders beyond 80 C

The performance grading of the unmodified and rubber modified asphalt shows that the blending of crumb rubber broadened the range of applicability of the asphalt. The high temperature increased from 64°C to 80°C with 10 and 15 percent A-R rubber blends and the lower temperature decreased from -22°C to -34°C with 15 percent A-R blends. There was, however, no indication of improvement in load-associated fatigue resistance. Among the asphalt-rubber binders tested in the study the 15 percent A-R blends marginally (3.1 Pa-s) exceeded the viscosity limits (3 Pa-s).

2.2.1.4 Swelling

When CRM is added to asphalt cement, the rubber particles will generally become swollen. The extent and rate of swelling has been found to be dependent on many factors, including temperature of the mixture, time of reaction, and physical and compositional characteristics of the asphalt and rubber. As the rubber particles swell, the interparticle distance between them is reduced which results in an increase in the viscosity of the blend.

Tolonen and Green (1977) concluded that rubber swelling was due to absorption of the oil or asphalt fluid that it was immersed in and that different oils were absorbed to greater or lesser degrees. Both rubber and asphalt cement are polymeric materials which have long, large molecules containing repeating structural units of the original molecule. Diffusion theory has been adopted to explain the transport of a liquid or oil into the rubber in polymer science. It is generally known that swelling of a rubber particle is a function of crosslinking density, interaction parameters, and temperature.

The plot of percent weight increase versus time at constant temperature has been used to quantify the swelling process. In the initial analysis of the swelling test data, a linear relationship was found by plotting ln (t+a) against percent swell (by weight), where t is time and a is a curve-fitting constant. The curve-fitting equation of the following form is used:

$$S = m \ln (t + a) - C \tag{2-1}$$

where

S = percent swell by weight
m = slope of the curve fitting line
a = curve-fitting constant
C = intercept of the curve fitting line on the vertical axis

The above equation provides a means for calculating the percent swell at a given elapse time. However, the maximum percent swell can not be predicted. It is therefore postulated that a relationship which would more closely describe the swelling mechanism should be in the form of the following equation:

$$S = S_m (1 - C e^{-kt}) \tag{2-2}$$

where

S = percent swell by weight
S_m = maximum possible swell when time of interaction is infinite
k = rate of swell

C = curve fitting constant

t = elapse time

This equation allows the prediction of maximum possible swell, which is an advantage over Eq. 2-1. For convenience of plotting, Eq. 2-2 is arranged as follows:

$$\ln\left[\frac{S_m}{S_m - S}\right] = kt - \ln C \qquad (2\text{-}3)$$

S_m is then adjusted until a straight line is obtained by trial and error; k is the slope; ln C is the intersection of the curve fitting line on the y-axis.

2.2.1.5 Aging Effect

Durability of an asphalt-rubber is considered as its resistance to change in properties when subjected to processing and weathering and is manifested primarily by its resistance to hardening with time. There are a number of factors that contribute to this phenomenon of hardening with time, such as oxidation, volatilization, polymerization, rheopexy, separation, and syneresis. In general, aging of asphalt-rubber can lead to reduced service life. This is brought about due to (i) the binder becoming brittle, (ii) reduction in adhesive characteristics, and (iii) possible loss of asphalt and CRM. The Rolling Thin Film Oven Test (RTFOT) and the Pressure Aging Vessel (PAV) test are selected by Superpave as an accelerated short-term and long-term aging test, respectively. The aging of binders is defined by measuring the changes in rheological and physical properties before and after the respective aging tests.

2.2.2 Rubber Aggregate

In addition to being used as a modifier for asphalt binder, CRM can be used as rubber aggregate. By limiting the time that the asphalt cement and CRM are maintained at the mixing temperatures and by specifying a coarse granulated CRM, the CRM can retain its physical shape and rigidity. This rubber aggregate product is only applied to hot-mixed asphalt designs and cannot be applied to non-plant mixed applications, such as surface treatments.

When the CRM is used as an elastic rubber aggregate by replacing some of mineral aggregate, the CRM and aggregate are heated separately and mixed at the elevated temperature. Asphalt binder, which is also heated, will be brought into the CRM-aggregate mixture just before construction. Control of the gradation curve for the CRM-aggregate mixture is critical. The aggregate gradation must provide enough space for CRM so that the CRM-aggregate mixture can be stable for mixture design.

There exists some swelling after mixing the CRM-aggregate with asphalt binder. Since the swelling of CRM is related to temperature, most of the swelling will occur shortly after construction. However, in the high temperature region, there exists a tendency for some life-long swelling process.

2.3 Existing CRM Technologies

At the present time, there exist two distinctive methods that can be classified as wet process: the McDonald (batch) technology and the continuous blending technology. On the other hand, there exist three methods for dry process: the PlusRide technology, the generic dry technology, and the chunk rubber technology.

2.3.1 Wet Process

In the wet process the CRM is blended with asphalt cement before mixing the CRM-modified binder with aggregates. In the McDonald technology, the CRM is mixed in a blending tank and allowed for reaction in a holding tank. Then, the modified binder is introduced into the mixture. In the continuous blending technology, the CRM and the asphalt cement can be mixed just before the binder is introduced into the aggregate or it can be mixed and placed in a storage tank for use later. The amount of CRM used in McDonald method is usually fifteen to twenty-two percent by weight of the asphalt cement. The size of CRM is generally between 10 to 30 mesh. The continuous blending technology uses a finer grade of CRM, with the amount ranging from five to twenty percent by weight of the asphalt cement.

2.3.1.1 Historical Development

The potential benefits of adding rubber to asphalt cement have been discussed for many years; however, its use was delayed due to lack of technology and equipment to economically mix the rubber in asphalt cement. In 1964 Charles McDonald, who worked for the city of Phoenix, developed a method to add small, ground, scrap-rubber particles to asphalt cement. The waste tires used contained vulcanized rubber which provided a material that would provide certain desirable properties in the asphalt cement. These techniques allowed the rubber to be processed and added to asphalt cement at a lower cost than had earlier been practiced.

In 1968, the first asphalt-rubber pavement section was placed with a slurry seal machine. For that construction the liquid asphalt-rubber was applied with an asphalt distributor followed by an application of chips. This is commonly referred to as a stress absorbing membrane (SAM). Use of the slurry seal machine and asphalt distributor improved the quality of the application and also greatly increased the production capacity. As a result, the construction cost of the asphalt-rubber section dropped considerably. When the SAM is used prior to an overlay, it is called a Stress Absorbing Membrane Interlayer (SAMI). The first SAMI was placed in 1971 in the city of Phoenix.

The consistency of the asphalt-rubber was initially very thick. It had to be heated to a higher temperature than the conventional asphalt cement. Even at high temperatures, it was still difficult to pump and spray. In 1972 kerosene began to be added to the asphalt rubber to lower the viscosity. This addition of kerosene improved the workability of the asphalt-rubber and thus improved the quality of the construction. In 1974 this mixture of asphalt cement and crumb rubber began to be used as a crack sealer.

Arizona Refinery Company (ARCO) in 1975 developed an asphalt rubber mixture to compete with that of the Sahuaro mix. The ARCO mix used 80 percent asphalt cement and 20 percent crumb rubber, including devulcanized CRM, along with an extender oil instead of kerosene. These two technologies eventually merged between 1983-1985 and became known as the McDonald technology.

The first use of asphalt-rubber in HMA in Arizona was in 1975. Two sections of asphalt-rubber in an open-graded friction course were placed on State Route 87. One section contained 10.5 percent binder having 25 percent vulcanized rubber, and the other section contained 8.5 percent binder having 20 percent devulcanized rubber.

California first began using McDonald's asphalt-rubber in 1978. From 1978 to 1988 California placed approximately 20 overlay projects using asphalt-rubber. This work was done with dense-graded mixes as well as gap-graded mixes. With additional experience, California developed a design guideline in 1992 that allowed a reduced overlay thickness for gap-graded HMA with asphalt-rubber.

Ludo Zanzotto et al. (1996) brought in an idea of depolymerization and devulcanization of scrap tires in asphalt to improve the asphalt-rubber binder properties. When the existing technologies of using scrap tires in asphalt pavements are analyzed, it has become obvious that the dry and wet technologies stop at using rubber as an elastomeric filler. In the case of dry processes the rubber forms part of the aggregate component. Even the wet processes use the rubber as a filler since only a small percentage of the three dimensional network of the vulcanized rubber is depolymerized or dissolved in asphalt. Most of the material remains intact with a somewhat loosened rubber matrix, swollen by the asphalt's oils. Furthermore, the prepared rubber/asphalt mixtures have to be used in a short time because of the danger of their gelation. The method of depolymerization used in the study are (i) heating; (ii) shear; and (iii) combination of heating and shear. For the heating method, tire rubber was incorporated into asphalt in a reactor at temperatures between 200°C and 280°C. For the shear method, tire rubber was added to asphalt with the use of high shear via a colloid mill at temperatures between 170°C and 180°C. Combination of the two methods was achieved by first subjecting the

CRM/asphalt mixture to the heat treatment in reactor and subsequently to the high shear in colloid mill. The asphalt used in the study is 200/300 pen grade. Tire rubber is 15%60-mesh, 10%60-mesh, 6%buffings, and 6%20-mesh. Results of testing of these depolymerized rubber/asphalt mixtures according to SHRP specification are compared with the results of the conventional asphalts (85/100 pen grade, 120/150 pen grade, 150/200 pen grade, and 200/300 pen grade). It was found that all rubber/asphalt mixtures meet with more stringent PG specification requirement than the base asphalt.

2.3.1.2 McDonald Technology

In the wet process, the CRM is blended with the asphalt cement before mixing the CRM-modified binder with the aggregate. In the McDonald technology, the CRM is mixed in a blending tank and reacted in a holding tank before introduction into the mix. The major differences between production of a McDonald HMA and a conventional HMA is the pre-blending and reaction of the CRM with asphalt cement to produce an asphalt-rubber binder for the resultant HMA mixture. The reaction is accomplished in insulated trucks and/or tanks. When the CRM is added to the asphalt, the temperature of the asphalt cement is between 350° F to 400° F. The asphalt cement and CRM are combined and mixed in a blender unit and then pumped into an agitated storage tank for reaction. The reaction tank has a mechanical agitating system that will keep the mixture dispersed. The temperature is maintained between 325° F to 375° F during the minimum 45 minutes reaction time. The required amount of asphalt-rubber binder is added at the mixing chamber of the HMA production plant to produce final paving mix.

2.3.1.3 Continuous Blending Technology

One concern regarding the McDonald wet process is the required batching and reaction time associated with blending CRM and asphalt cement to produce asphalt-rubber. Rouse Rubber Industries developed a continuous blending procedure which uses their 180-micron (No. 80) sieve CRM with an asphalt cement. The very fine gradation of 180-micron CRM substantially increased the dispersion of the CRM throughout the asphalt cement.

The difference between the McDonald technology and the continuous blending technology is the manner in which the CRM and asphalt cement are blended and reacted. Also, the McDonald technology uses a coarser CRM than the continuous blending process. Typically in the continuous blending technology, 5 to 20 percent ground rubber is blended with AC-5 or AC-10 asphalt. The idea is that the use of the fine rubber gradation will shorten the reaction time between CRM and asphalt cement. Florida DOT provided research funding to the National Center for Asphalt Technology (NCAT) to study the feasibility of continuous blending technology. The recommended CRM gradation is shown in Table 2-6.

Table 2-6 Continuous blending technology CRM gradation

Sieve size	Percent passing
No. 60	98-100
No. 80	88-100
No. 100	75-100

The first field application of the continuous blending technology was done in 1989

(Page, 1992). The trial results of the dense-graded surface mix (0%, 3.1%, 5.3% minus No. 80 mesh, and 11.1% minus No. 40 mesh) and the open-graded surface mix (0%, 5.5%, 11.1%, 17.7% minus No. 80 mesh, and 20.5% minus No. 24 mesh) showed that rideability and rutting depth were essentially not different from the control sections, after two years of service.

2.3.1.4 Summary of Field Performances (Wet Process)

A limited number of states have applied wet process technology in the field. Results of a recent survey by the Asphalt Rubber Producers Group (ARPG) indicate that, between 1975 and 1987, there were at least 35 projects in 12 different states that utilized asphalt-rubber as a binder in the mixture. Some field performance data are reviewed.

Arizona (1991): The Arizona Department of Transportation has been using asphalt-rubber for pavement construction since the mid 1960's. The major application has been in mitigating reflection cracking either as SAM or SAMI. However, since 1987, the asphalt-rubber has been used as binder in the open-graded and dense-graded paving mixtures. Performance of these field application has not been reported.

California (1989): The California Department of Transportation has been experimenting with rubber-modified asphalt mixtures for pavement overlays. The results from several experimental overlay projects indicated that asphalt-rubber mixtures are more abrasion resistant and have higher permeabilities than conventional asphalt concrete mixes.

Connecticut (1989): In October 1980, the Connecticut Department of Transportation placed an experimental 900-foot section of an asphalt-rubber overlay in Madison, Connecticut. Finely ground rubber was premixed with an AC-20 asphalt to produce a binder that was 80 percent asphalt and 20 percent rubber. An 8-year performance evaluation found that the asphalt-rubber pavement was performing better than the control pavement on the basis of transverse, longitudinal, and allegator cracking. Skid resistance and roughness were also found to be acceptable and were similar to these values measured on the control section.

Florida (1989): Florida DOT has used 5 percent of ground tire rubber (GTR) passing the No. 50 sieve (a maximum nominal 80 mesh) in the dense-graded friction course. Also, 12 percent of GTR passing the No. 30 sieve (a maximum nominal 40 mesh) has been used for open-graded friction course.

Measurements of pavement friction and rutting depth have been made at the three test sites and no differences have been identified that can be attributed to the use of GTR.

Kansas (1994): Kansas Department of Transportation has constructed eight rubber hot bituminous mix projects from 1990 to 1992. Four were wet process and four were dry process. Even though it is too early to evaluate the final performance of the rubber projects at the time of report, a few of preliminary conclusions are presented: (a) rubber may not inhibit the development of cracks in the higher-density mixes; (b) the gap-graded mixes showed the greatest potential in reducing the amount of cracking. (c) neither the rubber project sections nor the control sections have rutted.

Minnesota (1986): The Minnesota Department of Transportation constructed a project in 1984 that used asphalt-rubber (20 percent CRM) as a binder in a dense-graded mix. Field observations have shown no differences in the amount of cracking between the asphalt-rubber and conventional asphalt concrete overlays.

Virginia (1996): Four test sections using asphalt rubber hot mix were placed in Virginia from 1990 to 1993. CRM (minus 2-mm) contents varied from 5% to 20% and asphalt grades were AC10, AC20, and AC30. McDonald process and Rouse (continuous blending) process were used in preparing the asphalt rubber binders. Based on the limited time of evaluation, asphalt rubber mixes perform at least as well as regular mixes. Although the overall degree of rutting was small, there was a significant difference between the control and the rubber mixes. The mixes containing rubber had less rutting. More evaluation time is needed to determine if long-term performance of the asphalt rubber mixes is superior to the conventional mixes.

Wisconsin (1989): In 1987, the Wisconsin Department of Transportation constructed two experimental pavement that included test sections incorporating ground tire rubber in asphalt cement for a recycled mix. Performance observations made thus far showed rather disappointing results for the asphalt-rubber mixtures. Compared to a standard recycled mix, the recycled asphalt-rubber mix developed five times more transverse cracking during the first two years of services.

2.3.2 Dry Process

In the dry process, the crumb rubber materials are blended with the aggregate before adding the asphalt cement to the blended mixture. The mix production of the dry process is similar to the production of conventional HMA.

2.3.2.1 Historical Development

The original concept of dry process was developed by two Swedish companies, Skega AB and AB Vaegfoerbaetringar, in the late 1960's, as a product named Rubit. The Swedish incorporated 3 to 4 percent CRM (by weight of total mix) into a HMA mixture. The rubber particles were 1.6 to 6.4 mm (No. 16 to 1/4 in. sieve) in size, which were larger than the CRM used in the McDonald mixtures. The Swedish technology was patented for use in the United States in 1978 under the trade name PlusRide. The mix design was refined in the mid-1980's, establishing the gap-graded mix now commonly used.

2.3.2.2 PlusRide Technology

This process primarily uses CRM as a rubber aggregate which is incorporated into a gap-graded aggregate prior to mixing with asphalt cement, producing a rubber modified hot mix asphalt concrete (RUMAC). The coarse rubber particles act as elastic aggregates which flex on the pavement surface under traffic and break ice. The mix design was refined in the mid 1980's establishing the gap-graded mix now commonly called PlusRide. EnvirOtire, Inc. markets this technology at the present time as PlusRide II.
Rubber used in PlusRide mixtures must conform to the gradation shown below in Table 2-7.

Table 2-7 Recommended CRM gradation for PlusRide

Sieve size	Percent passing
1/4 in.	100
No. 4	76-88
No. 10	28-42
No. 20	16-42

Aggregates used in this process must possess one of the gradations given in Table 2-8. Three aggregate gradations, which reflect different maximum aggregate sizes, are specified in the PlusRide II system as shown in Table 2-8.

Table 2-8 Aggregate gradations and AC contents of PlusRide II

Property	PlusRide	PlusRide	PlusRide
3/4"	-	-	100
5/8"	-	100	-
3/8"	100	60-80	50-62
1/4"	60-80	30-44	30-44
No. 10	23-38	20-32	20-32
No. 30	15-27	13-25	12-23
No. 200	8-12	8-12	7-11
AC content	8-9.5 %	7.5-9 %	7.5-9 %

For the PlusRide II 12 and PlusRide II 16 mixtures, the gap grade requirements restrict the amount of aggregate passing the 1/4" sieve and retained on the No. 10 sieve to be 12 percent maximum. Failure to provide a sufficient gap grading would have caused the coarse rubber to resist compaction and result in a low density pavement with high air voids. In order to fill the air voids, the mixture also contains a higher minus-200 content compared to the conventional HMA mixtures. The CRM is handled like an aggregate, and is dry mixed with the hot mineral aggregate prior to mixing with the asphalt cement. Generally, a mix design using this concept will include a percentage of ground CRM passing No. 20 sieve which produces a partially reacted modified binder.

The limited reaction time does allow the surface of the coarse rubber particle to react with the asphalt cement, but does not permit sufficient time for the reaction to penetrate the entire rubber mass. This creates an asphalt/rubber interface which bonds the two materials together. The following advantages of the RUMAC have been claimed from laboratory studies: 1) increased fatigue life, 2) resistance to reflective, shrinkage and thermal cracking, 3) great resistance to rutting, 4) ice disbonding.

Mix Design Method

Since PlusRide II is a resilient/elastic RUMAC mixture, the conventional criteria of stability and flow do not apply to the mix design. The objective of design is to determine the gradation of aggregates, asphalt content and rubber content that yield a mix having:

- A high-coarse aggregate content. Gap-graded aggregate is to provide space for the rubber granules to form a dense, durable and stable mixture upon compaction.
- A rich asphalt/filler ratio. Asphalt cement and filler are used to fill voids. The mix must have a high asphalt content to ensure a workable mixture and durable pavement.
- A low void content in the compacted mix. The voids should be in the range of 2 percent to 4 percent, with 3 percent being the normal.

The mix design procedure used for PlusRide consists of the following basic steps (EnvirOtire, 1992):

- Preparation of mixtures.
  - Weigh out ingredients.
  - Heat aggregate and asphalt in 320° F oven (temperature should vary with asphalt grade.)
  - Dry mix aggregate and rubber for 15 second.
  - Wet mix for 2 minute.
  - Cure loose mix at 320° F for one hour.
- Prepare compaction mold and hammer - need to lubricate mold with silicon grease to prevent sticking.
- Compact using Marshall hammer (50 blows/side). Cool the confined specimen and remove it from the mold.
- Determine void content.

One mixture criterion normally used is voids (2 to 4%); however, some agencies have added other criteria such as minimum modulus and/or index of retained strength. Typical design criteria for PlusRide are given in Table 2-9.

Table 2-9 Mix design criteria for PlusRide

Property	Value
Voids (%)	2-4
Resilient Modulus @ 25° C (min.)	100,000 psi
IRS (%)-AASHTO T-283	75

2.3.2.3 Generic Dry Technology

The first generic dry technology for adding crumb rubber to HMA was developed by Takallou in 1986 as a result of his research on PlusRide at Oregon State University. The focus of this concept in CRM technology is to incorporate CRM into conventional dense and gap-graded HMA mixes using the dry process.

Unlike PlusRide, which specifies only gap gradation for the aggregate, the proposed technology considers the available "generic" aggregate gradations for the locality; hence the name, generic dry technology. Major drawbacks of the PlusRide system include the addition of crumb rubber to a unique "gap-graded" aggregate gradation, and nonconventional design criteria. These factors contribute to the high cost of using the material when compared with conventional asphalt concrete.

The generic dry process relates to a process producing asphalt concrete composition made up of coarse crumb rubber and fine crumb rubber incorporated into a standard dense-graded aggregate mixture. This process is characterized by the various constituents of the asphalt binder and fine crumb rubber, mixed immediately by a physical reaction. This will result in a higher viscosity binder in which the optimum reaction is achieved when the fine crumb rubber particles reach optimum swelling. A pre-reaction or pre-treatment of crumb rubber with a catalyst may be needed to achieve the optimum crumb rubber particle swelling. This system can be designed using conventional testing procedures and complies with conventional design criteria. The use of this system is in the public domain. This generic dry technology system is sometimes referred to as generic RUMAC or the TAK system.

The first field evaluation of RUMAC was done on two projects in the State of New York in 1989. These projects involved placing generic RUMAC sections with 1 percent, 2 percent, and 3 percent CRM, as well as a control section and PlusRide section. Field evaluation of RUMAC was also performed in a number of other places, including Ontario, Oregon, Illinois, and California. The first generic RUMAC used an equivalent or slightly lower percentage of CRM compared to PlusRide. The CRM was also finer than that used in PlusRide. A conventional dense-graded aggregate is used with only slight modification. The gradation of CRM was adjusted to suit the aggregate gradation. It is a two component system in which the fine crumb rubber interacts with asphalt cement, and the coarse crumb rubber performs as an elastic aggregate in the HMA mixture.

In Florida, another type of generic dry technology was developed (Page, 1989) which uses lower amounts of CRM and smaller size (No. 80 mesh) of CRM as compared to generic RUMAC in 1989. It is believed that the fine CRM modifies the asphalt binder during the mixing process, subsequent storage, and transportation of the HMA to the job site. Number 80 mesh CRM was used in an open graded friction course (nominal maximum aggregate size of 9.5 mm or 3/8 inch) at 10 percent by weight of binder.

Mix Design Method

Typical mixture design criteria for generic dry process are given in Table 2-10 for mix design using the Marshall method. Optimum asphalt content is selected based on the air voids. The voids should be in the range of 2 to 4 percent, with 3 percent being the normal. Similar to the control mix without CRM, the selected mix must meet the minimum stability requirement. Marshall flow should not exceed 20.

Table 2-10 Example of mix design criteria for generic type mixes

Property	Value
Marshall stability (min.)	800 lb
Flow (0.1 in.)	8-20
VMA (% min.)	17
Air voids (%)	3-5
IRS (%)	more than 75

2.3.2.4 Chunk Rubber Asphalt Rubber

As a part of the Strategic Highway Research Program, the Cold Regions Research and Engineering Laboratory (CRREL) of the U.S. Army Corps of Engineers was contracted to evaluate the ice-bonding characteristics of several asphalt paving materials. One of those materials was PlusRide. In addition to this research effort, the CRREL began modifying the design to determine if the use of CRM could further modify the properties of the paving material. Their work (Eaton et al., 1991) focused on increasing both the maximum size of the crumb rubber and the percent of CRM in the HMA.

The CRREL concept revised the aggregate gradation from the gap-graded PlusRide design to dense-graded aggregate, while maintaining the same nominal maximum aggregate size. The CRM gradation was revised to a narrow grading band (½ inch to No. 4 sieve) with a larger maximum crumb size. This revision of the gradations applies to mixes with CRM contents similar to PlusRide, namely 3 percent CRM by weight of the mix. As the CRREL research increased the percent of CRM, adjustments were made in the aggregate gradation to provide space in the aggregate matrix for the substitute rubber aggregate. This research examined chunk rubber asphalt concrete mixes with 3, 6, 12, 25, 57, and 100 percent crumb rubber by weight of aggregate. As expected, the optimum asphalt cement content increased as the percent CRM increased. Actual Marshall mix designs produced asphalt cement contents ranging from 6.5 percent for 3 percent CRM to 9.5 percent for 12 percent CRM.

This research initiative has been confined to laboratory testing. There are no scheduled experimental field applications for this concept. The CRREL is presently seeking sources of research funding to continue the development of these unique mixes. Until the material is subjected to actual field condition, it is impossible to estimate its performance or practical application.

2.3.2.5 Summary of Field Performance (Dry Process)

Mt. St. Helens Project (1987): PlusRide and control sections had been monitored from 1983 to 1986. Testing of the core samples was performed at Oregon State University. Tests conducted on these samples included:

- Bulk specific gravity.
- Diametral resilient modulus.
- Diametral fatigue.
- Hveem stabilometer.
- Indirect tension.

A comparison of the change in resilient modulus after construction for both materials shows that the rate of increase is decreasing for both materials, with the PlusRide sample showing a slightly greater overall increase compared to the control mixture. It should, however, be noted that the PlusRide sample is considerably more flexible than the conventional mixture, as determined by modulus testing.

After determining moduli, the samples were tested in the diametral configuration for fatigue using a constant stress-repeated load until failure. The R^2 (correlation coefficient) for the PlusRide samples are normally lower than those of the control mixes. This may indicate that diametral testing is not the most appropriate means of determining fatigue lives of rubber-modified materials. Even though both rubber-modified and control samples showed a decrease in expected life with time, the slope of regression fatigue curves for control samples are steeper, indicating a large change in laboratory fatigue life with a given change in strain level.

Hveem stabilometer tests were conducted by the Oregon Department of Transportation personnel at their laboratory in Salem, Oregon. Hveem stability values increased over the test period for the control samples, but remained approximately constant for the rubber-modified mixtures. The rubber asphalt stability values would normally be considered unacceptable while for the control would be considered marginal based on current Asphalt Institute criteria for asphalt concrete mixtures. However, field surveys showed no rutting apparent in either mix.

A limited number of indirect tensile tests were conducted on both control and rubber-modified samples. Their results would indicate slightly greater tensile strengths for the conventional mixture. From the visual field surveys, Mays ride meter data showed the rubber-modified section to be slightly rougher. When tested dry, the control section has higher skid numbers than the rubber-modified section. Also, the rubber-modified test sections have slightly more macro-texture than the control mix in the limited number of testing presented.

FHWA project by Shuler, Pavlovich, and Epps (1985): This project used the method of Improvement Rating Scale (IRS) to evaluate field performances for specific distress types including rutting, raveling, flushing, corrugations, alligator cracking, longitudinal cracking, transverse cracking, and patching. Dense-graded RUMAC and open-graded RUMAC were tried. Open-graded RUMAC appeared to perform significantly worse than the others. Overall performances of dense-graded RUMAC (1% of finely ground rubber by mixture) were better, compared with control sections. Mixtures containing approximately 3 percent 1/4 inch minus ground rubber indicated no improvement on most sections.

Minnesota DOT experience (1986): The Minnesota DOT constructed two experimental test sections using a rubber-modified asphalt mix. Their experience showed that

- Surface roughness: rubber-modified mixture had a slightly lower serviceability.
- Surface deflection: difference in deflection was negligible.
- The ability of reducing the amount of snow and/or ice adhering to the pavement surface: rubber-modified test section performed better.

Minnesota (1996): Five crumb rubber modified (CRM) asphalt concrete plus two control test sections were placed in Babbitt, Minnesota in fall of 1993. A 2.38-mm (No.10) mesh crumb rubber from waste passenger tires was pretreated with a low-viscosity petroleum-based product and used as an aggregate replacement in asphalt concrete mixtures. Variables in the CRM sections included pretreated crumb rubber and CRM mixtures in just the wear course or throughout the 150-mm pavement section. Results showed that the pretreated CRM mixture sections have some potential for reducing thermal cracking. The permanent deformation characteristics showed that the control mixtures had the highest stiffness, followed by the pretreated and then the untreated CRM mixtures. Little difference in moisture sensitivity is anticipated for any of these mixtures.

Alaska DOT experience (1988): The Alaska DOT&PF installed 12 experimental pavement sections totaling 34.1 lane-mile in Fairbanks, Anchorage, and Juneau between 1979 and 1986 to assess the benefits of rubber-modified asphalt mixes. The key finding was the average reduction of 25% in stopping distances compared to conventional asphalt sections.

Caltrans experience (1989): Caltrans constructed dense-graded PlusRide AC overlay with and without SAMI. PlusRide sections showed adequate skid resistance and more resistance to surface abrasion. The surprisingly large percentage decrease in deflection as a result of a thin overlay was observed.

2.4. Mixture Design Method

The design of asphalt paving mixes is largely a matter of selecting and proportioning materials to obtain the desired properties in the finished construction. According to Asphalt Institute MS-2 (1993), the overall objective for the design of asphalt paving mixes is to determine an economical blend and gradation of aggregates and asphalt that yield mixes having the following desirable properties:

- Sufficient asphalt to ensure a durable pavement.
- Sufficient mix stability to satisfy the demands of traffic without distortion

or displacement.

- Sufficient voids in the total compacted mix to allow for a slight amount of additional compaction under traffic loading without flushing, bleeding, and loss of stability, yet low enough to keep out harmful air and moisture.
- Sufficient workability to permit efficient placement of the mix without segregation.

The Marshall and Hveem methods of mix design have been widely used with satisfactory results. For each method, criteria have been developed by correlating the results of laboratory tests on compacted paving mixes with the performance of the paving mixes under service conditions.

Previous research pertaining to mix designs for rubberized asphalt concrete have indicated necessary changes in compaction temperatures, flow limits, and air voids criteria (Stroup-Gardiner, 1989). Suggested compaction temperatures reported in the literature were 275 - 300^0 F (Chehovits, 1989; Texas A & M, 1986), 325 - 350^0 F (Vallerga, 1981), 375^0 F (Shuler, 1986). Crafco Inc. suggested increasing the flow limits to 24 for light traffic, 22 for medium traffic, and 20 for heavy traffic.

Criteria for acceptable air voids differed substantially. Crafco, Inc. suggested that the limits be tightened to 3 to 4 percent. The research program conducted by Texas A & M reported using air void of 7 percent as acceptable criteria.

Gary V. Gowda et al. (1996) compared the Marshall design method with the SHRP Level I design method with crumb rubber modified mixes. Three mix types, an unmodified hot-mix asphalt mix, a dry process rubber modified asphalt mix (1%, 2%, and 3%CRM in aggregate blend), and a wet process rubber modified mix (5%, 10%, and

15%CRM in A-R) were included in the investigation. It was found that the design binder content and the VMA were reduced for the SHRP Level I method relative to the Marshall method. It was also found that for aged mixes, incorporation of crumb rubber into hot mix asphalt concrete provided increased rutting resistance; however the rubber modified mixes did not show enhanced resilient and tensile properties when tested at 25°C. For unaged mixes, the dry process of incorporating CRM in the asphalt mixes tends to reduce the stiffness of the rutting mix compared to the wet process.

Ihab H. Hafez et al. (1995) made comparison of the Marshall and Superpave Level I mix design methods for asphalt mixes. Mix designs were conducted on totally 20 different mixtures categorized as (a) conventional, (b) wet process asphalt rubber (manufacture pre-blended), (c) dry process rubber asphalt, (d) polymer modified mixes, and (e) wet process asphalt rubber (plant-blended). The primary conclusion is that the Superpave gyratory Level I design can not be used to evaluate dry process rubber mixtures. The reasons for this are due to the high resilience of the rubber particles during the compaction process and the time dependent swelling, after compaction, of these mixtures.

Raghu Ram Madapati et al. investigated the feasibility of crumb rubber use for asphalt pavement construction in Rhode Island and compared the Marshall and Superpave Level I mix design methods. Both wet process (continuous blending, dense-graded and dense-graded friction course) and dry process (PlusRide, gap-graded) were included in the study. It was found that the optimum binder content determined by Superpave Level I mix design was lower than that determined by Marshall mix design in all cases with difference of 0.1 to 0.5 percent. The mechanical properties of mixtures with CRM did not show any significant trend, but, in a few cases, mixtures with CRM showed remarkable improvement over control mix.

2.4.1 Dense-Graded HMA Using Asphalt-Rubber

Dense-graded HMA using asphalt-rubber is composed of dense-graded aggregates and appropriate asphalt-rubber binder. Aggregate should meet the same quality requirements as for conventional hot mix asphalt concrete.

When using asphalt-rubber with less than 5 percent of fine CRM (- 50 mesh), traditional dense-graded aggregate gradations can be used. With higher CRM content in the asphalt-rubber binder, the aggregate gradation for dense-graded mixtures should be maintained on the coarse side of the gradation band. Gradations that plot between the maximum density line and the upper limit of the band should be avoided. Maintaining the gradation on the coarse side of typical dense-graded gradation band is important to provide sufficient void for the rubber particles. If the aggregate gradation is too fine or the rubber particles are too large, compaction problems can result. This is indicated by two problems that can occur during the mixture design procedure. First, immediately after compaction while the sample is still hot, the mixture will be somewhat unstable and "spongy" when coarse aggregate is pressed into the mix. Second, a relatively level trend is observed for the relationship between the air void of the mixture and the asphalt-rubber content, rather than a typical decreasing trend observed in the conventional hot mix. Both of these effects can generally be reduced or eliminated by using a coarser aggregate gradation or by reducing the rubber particle size. Suggested gradation limits for 3/8 inch, ½ inch, and 3/4 inch maximum sized dense-graded mixtures for use with high CRM content asphalt-rubber binders are shown in Table 2-11.

Table 2-11 Dense-graded HMA aggregate requirements (International Surfacing, Inc., 1992)

Sieve size	Gradation, percent passing		
	Max. size: 3/8"	Max. size: ½"	Max. size: 3/4"
1 in.	100	100	100
3/4 in.	100	100	90-100
½ in.	100	90-100	70-90
3/8 in	90-100	75-95	60-80
No. 4	60-80	50-70	40-60
No. 8	40-60	35-50	30-45
No. 30	18-30	15-18	12-22
No. 50	8-18	6-16	5-14
No. 200	2-8	2-8	2-6

When using high CRM content (10-25%) asphalt-rubber binder, it is recommended that the asphalt-rubber be heated to 350 ± 10^0 F, and the aggregate to 350 ± 10^0 F. The asphalt-rubber should be heated using an indirect method such as a forced-draft oven to maintain temperature and the asphalt-rubber should be stirred to assure uniformity immediately before adding to the heated aggregate. For asphalt-rubber binder with a low CRM content (less than 5%), mixing temperature can be more similar to those for the base asphalt cement.

Mixing of the asphalt-rubber with the aggregate should be performed using standard types of mechanical mixers such as wipes or paddles. Mixing should be performed immediately after addition of the asphalt-rubber to the aggregate and should continue for at least 30 seconds beyond the time required to obtain complete aggregate coating.

The recommended compaction temperature can be between 275 and 300^0 F. Specimen compaction consists of removing the specimen from the oven, placing it into heated Marshall molds, spading 15 times, and compacting using standard Marshall procedures.

Two modifications in design criteria should be used for AR dense-graded HMA. First, due to the increased viscosity, elasticity, and softening point of the asphalt-rubber, HMA mixtures tend to experience less compaction and densification from traffic after construction. Therefore, for dense-graded mixtures containing asphalt-rubber binder, the design air void level can be set at the lower end of the 3 to 5 percent range. The second modification is that maximum flow values can be raised to 24 for light traffic, 22 for medium traffic, and 20 for heavy traffic. This is due to the higher binder contents that are typically required and the flatter slope of the load versus deformation curve from the Marshall test.

Typical asphalt-rubber content for dense-graded HMA ranges from 6.0 to 7.5 percent by mixture weight. Generally, HMA mixtures containing high CRM content (10-25%) in the asphalt-rubber will increase VMA and flow, and decrease stability. For asphalt-rubber containing low CRM content (0-5%), the Marshall test results are typically similar to the conventional HMA.

2.4.2 Gap-graded HMA Using Asphalt-Rubber

Gap-graded HMA is a variation of dense-graded HMA in which the aggregate gradation is coarsened to provide a greater amount of coarse aggregate contact and to increase VMA to permit increased binder contents. Coarsening of aggregate gradation provides the needed space for the CRM particles and permits the use of larger sized CRM than with dense-graded mixtures.

Gap-graded HMA using asphalt-rubber with high CRM content has been found to offer improved performance (Cano et al., 1992). Suggested gradations for gap-graded HMA from International Surfacing, Inc. are shown in Table 2-12.

Table 2-12 Gap-graded HMA aggregate requirements
(International Surfacing, Inc., 1992)

Sieve size	Gradation, percent passing		
	Max. size: 3/8"	Max. size: ½"	Max. size: 3/4"
1 in.	100	100	100
3/4 in.	100	100	90-100
½ in.	100	90-100	65-85
3/8 in	78-92	70-90	50-70
No. 4	28-42	24-42	22-42
No. 8	15-25	15-25	15-25
No. 30	5-15	5-15	5-15
No. 200	3-7	3-7	3-7

Both Marshall and Hveem procedures can be used to design gap-graded HMA with asphalt-rubber binder. The modification of compaction and testing procedures should be the same as the modification for dense-graded mixtures as previously discussed. The design binder content should be chosen to satisfy 3 to 5 percent air voids and 20 percent minimum VMA. Both Marshall and Hveem stabilities are typically lower than conventional dense-graded mixtures by as much as 50 percent. Typical design asphalt-rubber binder contents range from 6.5 to 9.0 percent by mixture weight.

2.4.3 Open-Graded Friction Courses Using Asphalt-Rubber

Open-graded friction courses (OGFC) constructed with high quality aggregates have an outstanding capacity for providing and maintaining good frictional characteristics over the operating range of vehicle speeds on high speed highways. Their macrostructure facilitates drainage of water from the tire/pavement interface, improving tire contact with the pavement and reducing the potential for hydroplaning.

When compared to other types of surfaces, OGFC have demonstrated the following advantages:

- Provide and maintain good high speed, frictional qualities.
- Reduce the potential for hydroplaning.
- Reduce the amount of splash and spray.
- Provide a 3 to 5 decibel reduction in tire noise.
- Improve the wet weather, night visibility of painted pavement marking.
- Conserve high quality, polish resistant aggregates, which may be scarce in some areas, because they are placed only as a surface layer, up to 3/4 inch thick.

OGFC exhibit the following limitations:

- Increase the potential for stripping of the surface and underlying pavement (they do not seal the underlying pavement against moisture and air intrusion).

- Require special snow and ice removal methods and generally remain icy longer.
- Require special patching and rehabilitation techniques.
- Do not add structural value to the pavements (their performance is governed by the condition of underlying pavement).
- May ravel and shove when used at intersections, locations with heavy turning movements, ramp terminals, curbed sections, and other adverse geometric locations (Smith, NCHRP 180, 1992).

The modified physical properties of high CRM content asphalt-rubber permit its use in a variety of manners with OGFC. Due to the increased viscosity of asphalt-rubber, binder contents of up to approximately 10 percent can be used without experiencing excessive drain off. The higher binder contents produce thicker binder films which increase mixture aging resistance and durability.

Design procedures for OGFC (FHWA T-5040.31, 1990) generally consist of (1) selecting the aggregate gradation (Table 2-13), (2) determining the binder content, (3) evaluating mixture drainage versus temperature characteristics, and (4) determining moisture resistance properties. Details of the design procedures with asphalt-rubber binder will be discussed in Chapter IV.

US Army Corps of Engineers (USACE) and ARPG conducted a joint research project to determine the potential benefits of asphalt-rubber binders when used in porous friction courses (Anderton, 1992). The results of the study indicated that porous friction courses made with asphalt-rubber binders would be more durable, longer lasting, and have better water drainage when compared with unmodified asphalt cement porous friction courses.

Table 2-13 OGFC aggregate requirements for asphalt-rubber binder (International Surfacing, Inc., 1992)

Sieve size	Gradation, percent passing	
	Max. size: 3/8"	Max. size: ½"
3/4 in.	100	100
½ in.	100	95-100
3/8 in	85-100	75-95
No. 4	25-55	20-45
No. 8	5-15	5-15
No. 30	0-10	0-10
No. 200	0-5	0-5

Florida Department of Transportation (FDOT) used asphalt-rubber made with a low percentage of very fine ground tire rubber (minus No. 50 mesh) for both dense-graded (FC-4) and open-graded (FC-2) HMA. (Page et al., 1992).

CHAPTER III

BINDER TESTS AND RESULTS

In the late 1960's and early 1970's, the blending of ground tire rubber and paving asphalt yielded a patented modified binder which was reported to have an improved performance relative to the standard unmodified paving asphalts.

Although the use of asphalt-rubbers for paving applications is not new, only few studies have been reported on how these modified binders differ from the unmodified binders in general, and how the rubber particles change the properties of the base asphalt cements in particular.

This chapter provides the results of a series of tests used to quantify the rheological and mechanical properties of both unmodified and CRM (crumb rubber modifier) modified binders. These tests include the swelling test, viscosity test, thin film oven test, and dynamic shear rheometer test. Interpretation of these test results in light of the SHRP binder specifications is presented in this chapter as well.

3.1 Description of Materials Used

3.1.1 Asphalt Cement

For this study, AC-10 (used only for swelling tests) and AC-20 obtained from the North Star Asphalt Co. in North Canton, Ohio and AC-5, AC-10, and AC-20 supplied by

Ashland Petroleum Co. in Canton, Ohio were used. Ecoflex, a proprietary rubberized asphalt manufactured by BITUMAR Inc. Quebec, Canada was also studied. Detailed specifications of Ecoflex binder is given in Table 3-1.

3.1.2 CRM

The majority of CRM used in this study were obtained from Baker Rubber Inc. (BRI) in Chambersberg, PA. Different sizes of CRM were available directly from the supplier, including WRF 1/4", WRF 10, and WRF 30. Product description of BRI's CRM is shown in Table 3-2. An ultra fine CRM was obtained from Goodyear Tire Rubber Co. in Cleveland, Ohio. The results of sieve analysis of each CRM used in this study are shown in Table 3-3.

Table 3-1 Ecoflex rubberized asphalt specification

Test Requirement	Min.	Max.	Typical
Viscosity, 275°F	500		630
Penetration, 77°C	70	100	85
% penetration	50		57
Flash point COC,°F	450		520
Softening point,°F	113	130	120
Solubility in Trichloroethylene, %	96		97
Ductility, 4°C	6		7.5
P.V.N	0		0.3
Storage stability, 48hrs	+/- 2°F		+/-1°F
Test on residue from thin Film Oven Test			
Loss on heating, %		0.8	0.2
Viscosity, 275°F		2000	1100
Ductility	3		3.8
Penetration % of origin	55		66

Table 3-2 Product description of BRI GRANULITE® ground rubber

Product Analysis	Minimum	Maximum
Acetone Extract	0.1	19.0 %
Ash Content	-	8.0 %
Carbon Black	0.28	39.0 %
Moisture Content	-	0.75 %
Rubber Hydrocarbon	0.45	-
Free Iron Content	0.0002	
Free Fabric Content	0.0005	
Bulk Density (typical)	100 gram/210 cc	

Table 3-3 CRM gradations and suggested rubber gradation for different graded HMA

Sieve Number	Gradation, percent passing (%)				
	WRF10	WRF30	Goodyear	dense-graded	open-graded
No. 10	100	100	100	100	100
No. 16	76.3	100	100	98-100	75-100
No. 30	22.9	95.72	98.6	70-100	25-60
No. 50	5.2	18.26	59.2	10-40	0-20
No. 200	0.3	1	1.6	0-5	0-5

3.2 Laboratory Test Program

Test program adopted in this study can be categorized into two types: one is asphalt-rubber binder tests, the other one is mix design and mechanical properties characterization tests. The procedures and equipments for various tests are discussed for each category. Table 3-4 provides a summary of the tests performed in this study.

Table 3-4 Types of tests performed in study

Asphalt-rubber binder tests	- Swelling test - Viscosity test - Aging tests * TFOT (short-term) - Dynamic shear rheometer test
Mix design tests	Marshall mix design - Wet process * dense-graded w/ AC-20 * dense-graded w/ AR * dense-graded w/ Florida process * dense-graded w/ Ecoflex * gap-graded w/ AR - Dry process * dense-graded RUMAC * gap-graded RUMAC Open graded mix design
Mix property tests	- Indirect tensile strength test * Unaged * Short-term aged * Long-term aged - Resilient modulus test * Unaged * Short-term aged * Long-term aged - Fatigue beam test - Thermal Stress Restrained Specimen Test (TSRST) - Incremental Creep Test - Water sensitivity Test - Loaded Wheel Track Test

3.3 Asphalt-Rubber Binder Tests

3.3.1 Swelling Test

When CRM is immersed in liquid asphalt cement, the rubber particles generally experience volumetric swelling. The extent and rate of volumetric swelling can be related to factors such as temperature of the asphalt cement, time of immersion, and physical and compositional characteristics of asphalt cement and rubber. Tolonen and Green (1977) provided some detailed study of the swelling process. As the rubber particles swell, the interparticle distance between them was reduced, which resulted in an increase in the viscosity of the blend. Tolonen and Green also determined that rubber swelled due to absorption of the oil or asphalt fluid that was immersed in and that different oils were absorbed to greater or lesser degrees.

To quantify changes that occur to the rubber during swelling process, two different sizes of rubber specimens were immersed in asphalt at a constant elevated temperature. Weight change of the specimens were calculated in order to quantify the absorption of light oil components from asphalt into rubber specimens.

Swelling Test Simulating the Wet Process

In order to simulate the reaction between CRM and asphalt cement in an elevated temperature, specimens of rubber cut from a scraped tire were immersed in asphalt cement at a constant elevated temperature. The specimen size used included 0.5" x 0.5" x 0.2" (designated as Small specimen) and 1" x 1" x 0.2" (designated as Large specimen). Two temperatures were adopted, including 225^0 F and 250^0 F. The volumetric swelling of the rubber specimens was represented by the amount of weight increase in reference to the initial dry weight of the rubber specimen. The time duration of the swelling test lasted for more than 120 hours.

Swelling Test Simulating the Dry Process

In order to simulating dry process, asphalt cement and rubber specimens were heated separately up to 300^0 F. Then, the rubber specimens were inserted into the asphalt cement, while the temperature gradually dropped to atmosphere room temperature. The cooling down process took about 30 minutes. The total amount of swelling at the end of cooling down period was measured by the weight increase, in comparison to the initial dry weight of the specimen. The specimens size used is the same as in the wet process.

3.3.2 Swelling Test Results

Wet Process

The results of swelling test simulating the wet process are shown in Fig. 3-1, where the y-axis represents the percent of weight increase and the x-axis represents the time of immersion (i.e., the reaction time). As can be seen, given the same amount of the reaction time, the higher the reaction temperature, the larger amount of weight increase. There seemed to be a limit to the final amount of weight increase for a given reaction temperature. This limit was reached faster at lower reaction temperature than that at higher reaction temperature. For the two specimen sizes investigated, there seemed to have some, albeit small, difference in the rate and amount of weight increase.

A curve fitting technique using Eq. 3-1 was performed on the measured data point to quantitatively estimate the maximum amount of weight increase.

$$\ln\left[\frac{S_m}{S_m - S}\right] = k\,t - \ln C \tag{3-1}$$

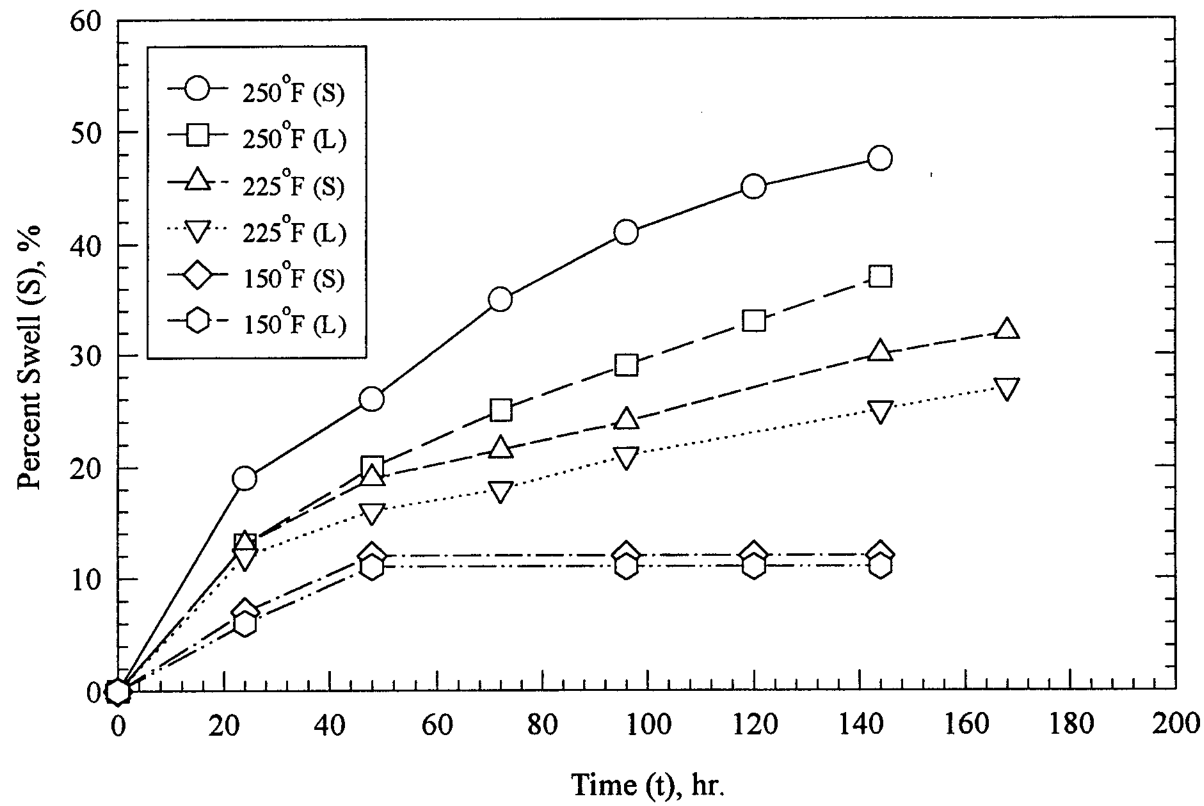


Fig. 3-1 Measured percent swell (S) versus time (t) for different temperatures and sizes

where S_m is the maximum weight increase in percent of initial dry weight, S is the percent of weight increase at any time t after immersion, and both k and C are curve fitting constants.

As shown in Figs. 3-2 to Fig. 3-5, the measured data can be perfectly fitted by Eq. 3-1. The corresponding curve fitting constants are tabulated in Table 3-5.

Table 3-5 Results of curve-fitting analysis for maximum percent swell

S_m, max % swell	k	C	temp/size
45	0.01005	1.08	250°F/large
60	0.0105	1.118	250°F/small
50	0.3571	1.21	225°F/large
45	0.6251	1.21	225°F/small

The curve fitting constant k can be interpreted as the rate of weight increase; the larger the value of k, the slower the weight increase process. As can be seen from Table 3-5, the reaction temperature seems to play a significant role in affecting the rate of weight increase. However, the projected maximum percent weight increase seemed to be around 50 percent, for the temperature range investigated.

<u>Dry Process</u>

As discussed in the previous section, the reaction between asphalt cement and rubber in the dry process was somewhat limited. This was due to a rapid temperature cool down during post-construction (lay-down) period. The percent weight increase was determined to be 1.41% for the small rubber specimens and 1.93% for the large rubber

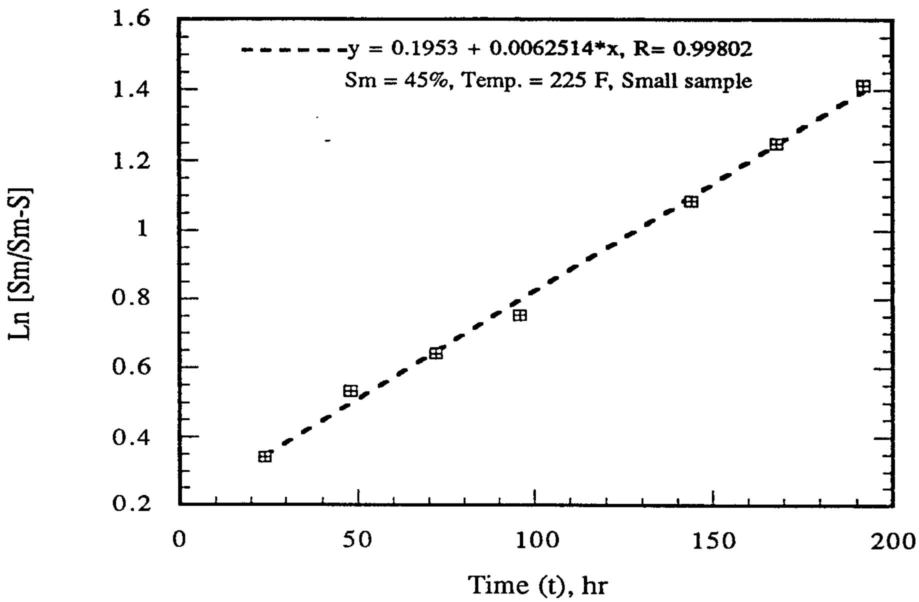


Fig. 3-2 ln[Sm/Sm-S] versus time at 225°F, small sample

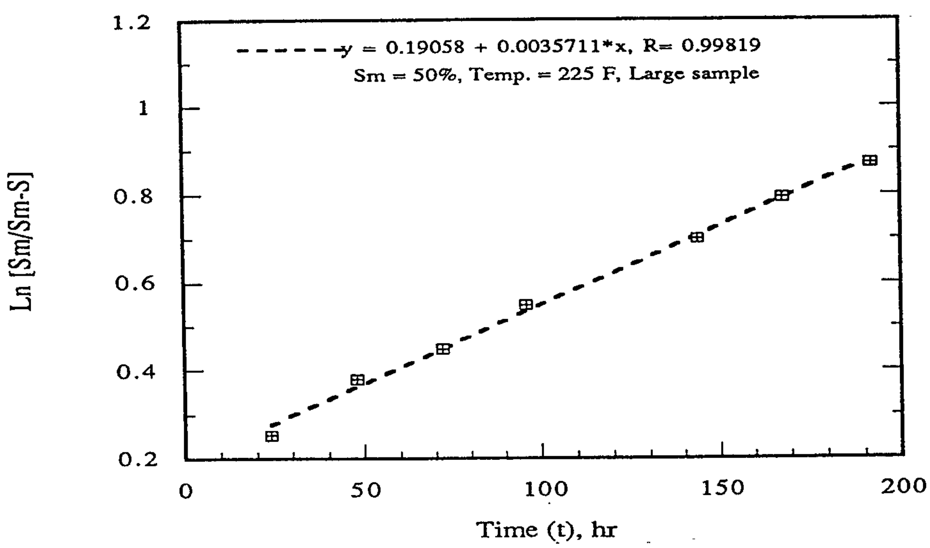


Fig. 3-3 ln[Sm/Sm-S] versus time at 225°F, large sample

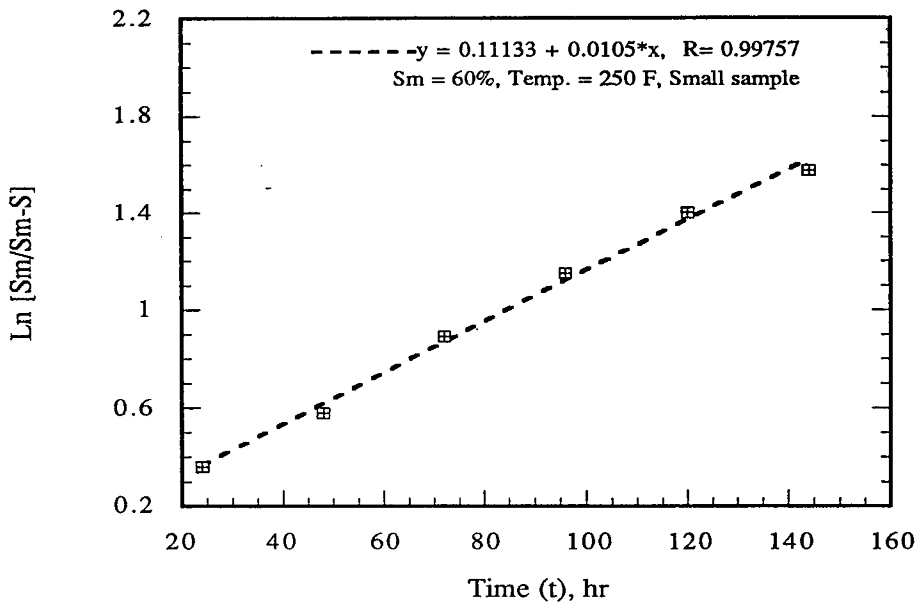


Fig. 3-4 ln[Sm/Sm-S] versus time at 250^0 F, small sample

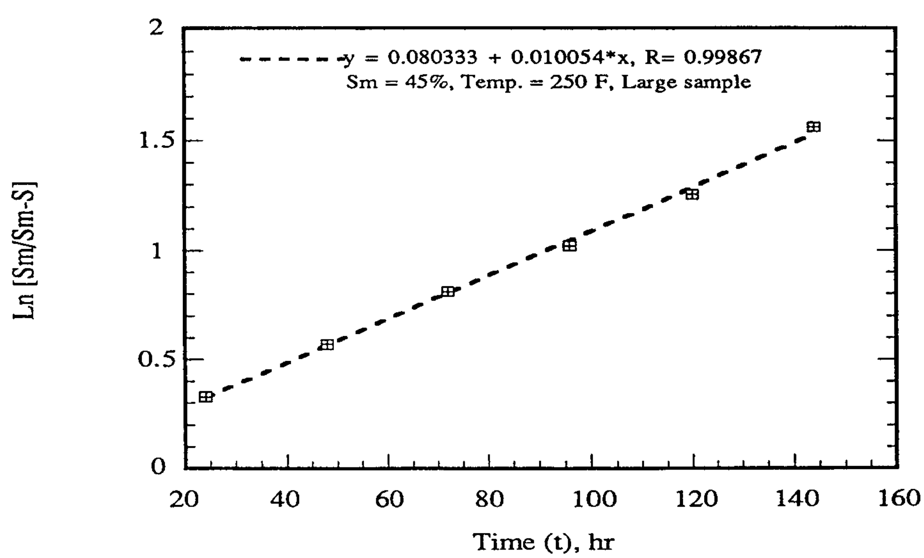


Fig. 3-5 ln[Sm/Sm-S] versus time at 250^0 F, large sample specimen, both were measured after 30 minutes of cooling period.

3.4 Rheological Properties

Rheology is the study of material whose deformation characteristics vary not only with load but also with the time rate of load application. Asphalt-rubber, like many engineering materials such as soil, polymer, and concrete, are rheological materials; that is, their stress versus strain characteristics are time dependent.

The current method to characterize the rheological properties of asphalt cement can be done by either penetration or viscosity test. Although viscosity is a fundamental measure of flow, it only provides information about high temperature (60^0 C and 135^0 C) viscous behavior; it does not relate to the low temperature elastic behavior needed to completely predict the performance. Penetration describes only the consistency at a medium temperature, 25^0 C. No low temperature properties can be directly measured in the current grading system.

In 1987, the SHRP began developing new tests for measuring physical properties of asphalt. One result of this research effort was a new asphalt specification with a new set of tests. The document was called a binder specification because it was intended to function equally well for modified as well as unmodified asphalt. The final product of SHRP asphalt research program is a new system referred to as Superpave, which stands for Superior Performing Asphalt Pavements. Superpave software is a computer program that assists engineers in materials selection and mix design. However, the term "Superpave" refers to more than just the computer program. Most importantly, it represents an improved system for specifying component materials, asphalt mixture design and analysis, and pavement performance prediction. Table 3-6 and Fig. 3-6 summarize the Superpave asphalt binder test and their intended purpose. Fig. 3-6 shows schematically the applicable temperature range for each Superpave binder test.

Table 3-6 Superpave asphalt binder product

Superpave asphalt binder tests	Purpose
Dynamic Shear Rheometer	Measure properties at high and intermediate temperatures
Rotational Viscometer	Measure properties at high temperatures
Bending Beam Rheometer Direct Tension Tester	Measure properties at low temperature
Rolling Thin Film Oven Pressure Aging Vessel	Simulating hardening (durability) characteristic

3.5 Effect of Aging

3.5.1 Mechanism of Binder Aging

Durability of an asphalt is considered to be its resistance to change in properties when subjected to processing (e.g. mixing with aggregate) and weathering and is manifested primarily by a resistance to hardening with time. A number of factors contributed to this hardening with time process (aging process); These are: (1) oxidation, (2) volatilization, (3) polymerization, (4) rheopexy, (5) separation, and (6) syneresis. Oxidation is the reaction of oxygen with asphalt, the rate of which depends upon the character of the asphalt and the temperature. At normal temperatures, the reaction of

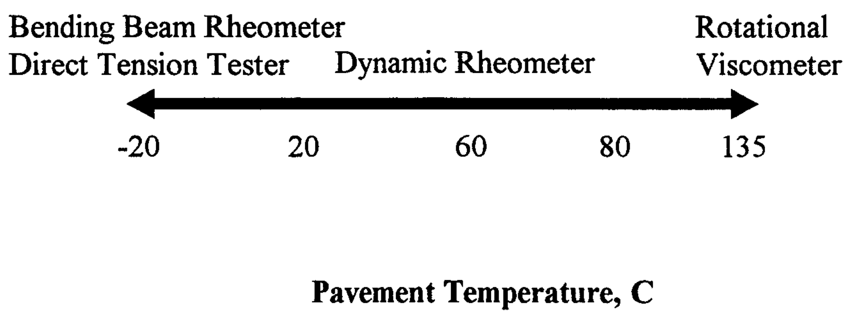


Fig. 3-6 Superpave asphalt binder specification with temperature coverage

oxygen with the asphalt is a slow process and the oxygen is primarily absorbed by the surface of the asphalt which, in undisturbed state, will prevent the further reaction of oxygen with the material. If the film is cracked, however, new surfaces of asphalt will be exposed which in turn will permit additional oxidation to occur. This is believed to be one of the primary causes of hardening of the asphalt in the road.

Oxidation of asphalt at temperatures associated with the mixing of asphalt cements and aggregates is rapid. The amount of hardening that occurs during the hot mix process is a function not only of temperature but also the thickness of the asphalt film, time of exposure, and the type of atmosphere present (oxygen rich or oxygen depleted).

Volatilization is the evaporation of the light constituents from the asphalt which is a mixture of hydrocarbons with a large range in molecular weight. Increasing temperature will accelerate this phenomenon and it is thought that in the mixing process, where high temperature is combined with violent agitation, this is one of the primary reasons for hardening of the asphalt.

Polymerization is a combining of like molecules to form larger molecules. In the case of asphalts, it is the combining of small molecular weight hydrocarbons into larger molecular weight hydrocarbons. Considerable evidence has been presented in recent years to indicate that the resins are the ones most susceptible to change and that the polymerization process involves the conversion of resins to asphaltene with only minor changes occurring in the oily fractions. This would explain, for example, the fact that weathered asphalts are more non-Newtonian in behavior than corresponding unweathered asphalts. This change can be detrimental since it tends to make the asphalts more brittle and, therefore, the and, therefore, the pavement more susceptible to cracking.

Rheopexy phenomenon, as it occurred in remolded quick clay, has been offered as an explanation for the more rapid hardening of untrafficked areas as compared to the same pavement under loading. Asphalts, when allowed to remain in an unloaded condition, may increase in viscosity in time. It should note that this apparent increase in viscosity can be completely eliminated either through large shearing deformations or heat or a combination of both. This change is termed thixotropy. In case of air-blown asphalt, again where the asphaltness are poorly dissolved in the oil-resin phase, a structure will gradually develop with time which results in an increase in viscosity. Either large shearing deformations or heat will tend to destroy this structure and the asphalt will again exhibit the same viscosity as it had before.

Separation is a term used to describe the removal of oily constituents or resins or asphaltenes from the asphalt as caused by the selective absorption of some aggregates on which an asphalt film has been placed. This action may result in a hardening or softening of the asphalt film.

Syneresis is an exudation reaction occurring in asphalts in which, due to the formation of a structure within the asphalt, a thin oily liquid containing either dispersed or dissolved intermediate and heavier bodies is exuded to the surface. With the elimination of some of the lighter oils, the asphalt will progressively harden with time. This reaction in its less extreme form is referred to as the straining tendency of asphalt.

3.5.2 Test Method for Aging

In general, regardless of the mechanism, the hardening of asphalt can lead to reduced service life. This is brought about due, among other factors, to the binder becoming brittle, reduction in adhesive characteristics, and possible loss in asphalt, since the oxidation products are water soluble. Of the reactions discussed above, volatilization and oxidation appear to be the most important factors and are also the two over which the engineer has the most control.

As can be seen from Table 3-6, The Superpave binder specification provided two aging tests; one is RTFOT as a short-term aging device, which represents aging during mixing. The other one is PAV test as a long-term aging device, which represents aging after construction.

Thin film oven test (TFOT; ASTM D 1754) is used in stead of RTFOT for short-term aging test in this study. The TFOT is intended for the determination of the effect of heat and air on a film of semisolid asphaltic material. About 50 grams of asphalt cement sample in the pan is heated in an oven for 5 hours at 325° F. The effects of heat and air are determined from changes occurring in physical properties measured before and after the oven treatment.

3.6 Viscosity Test

Rotational viscosity is used to evaluate high temperature workability of binders. A rotational coaxial cylinder viscometer, such as the Brookfield apparatus is used rather than a capillary viscometer. The rotational viscosity is determined by measuring the torque required to maintain a constant rotational speed of a cylindrical spindle while submerged in a sample at a constant temperature. The torque required to rotate the spindle at a constant speed is directly related to the viscosity of the binder. A schematic drawing of Brookfield Viscometer and Thermosel system is illustrated in Fig. 3-7. This method of measuring viscosity is detailed in ASTM D 4402,"Viscosity Determination of Unfilled Asphalts Using the Brookfield Thermosel Apparatus."

The purpose of viscosity test on binders can be summarized as follows:

1. Since the viscosity is defined as a coefficient of the shear stress to the rate of shear, the measurement of viscosity will provide insight on the rheological behavior of asphalt-rubber, that is either Newtonian (linear relationship between shear stress and rate of shear) or non-Newtonian (either shear thinning or shear thickening).
2. The viscosity measured on unaged or tank asphalt must not, according to the Superpave binder specification, exceed 3 Pa.s (3000 cp) when measured at 135^0 C (275^0 F).
3. The viscosity measurement before and after aging test (TFOT and/or PAV) will give the effect of aging due to oxidation.
4. The viscosity measurements in a wide range of temperature will show the temperature susceptibility of each asphalt-rubber binder.
5. FHWA (Heitzman, 1992) recommended that the viscosity of asphalt-rubber binder at 350°F should lie between 1000 cp and 4000 cp. Therefore, the viscosity measurement at this temperature can be used for quality control.
6. The viscosity measurements of asphalt-rubber binder during reaction period can provide quantitative indication of the extent of reaction between asphalt and rubber.

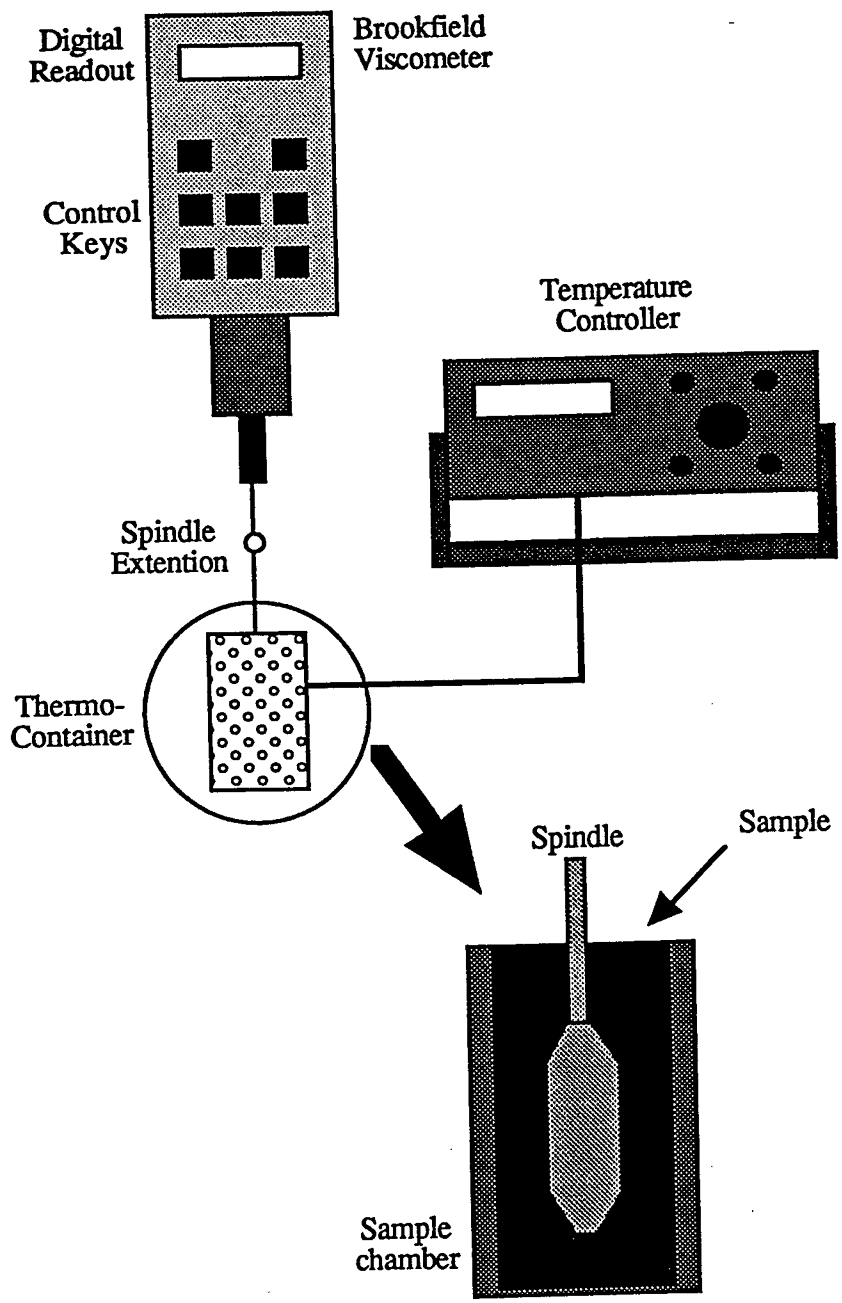


Fig. 3-7 Brookfield viscometer and thermosel system

<u>Procedures of Viscosity Test</u>

Viscosity test was performed using the Brookfield Digital Viscometer Model DV-II Version 2.0 with a Thermosel temperature control system. The test procedure can be described as follows.

- Preheat the thermo-container at a specified temperature using the temperature controller.
- Prepare a specific volume of binder and put it into the sample chamber. (Volume of binder is dependent on the type of spindle used.)
- Wait for about one hour until sample temperature reaches to the specified temperature.
- Lower the viscometer and align the thermo-container.
- Insert the spindle into the binder in the chamber and couple it to the viscometer. Check that the binder level on the spindle shaft is 1/8 inch above the upper conical interface.
- Put the insulating cap on the top of the thermo-container.
- Operate the viscometer with specified shear rate and record the viscosity.

3.6.1 Results of Viscosity Test

A total of 18 different asphalt-rubbers, consisting of a combination of either AC-5 or AC-10 with three different CRM types and various contents, were tested to quantify the reaction between asphalt cement and CRM. Table 3-7 and 3-8 show combinations of asphalt and rubber and their viscosities after two hours reaction period at 375^0 F. Table 3-

9 gives the viscosity measured by Brookfield DV-II+viscometer with thermosel using spindle SC4-27 for asphalt-rubber (AC10 base).

Table 3-7 Viscosity of asphalt-rubbers (AC-5 base) with different combination

Asphalt-Rubber	AC type	CRM type	CRM content, %	Viscosity
AC5/10% W10	AC-5	WRF 10	10	75 cp
AC5/15% W10	AC-5	WRF 10	15	367 cp
AC5/20% W10	AC-5	WRF 10	20	2100 cp
AC5/10% W30	AC-5	WRF 30	10	167 cp
AC5/15% W30	AC-5	WRF 30	15	2050 cp
AC5/20% W30	AC-5	WRF 30	20	3200 cp
AC5/10% GY	AC-5	Goodyear	10	167 cp
AC5/15% GY	AC-5	Goodyear	15	1930 cp
AC5/20% GY	AC-5	Goodyear	20	4056 cp

Table 3-8 Viscosity of asphalt-rubbers (AC-10 base) with different combination

Asphalt-Rubber	AC type	CRM type	CRM content, %	Viscosity
AC10/10% W10	AC-10	WRF 10	10	275 cp
AC10/15% W10	AC-10	WRF 10	15	550 cp
AC10/20% W10	AC-10	WRF 10	20	1850 cp
AC10/10% W30	AC-10	WRF 30	10	483 cp
AC10/15% W30	AC-10	WRF 30	15	1408 cp
AC10/20% W30	AC-10	WRF 30	20	4000 cp
AC10/10% GY	AC-10	Goodyear	10	400 cp
AC10/15% GY	AC-10	Goodyear	15	2116 cp
AC10/20% GY	AC-10	Goodyear	20	6500 cp

* Viscosity was measured after 2 hr. reaction.
* Brookfield DV-II+ viscometer with RV3 or RV4 spindle and 12 RPM were used.
* Reaction temperature = 375°F. Viscosity was measured at 350°F.

Table 3-9 Viscosity of asphalt-rubber (AC10 base) using spindle SC4-27

Binder	CRM type	CRM content %	Viscosity at 350^{0}F (cp)
AC10+10%WRF10	WRF10	10	275
AC10+15%WRF10	WRF10	15	550
AC10+20%WRF10	WRF10	20	1600
AC10+5%WRF30	WRF30	5	94
AC10+10%WRF30	WRF30	10	125
AC10+15%WRF30	WRF30	15	427
AC10+20%WRF30	WRF30	20	3875
AC10+10%GY	Goodyear	10	166.7

Fig. 3-8 and Fig. 3-9 are prepared to demonstrate typical differences of the development of viscosity in different sizes of CRM. Fig. 3-8, which represents reaction behavior of finer (WRF 30) CRM, shows that viscosity quickly reaches the maximum viscosity at the beginning of the reaction period. Thereafter, the viscosity is slowly decreased. However, Fig 3-9, which represents reaction behavior of coarser (WRF 10) CRM, shows that the viscosity continuously increases even after 24 hours reaction period.

Fig. 3-10 shows typical reaction curves for five different CRM contents at 350^0 F. As can be seen, reaction curve for 25% of WRF 10 asphalt-rubber shows the highest viscosity.

For convenience of comparison, the viscosities of asphalt-rubbers after 2 hours reaction summarized in Table 3-7, Table 3-8, and Table 3-9 are plotted in Figs. 3-11 through 3-13, which show the viscosity of AC-5 and AC-10 based asphalt-rubbers, respectively, with three different CRM contents (10%, 15%, and 20%) and three different CRM types (WRF 10, WRF 30, and Goodyear's Ultrafine). These graphs can be used for

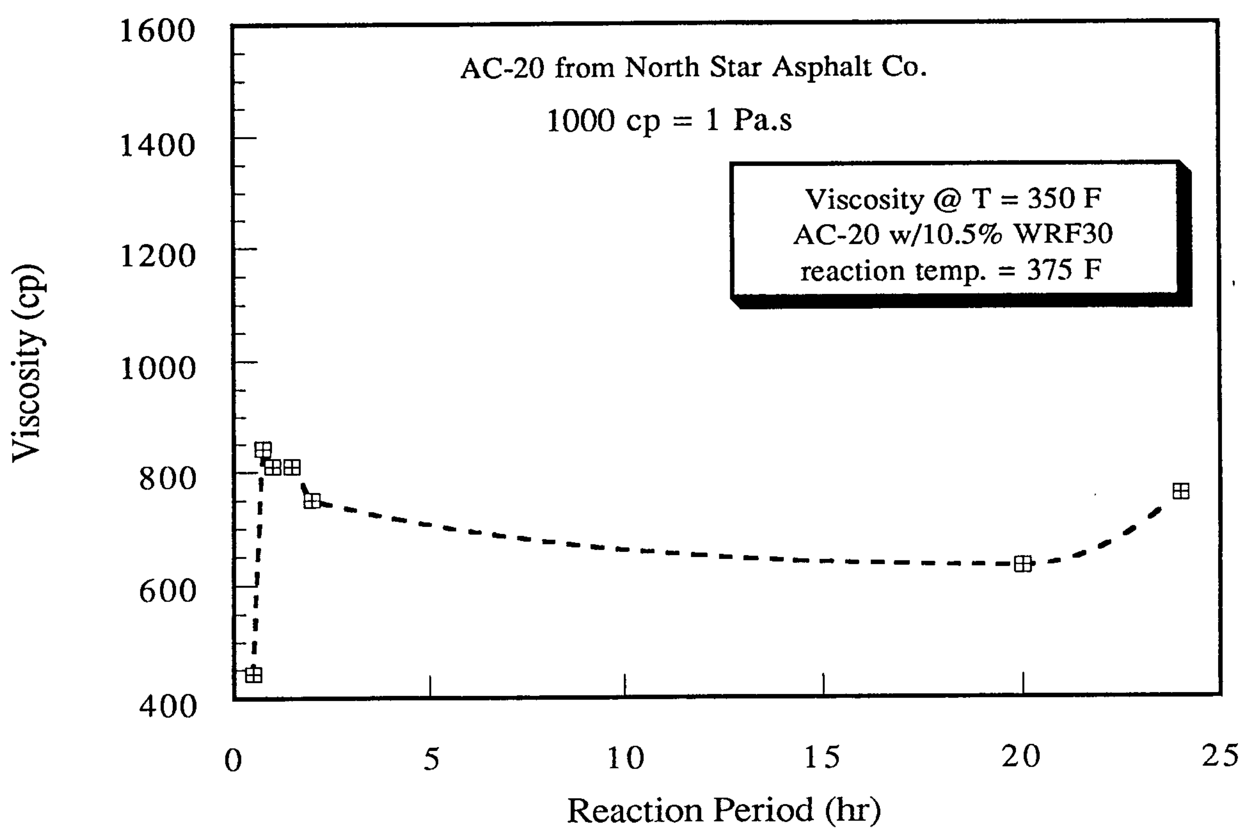


Fig. 3-8 Viscosity versus reaction time for finer CRM (WRF 30) at 350^0 F

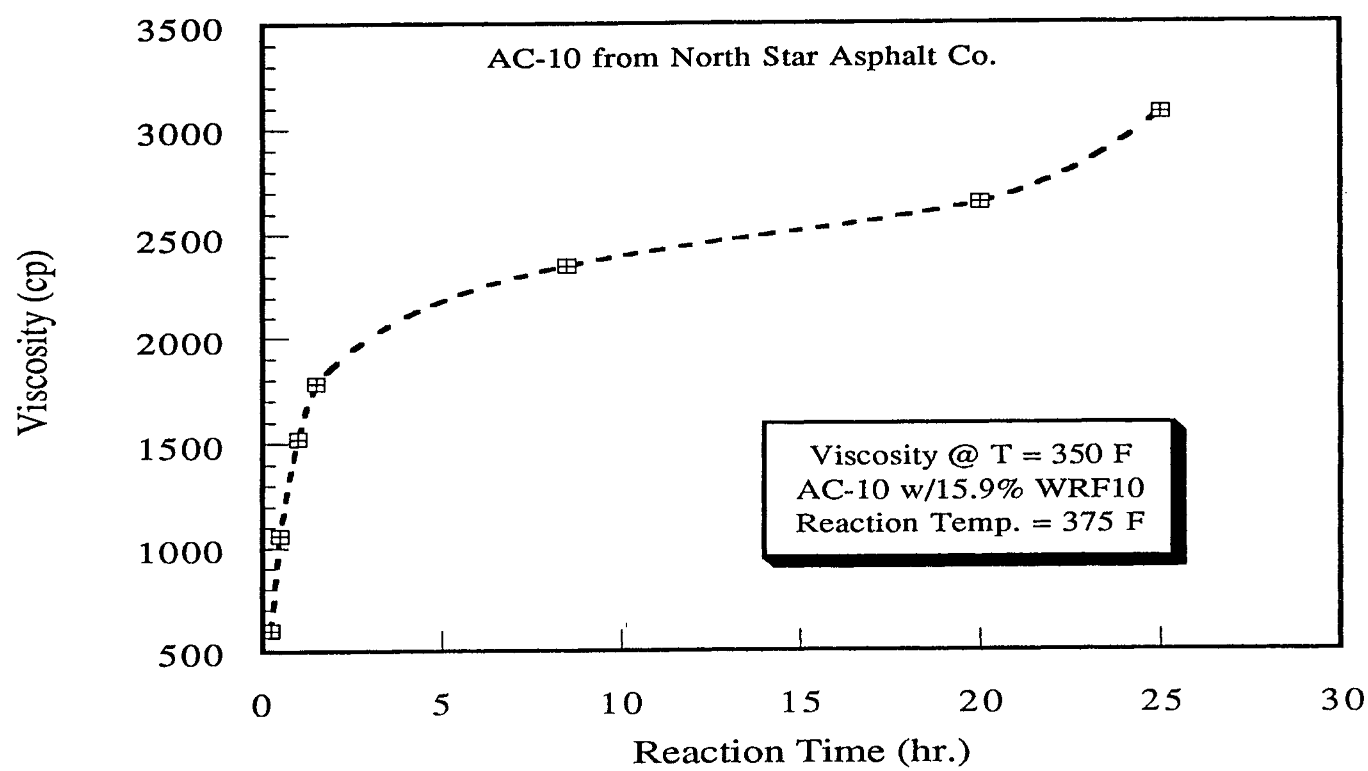


Fig. 3-9 Viscosity versus reaction time for coarser CRM (WRF 10) at 350^0 F

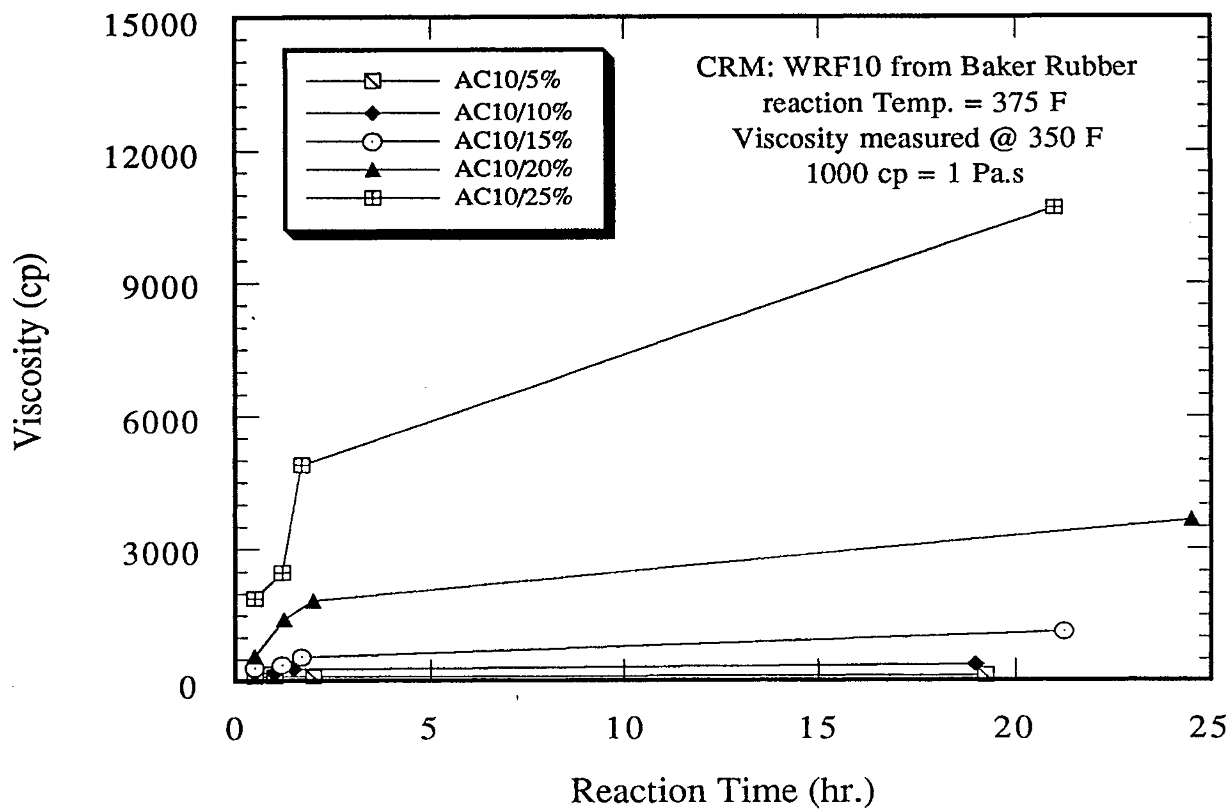


Fig. 3-10 Viscosity of asphalt-rubber (AC-10 base) versus reaction time at 350^0 F

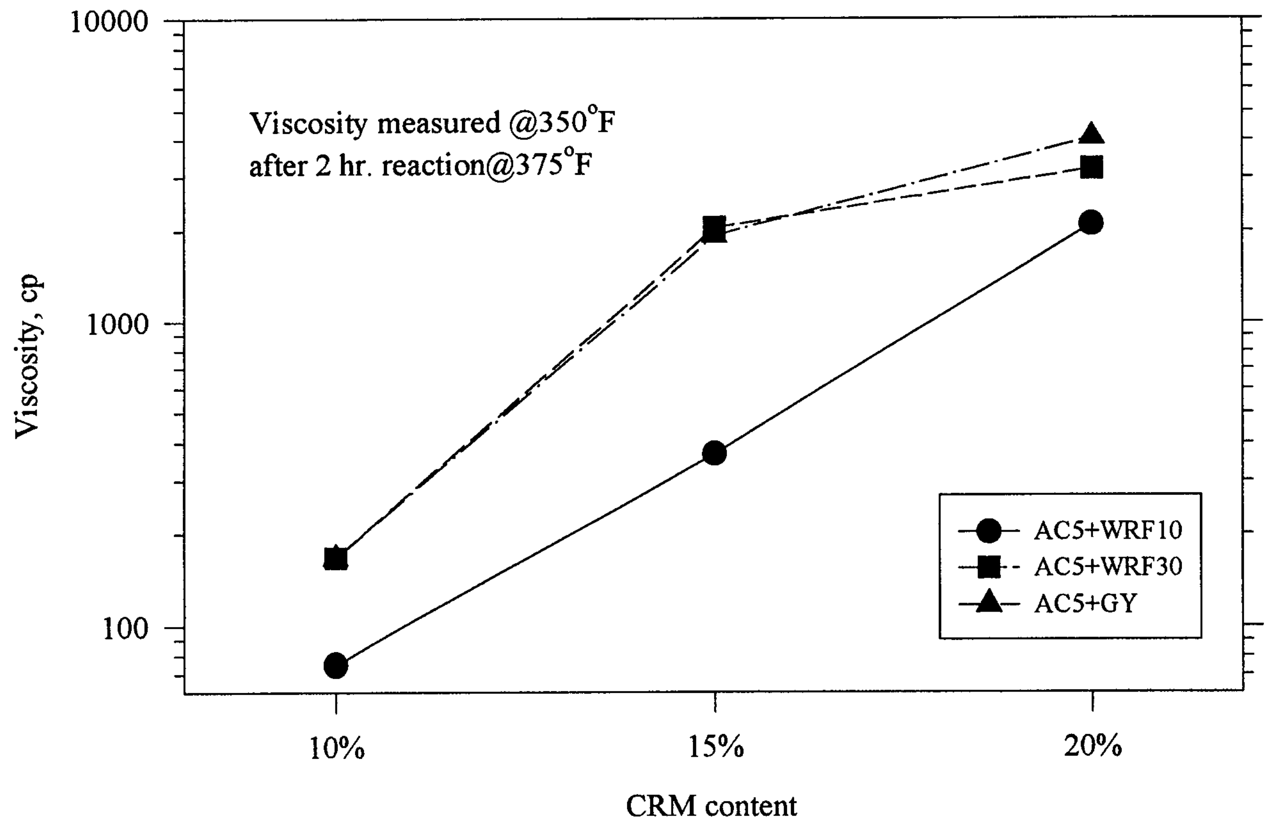


Fig. 3-11 Viscosity of asphalt-rubbers (AC5 base) after 2 hour reaction at 375°F

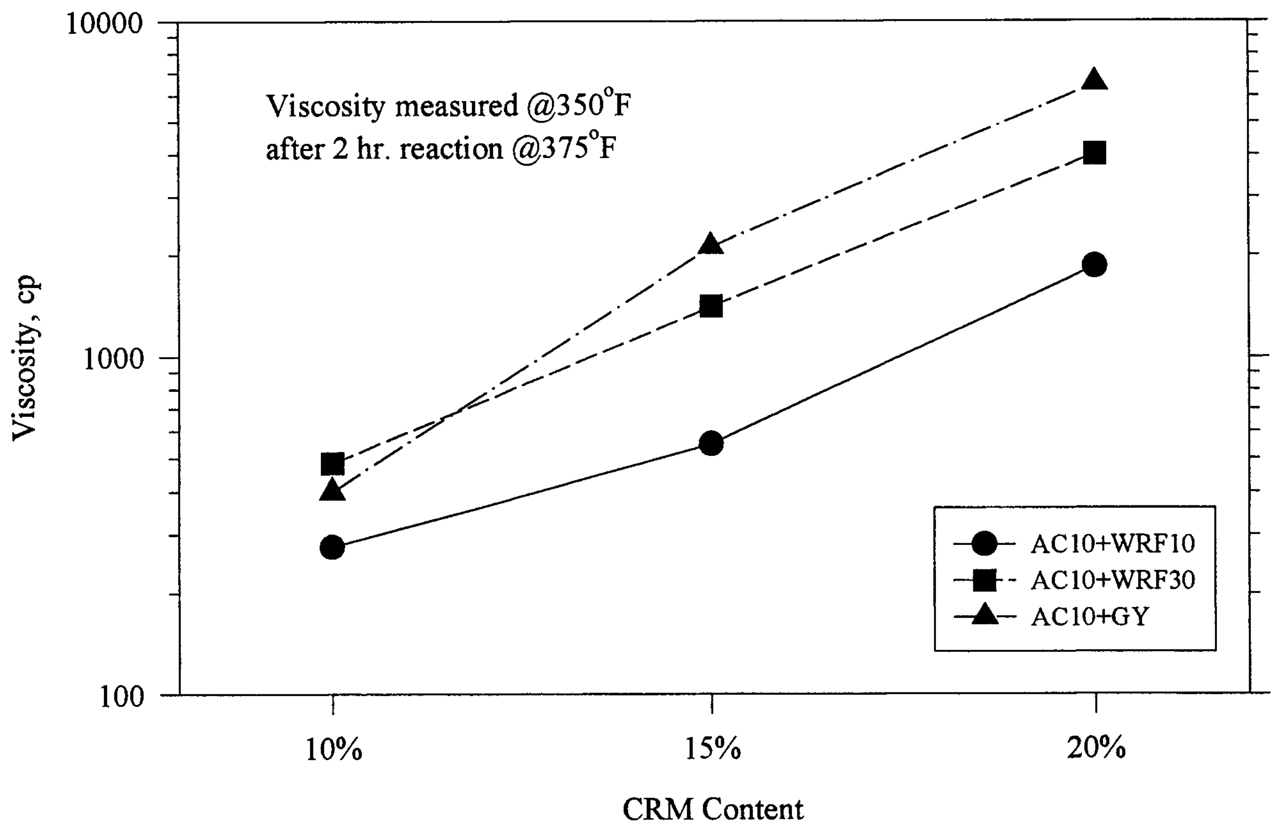


Fig. 3-12 Viscosity of asphalt-rubbers (AC10 base) after 2 hour reaction at 375°F

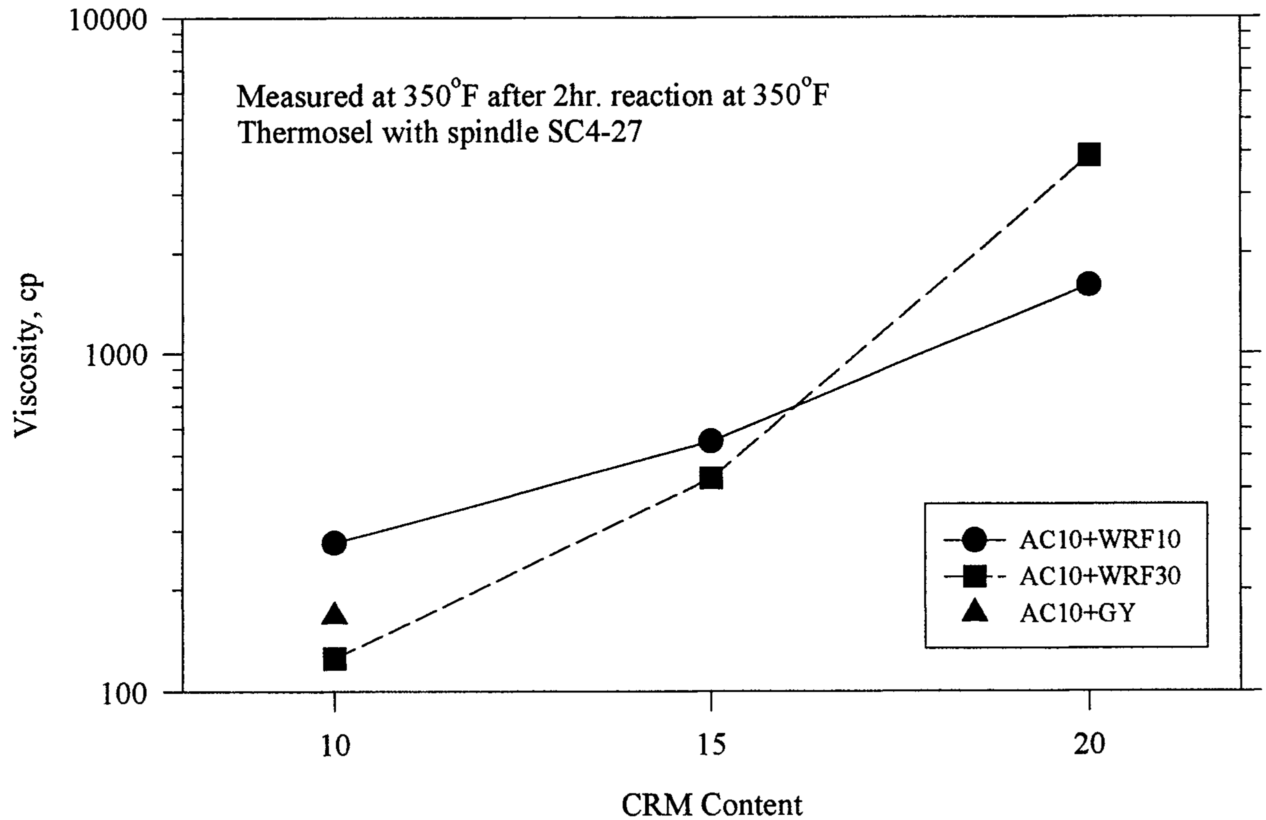


Fig. 3-13 Viscosity of asphalt-rubbers (AC10 base) with thermosel after 2 hr. reaction at 350°F

screening purpose in order to choose asphalt-rubbers which satisfy FHWA recommendation.

Fig. 3-14 represents the temperature susceptibility of asphalt-rubbers (AC-10 based) in the range of 200^0 F to 300^0 F. It can be observed that the slopes of viscosity versus temperature for all asphalt-rubbers are flatter than those of AC-5, AC-10, and Ecoflex.

The viscosity was measured at different shear rates for investigating the effect of shear rate on the measured asphalt-rubber viscosity. The shear rate of 1, 5, 12, 50 and 100 rpm was performed. Non-Newtonian behavior of unmodified AC-10 and asphalt-rubber of AC-10 with 20 percent of WRF 10 at various temperatures are shown in Figs. 3-15 and 3-16, respectively. The unmodified AC-10 asphalt shows mild shear-thinning behavior in all temperature ranges, while the asphalt-rubber with 20 percent of WRF 10 shows pronounced non-Newtonian (shear thinning) behavior at low temperature range (225^0 F and 250^0 F). Fig. 3-17 presents non-Newtonian behavior of unmodified AC-5 and AC-5 based asphalt-rubbers with 15 and 20 percent of WRF 30 at the temperature of 275°F. Fig. 3-18 presents the non-Newtonian behavior of unmodified AC10 and AC10 based asphalt-rubbers with 10, 15, and 20 percent of WRF30 at the temperature of 350°F.

3.6.2 Result of Thin Film Oven Test

AC-5, AC-10, and AC-10 based asphalt-rubbers with three different content of WRF 10 were prepared for TFOT. Viscosity at 275^0 F and weight of sample were recorded before and after the test. The results of viscosity tests and the ratio of viscosity

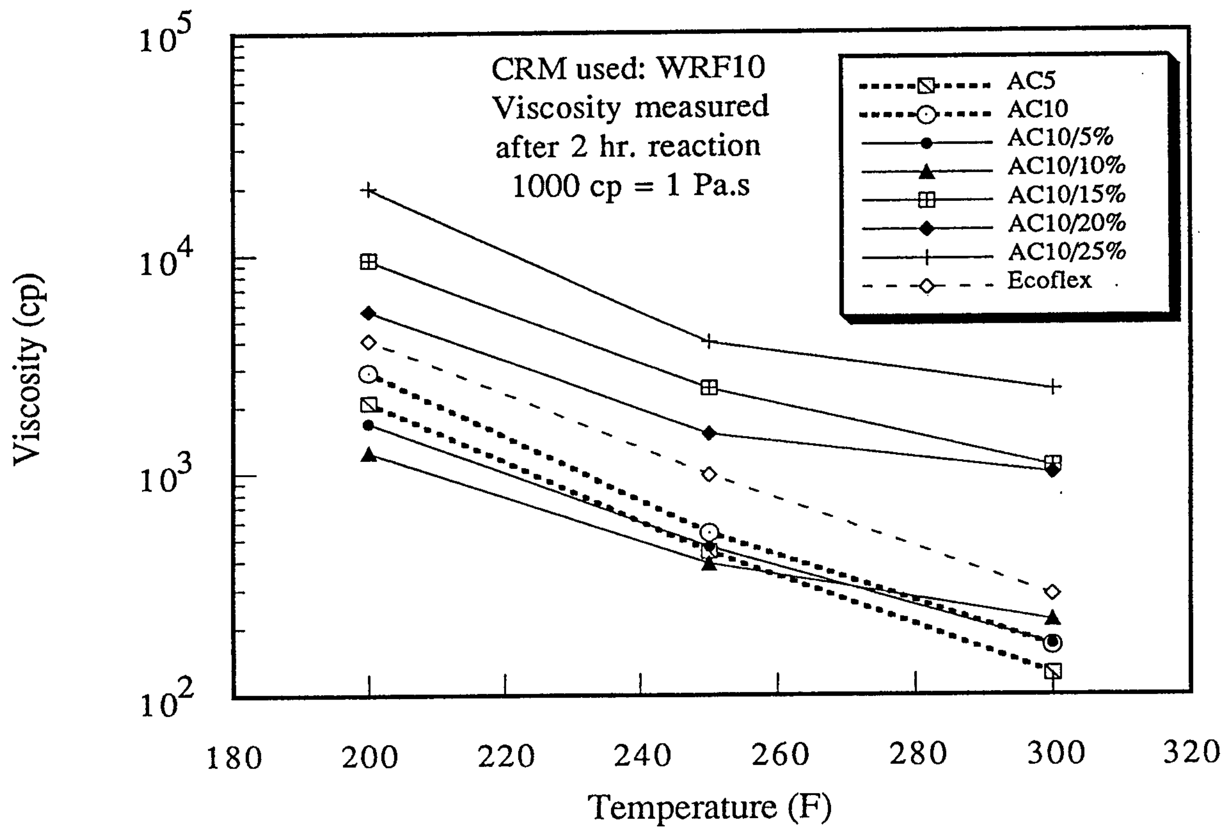


Fig. 3-14 Temperature susceptibility of asphalt-rubbers (AC10 base)

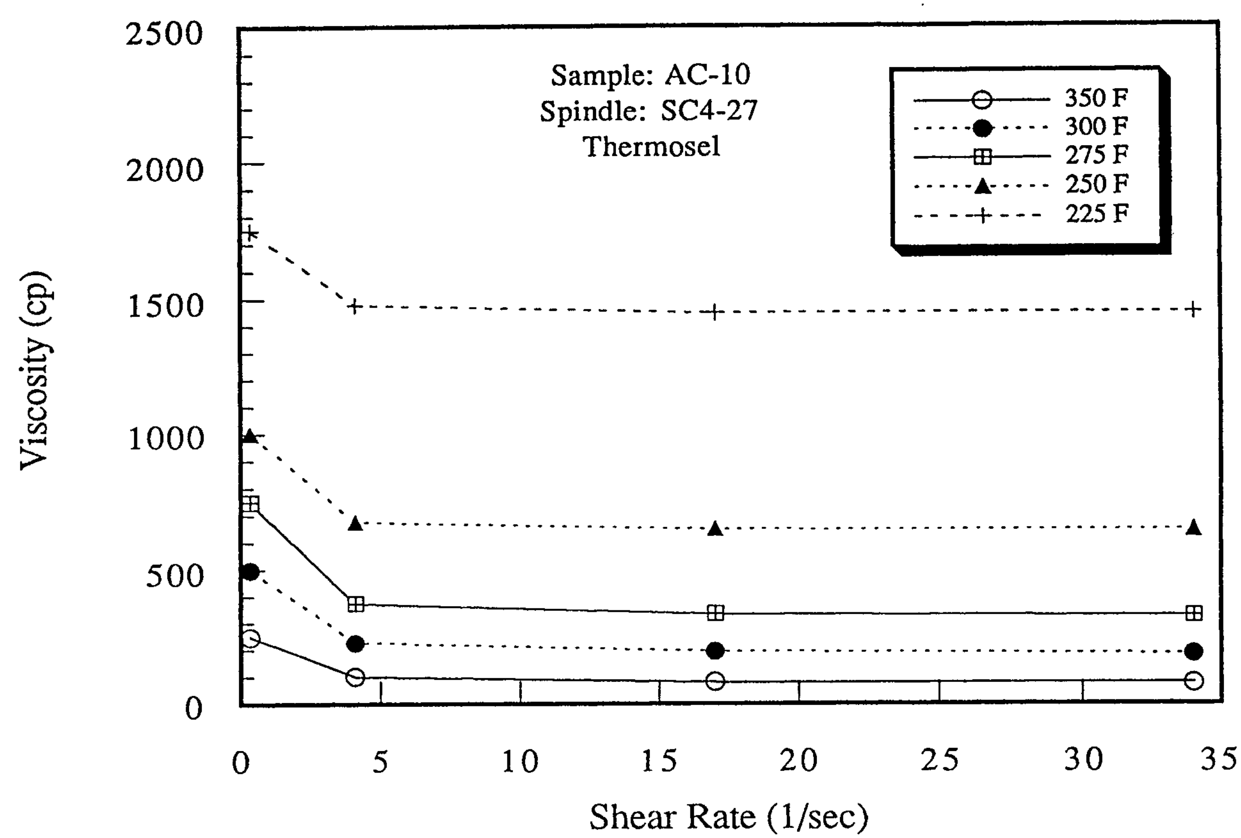


Fig. 3-15 Non-Newtonian behavior of AC-10 asphalt at different temperatures

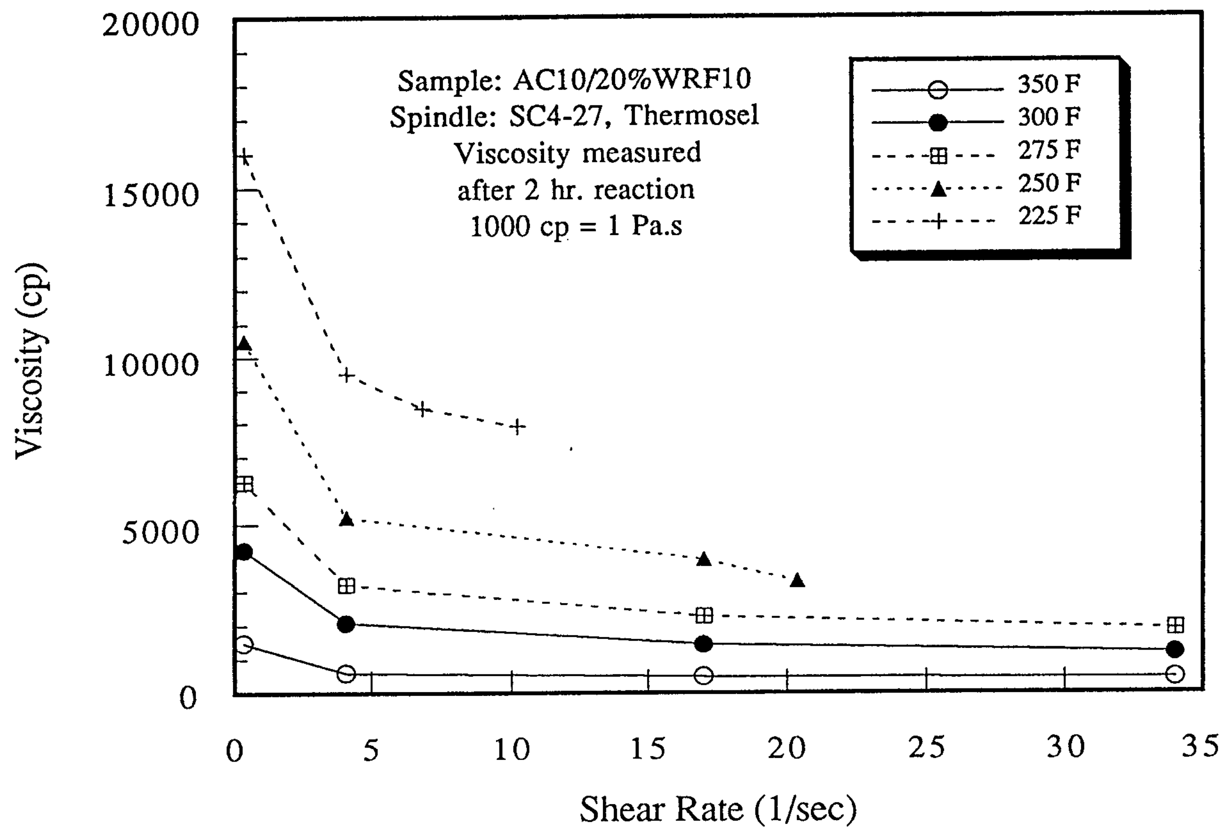


Fig. 3-16 Non-Newtonian behavior of AC10/20%WRF10 asphalt-rubber at different temperatures

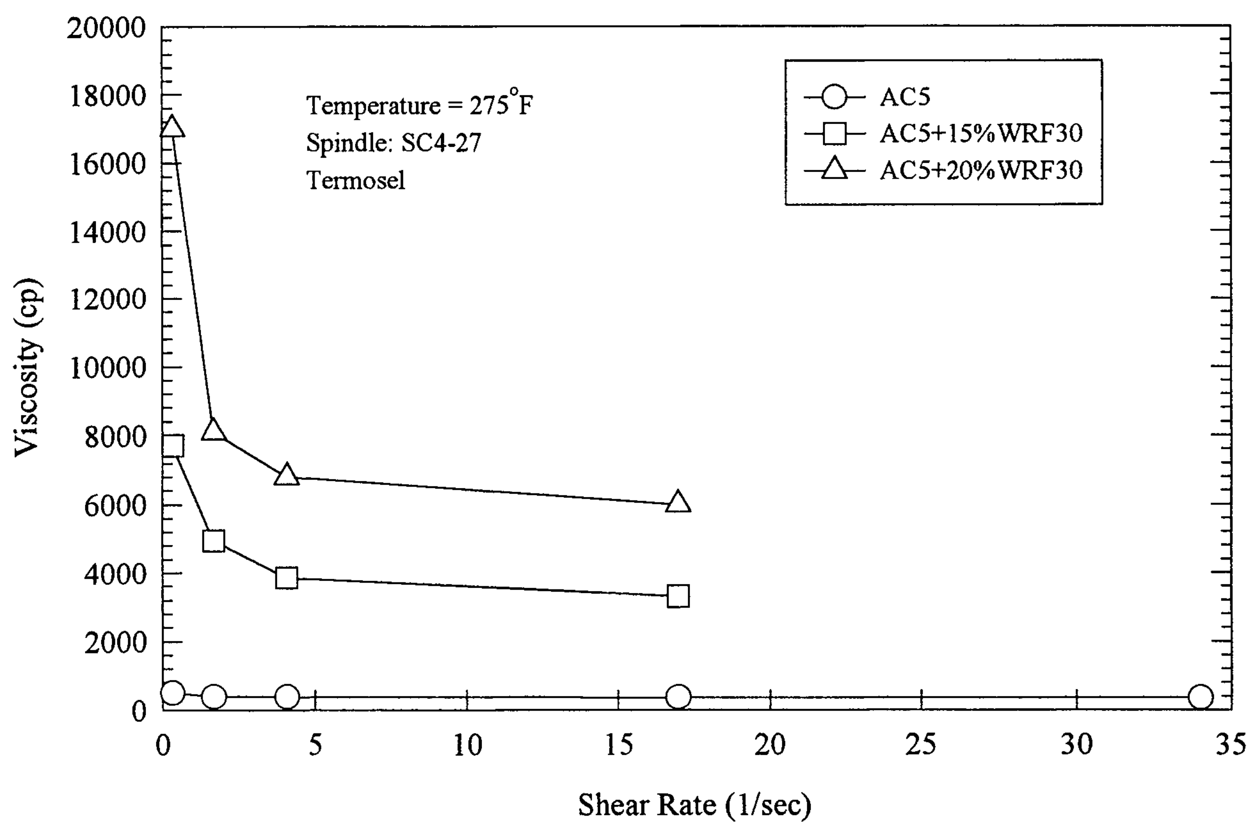


Fig. 3-17 Non-Newtonian behavior of AC5 based asphalt-rubber binders at 275°F

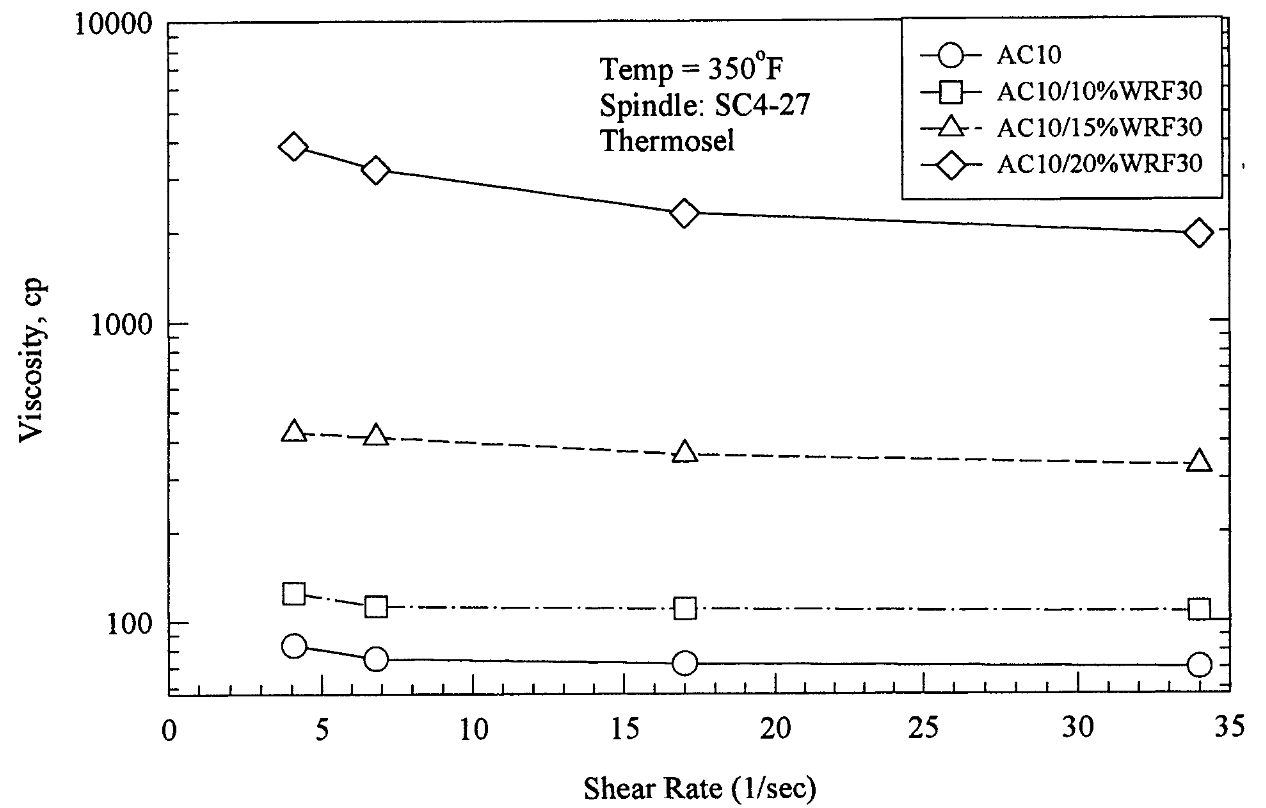


Fig. 3-18 Non-Newtonian behavior of AC-10 based asphalt-rubbers at 350°F

are shown in Table 3-10. Weight loss of binders due to short-term aging was also measured. The results are shown in Table 3-11. A bar graph is shown in Fig. 3-19.

Table 3-10 Viscosity measured before and after TFOT, cp

Types of binder	Before TFOT	After TFOT	Ratio
AC-5	260	333	1.28
AC-10	375	604.2	1.61
AC10/10%WRF10	1150	1812	1.58
AC10/15%WRF10	2100	4900	2.33
AC10/20%WRF10	3200	8600	2.69

It seems that AC-10 based asphalt-rubber with 15% and 20% of CRM have the higher percentage increase of viscosity than that of unmodified asphalt as indicated by the ratio. However, the proprietary Ecoflex seemed to show the highest short-term aging effect, while asphalt-rubbers (especially with finer CRM) showed very little change in weight as indicated by weight loss percentage.

Table 3-11 Weight loss measured before and after TFOT

Types of binder	Weight before TFOT, g	Weight after TFOT, g	Percent of weight loss, %
AC-10	51.489	51.423	0.128
Ecoflex	50.862	50.761	0.2
AC10/10% WRF10	53.151	53.104	0.088
AC10/15% WRF10	50.791	50.782	0.0178
AC10/25% WRF10	52.123	52.102	0.04
AC10/15% GY	50.726	50.725	0.0
AC5/15% WRF30	53.711	53.708	0.005

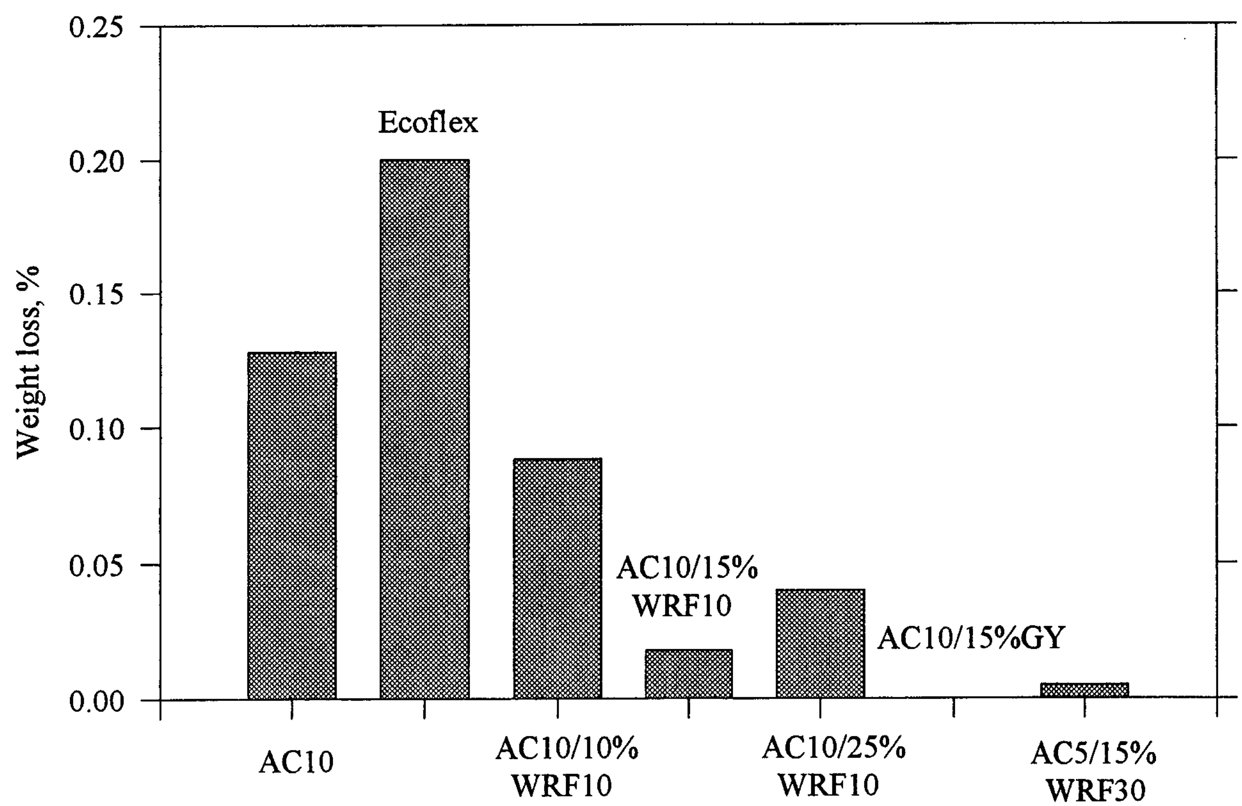


Fig. 3-19 Percent weight loss of the different binders after TFOT

Fig. 3-20 represents the non-Newtonian behavior of AC-5 and AC-10 asphalt at 275^0 F before and after the TFOT. It is observed that AC-5 and AC-10 asphalt samples after TFOT show reduced non-Newtonian behavior.

Fig. 3-21 represents the non-Newtonian behavior of AC-10 based asphalt-rubbers before and after TFOT. The TFOT was performed on AC-10 asphalt with 10%, 15%, and 20% of WRF 10 to investigate the sensitivity of non-Newtonian pattern. All three asphalt-rubbers in this figure show that the non-Newtonian behavior after TFOT is more pronounced than that of before TFOT. Also, CRM content affects the degree of non-Newtonian behavior. Therefore, 20% of WRF 10 modified asphalt-rubber shows the higher degree of non-Newtonian behavior than those of 10% or 15%, respectively.

3.7 Dynamic Shear Rheometer Test

This test (AASHTO Designations:TP5 Standard Test Method for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer) provides a means for measuring the complex shear modulus (G*) and phase angle (δ) of asphalt binder. It is applicable to the asphalt binders having complex shear modulus values in the range from 100 Pa to 10 MPA, which is typically found for binders in the temperature between 5^0 C and 85^0 C. This test method is intended for determining the linear viscoelastic properties of asphalt binders as required for Superpave specification.

The complex modulus (G*), defined as the ratio of the maximum shear stress (τ_{max}) to the maximum shear strain (γ_{max}), is a measurement of total resistance of a material to the deformation during repeated shearing. The phase angle (δ), defined as the time lag

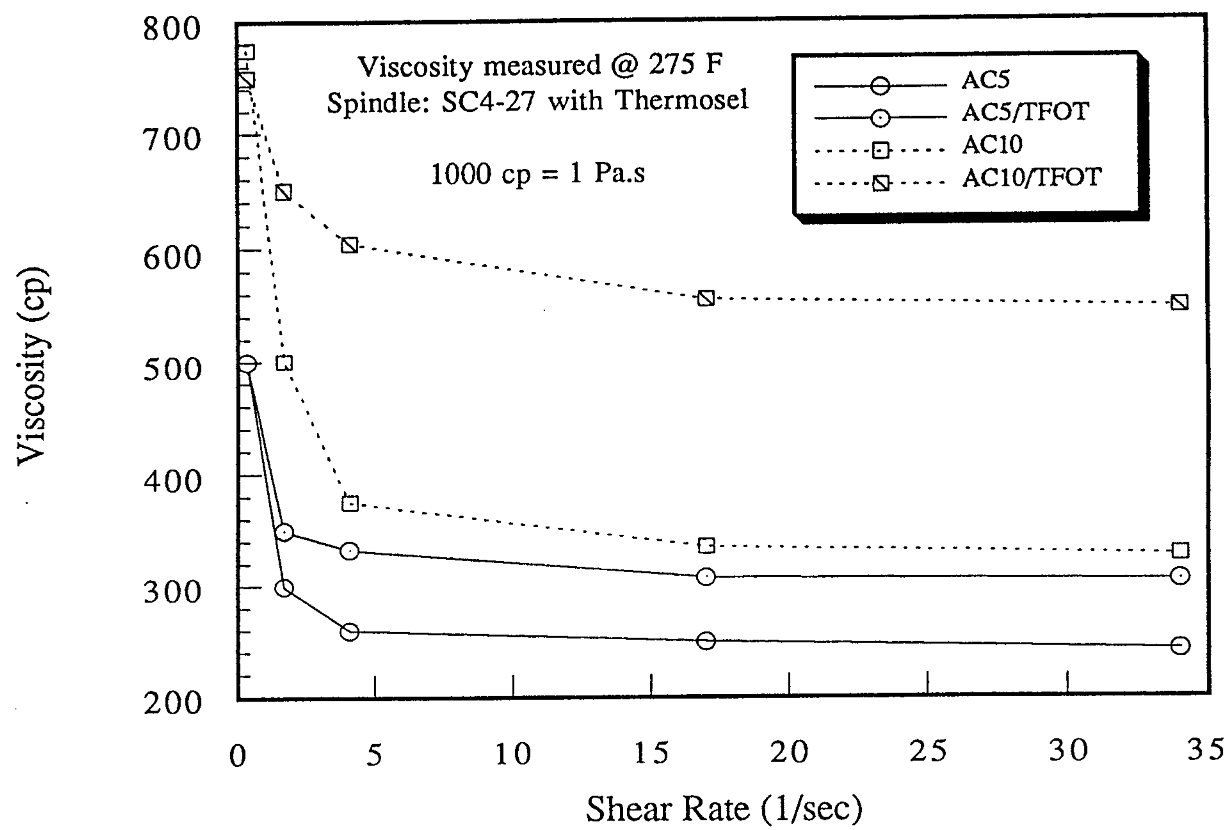


Fig. 3-20 Non-Newtonian behavior of AC5 and AC10 asphalt before and after TFOT at 275°F

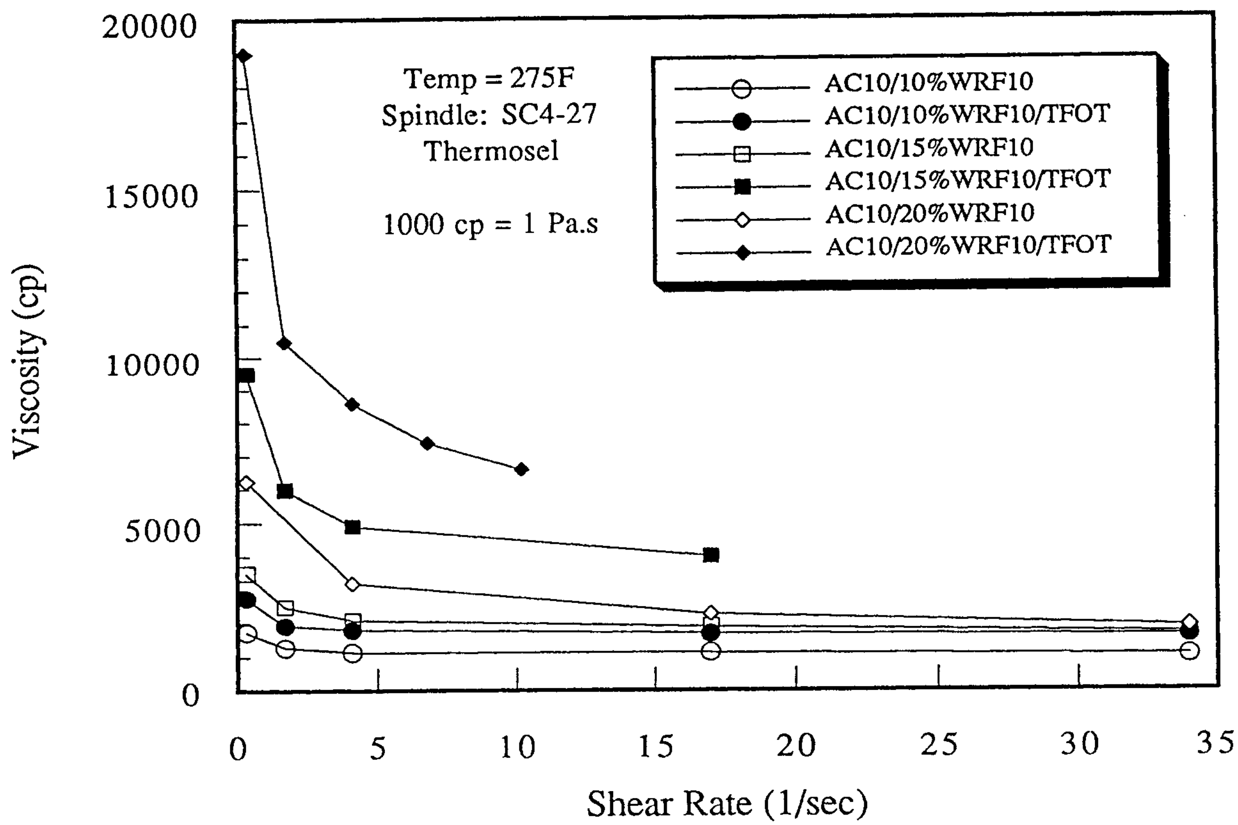


Fig. 3-21 Non-Newtonian behavior of AC10 based asphalt-rubber before and after TFOT at 275°F

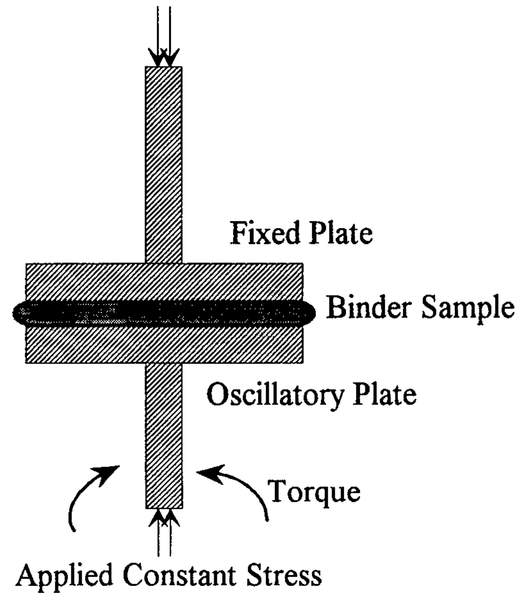
between applied stress to the resulting strain, is an indicator of the relative amount of recoverable and nonrecoverable deformation. For perfectly elastic material, an applied load coincides with an immediate strain response, and time lag is zero. A viscous material such as asphalt has a relative large time lag between load and response, leading to a phase angle that approaches 90 degree. Fig. 3-22 shows the schematic drawing of sample set-up and interpretation of dynamic shear rheometer test.

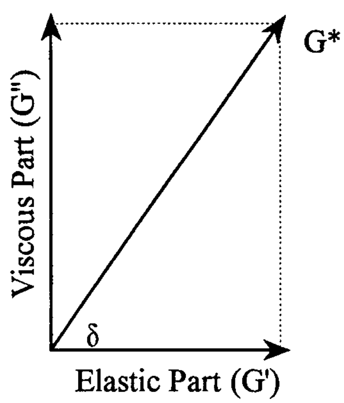
As a result of SHRP research, two parameters (G* and sin δ) were selected to describe the rheological behavior of binders. In AASHTO Designation MP1 (Standard Specification for Performance Graded Asphalt Binder), the minimum values of G*/sin δ for the original binder and short-term aged residue were specified as 1 and 2.2 kPa, respectively. A higher value of G*/sin δ corresponds to binders with less rutting. Also, the maximum value of G* sin δ for PAV residue was limited to 5000 kPa. A smaller value of G* sin δ corresponds to binders with less fatigue cracking potential.

The objective of this test in this study was to measure the effects of both CRM and short-term aging on these two parameters within the range of intermediate temperatures. The measurements were collected in a range of temperatures from 40^0 to 60^0 C.

The dynamic shear rheometer and software (Rhios) were provided by Rheometrics, Inc. The testing was conducted at the Ohio DOT, Bureau of Testing, Bituminous Laboratory. Because the CRM was finer than 0.6 mm, a 1 mm gap between the 25 mm parallel plates was used.

Summary of Standard Test Method Recommended by SHRP





G*: Complex Shear Modulus
δ: Phase Angle

Fig. 3-22 Schematic drawing of sample set-up and interpretation for dynamic shear rheometer test

- This standard contains the procedure used to measure the complex shear modulus and phase angle of asphalt binders using a dynamic shear rheometer and parallel plate test geometry.
- The standard is suitable for use when the complex shear modulus varies between 100 Pa and 10 MPa. This range in modulus is typically obtained between 5^0 C and 85^0 C, depending upon the grade, test temperature, and conditioning (aging) of the asphalt binder.
- Test specimens 1 mm thick by 25 mm in diameter or 2 mm thick by 8 mm in diameter are formed between parallel metal plates. During testing, one of the parallel plate is oscillated with respect to the other at pre-selected frequencies and rotational deformation amplitudes (or torque amplitudes). The required amplitude depends upon the value of the complex shear modulus of the asphalt binder being tested. The required amplitudes have been selected to ensure that the measurements are within the region of linear behavior.
- The test specimen is maintained at the test temperature to within +/- 0.1^0 C by positive heating and cooling of the upper and lower plates.
- Oscillatory loading frequencies using this standard can range from 1 to 100 rad/s using a sinusoidal waveform. The recommended testing is performed at a test frequency of 10 rad/s. The complex shear modulus and phase angle are calculated automatically as part of the operation of the rheometer using the proprietary computer software supplied by the equipment manufacturer.

Significance and Use

- The test temperature for this test is related to the temperature experienced by the pavement in the geographical area for which the asphalt binder is intended for.
- The complex modulus is an indicator of the stiffness or resistance of asphalt binder to deformation under load. The complex shear modulus and the phase angle define the resistance to shear deformation of the asphalt binder in the linear viscoelastic region. Other viscoelastic properties, such as storage shear modulus (G'), or the loss shear modulus (G"), can be calculated from the complex modulus and phase angle. The loss modulus is a measure of the energy dissipated during each loading cycle.
- The complex modulus and the phase angle are used to calculate performance-related criteria in accordance with AASHTO Designation: MP1 (Standard Specification for Performance Graded Asphalt Binder).

Results of Dynamic Shear Rheometer Test

Both unaged and short-term aged binders of AC-5, AC-10, AC-20, Ecoflex, and CRM modified binders (10%, 15%, and 20% of WRF 30) were tested. Three different temperatures (40^0, 50^0, and 60^0 C) and one logarithmic frequency sweep mode (1 rad/s to 100 rad/s) were selected to investigate both the temperature and frequency effects. The applied shear stresses were 120 Pa for unaged binder, 220 Pa for short-term aged binder, and 5 kPa for asphalt-rubbers both unaged and short-term aged.

The complex shear modulus G*, dynamic vicosity η, phase angle δ, rutting potential resistance index $G^*/\sin\delta$, and fatigue cracking resistance index $G^*\sin\delta$ at 10 rad/s are summarized in Table 3-12, 3-13 and 3-14 for various binder systems. Figs. 3-23 to 3-26 show that the dynamic viscosity, defined as G^* over the frequency, of each unmodified binder decreases with increasing temperature and frequency. Also, the difference in dynamic viscosity between unaged and short-term aged binders was observed. The same trend in terms of the effect of aging was observed in asphalt-rubber binders with various CRM contents (see Figs. 3-27, 28, and 29 for 10%, 15%, and 20% of WRF30, respectively). The differences of the dynamic viscosity values between the unaged and short-term aged asphalt-rubber binders were found to be greater than those for unmodified binders.

Phase angles measured at 10 rad/sec frequency are plotted in order to quantify the effect of short-term aging on different graded binders and asphalt-rubbers. Figs. 3-30 and 3-31 show the decrease in phase angle due to the effect of short-term aging at different temperature. The phase angle of Ecoflex shows a smaller number than those of the other asphalts due to the melted rubber components in the Ecoflex. Fig. 3-32 shows that the decrease in phase angle of asphalt-rubbers is more prominent. Because of the effect of rubber, the phase angles of the asphalt-rubber binders generally showed smaller values (see Table 3-14 and Fig. 3-33).

Table 3-12 Results of the dynamic shear rheometer test for unmodified binders at different temperatures and ages (frequency = 10 rad/s)

Designation	G*, kPa	η, Pa.s	G*/sin δ, kPa	G*sin δ, kPa	δ,°
AC5/U/40	20.713	2071.3	20.955	20.474	81.28
AC5/SA/40	30.363	3036.3	31.037	29.704	78.04
AC5/U/50	4.687	468.7	4.704	4.670	85.12
AC5/SA/50	6.650	665.0	6.698	6.554	83.12
AC5/U/60	1.249	124.9	1.249	1.249	89.99
AC5/SA/60	1.257	125.7	1.257	1.257	89.10
AC10/U/40	32.239	3223.9	32.747	31.738	79.89
AC10/SA/40	59.443	5944.3	61.729	57.242	74.36
AC10/U/50	6.474	647.4	6.505	6.443	84.37
AC10/SA/50	8.420	842.0	8.557	8.258	79.73
AC10/U/60	1.719	171.9	1.719	1.719	88.91
AC10/SA/60	3.227	322.7	3.240	3.215	84.96
AC20/U/40	49.942	4994.2	51.054	48.854	78.02
AC20/SA/40	91.850	9185.0	95.579	88.270	73.95
AC20/U/50	12.134	1213.4	12.252	12.017	82.03
AC20/SA/50	19.276	1927.6	19.660	18.900	78.66
AC20/U/60	3.219	321.9	3.226	3.212	86.19
AC20/SA/60	4.752	475.2	4.782	4.723	83.63

Table 3-13 Results of the dynamic shear rheometer test for CRM modified binders (AC5 base) at different temperatures and ages (frequency = 10 rad/s)

Designation	G*, kPa	η, Pa.s	G*/sin δ, kPa	G*sinδ, kPa	δ, °
10%R/U/40	27.529	2752.9	28.553	26.542	74.61
10%R/SA/40	57.031	5703.1	64.7	50.271	61.82
10%R/U/50	3.921	392.1	3.933	3.909	85.58
10%R/SA/50	4.992	499.2	5.25	4.747	71.97
10%R/U/60	1.34	134.0	1.34	1.34	90.75
10%R/SA/60	3.196	319.6	3.227	3.165	82.03
15%R/U/40	40.014	4001.4	43.991	36.397	65.45
15%R/SA/40	71.775	7177.5	85.341	60.366	57.25
15%R/U/50	10.221	1022.1	10.573	9.881	75.18
15%R/SA/50	21.851	2185.1	24.526	19.468	62.99
15%R/U/60	2.949	294.9	2.967	2.933	83.69
15%R/SA/60	5.948	594.8	6.191	5.715	73.91
20%R/U/40	46.258	4625.8	53.709	39.841	59.46
20%R/SA/40	84.197	8419.7	105.52	67.181	52.93
20%R/U/50	17.516	1751.6	18.939	16.2	67.65
20%R/SA/50	30.034	3003.4	36.859	24.472	54.57
20%R/U/60	4.29	429.0	4.394	4.189	77.52
20%R/SA/60	11.488	1148.8	12.948	10.193	62.53

* R: WRF 30
* U: Unaged binder
* SA: Short-term aged binder

Table 3-14 Results of dynamic shear rheometer test for CRM modified binder (AC10 base) at different temperature.(frequency=10rad/s)

Binder&Temperature	G*, kPa	η, Pa.s	G*/sinδ, kPa	G*sinδ, kPa	δ, °
AC10+10%WRF30--40°C	48.490	4849.0	53.263	44.145	65.56
AC10+10%WRF30--50°C	13.130	1313.0	13.833	12.462	71.65
AC10+10%WRF30--60°C	3.867	386.7	3.956	3.780	77.83
AC10+15%WRF30--40°C	66.994	6699.4	79.99	56.109	56.88
AC10+15%WRF30--50°C	21.926	2192.6	24.610	19.534	62.99
AC10+15%WRF30--60°C	7.510	751.0	7.939	7.104	71.08
AC10+20%WRF30--40°C	94.539	9453.9	125.494	71.219	48.88
AC10+20%WRF30--50°C	37.127	3712.7	47.231	29.185	51.82
AC10+20%WRF30--60°C	15.688	1568.8	18.531	13.281	57.84
AC10+10%GY--40°C	31.809	3180.9	33.113	30.557	73.87
AC10+10%GY--50°C	7.249	724.9	7.378	7.123	79.29
AC10+10%GY--60°C	1.813	181.3	1.821	1.805	84.78
AC10+15%GY--40°C	72.413	7241.3	80.349	65.261	64.32
AC10+15%GY--50°C	15.771	1577.1	16.179	15.373	77.10
AC10+15%GY--60°C	4.428	442.8	4.466	4.390	82.52
AC10+20%GY--40°C	90.851	9085.1	105.129	78.512	59.79
AC10+20%GY--50°C	23.678	2367.8	24.705	22.694	73.42
AC10+20%GY--60°C	7.262	726.2	7.375	7.151	79.97

3.8 Summary

AC10 with 10% of WRF30 and AC5 with 15% of WRF 30 were selected as final asphalt-rubber binders for mix design after careful review of the test results and FHWA recommendations. AC-20 was selected as a control mix. Also, the result of sieve analysis showed that WRF 30 satisfied FHWA recommendation for dense-graded mix.

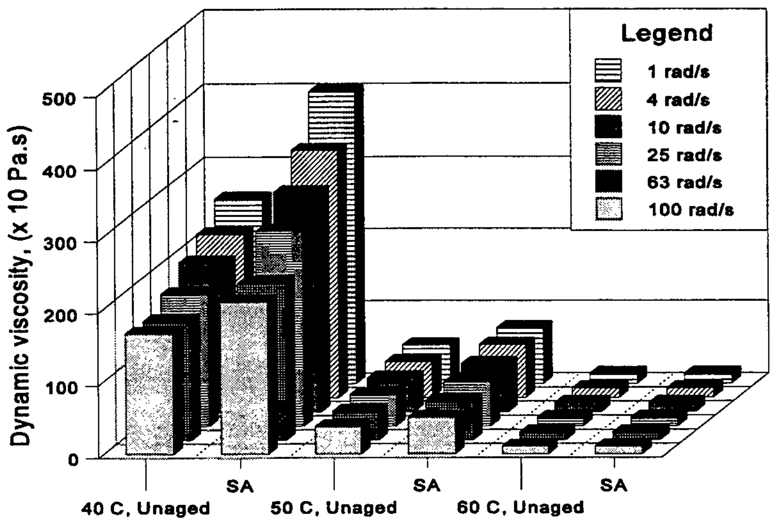


Fig. 3-23 Dynamic viscosity variation due to temperature and frequency for AC-5 binder

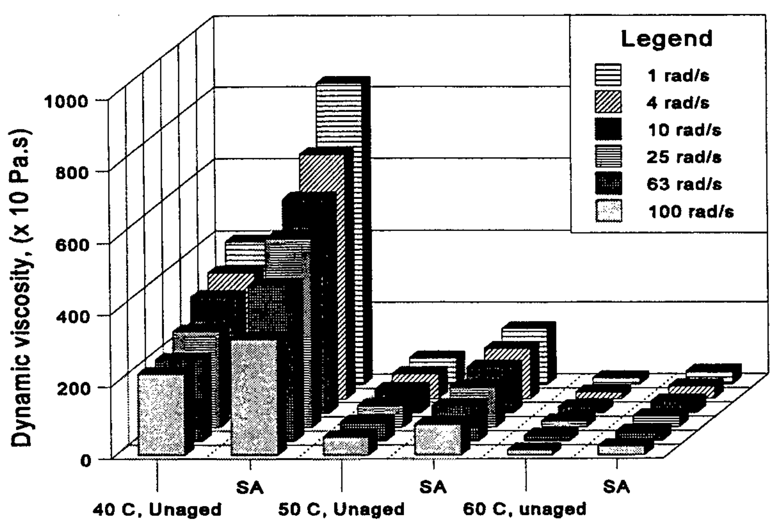


Fig. 3-24 Dynamic viscosity variation due to temperature and frequency for AC-10 binder

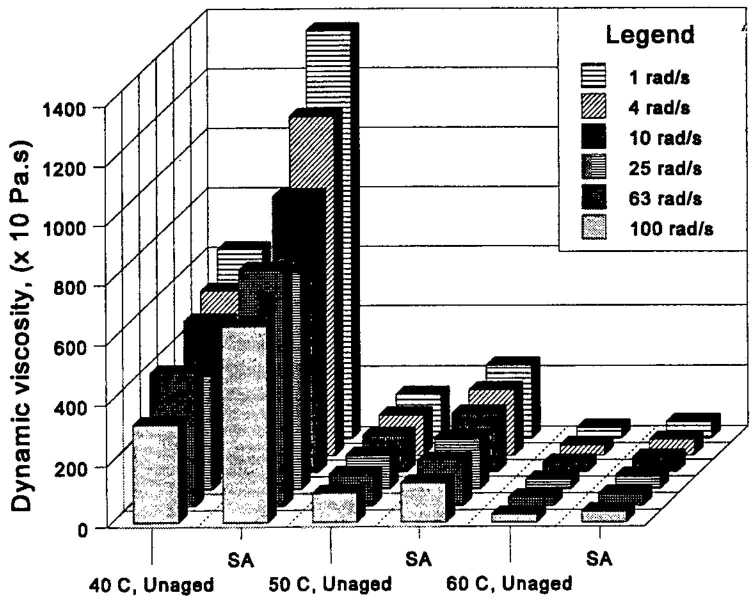


Fig. 3-25 Dynamic viscosity variation due to temperature and frequency for AC-20 binder

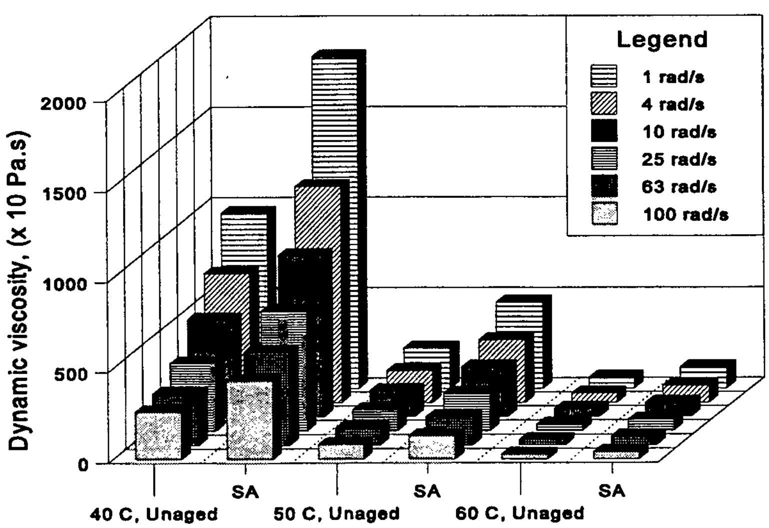


Fig. 3-26 Dynamic viscosity variation due to temperature and frequency for Ecoflex binder

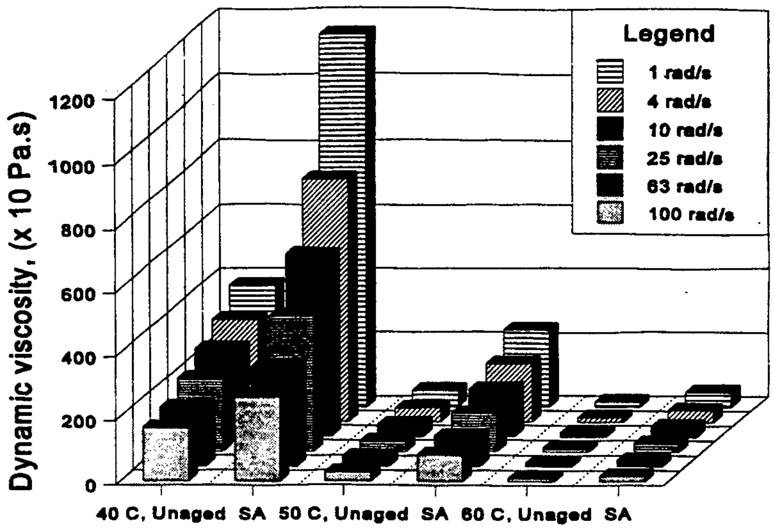


Fig. 3-27 Dynamic viscosity variation due to temperature and frequency for AR binder (AC-5 with 10%WRF30)

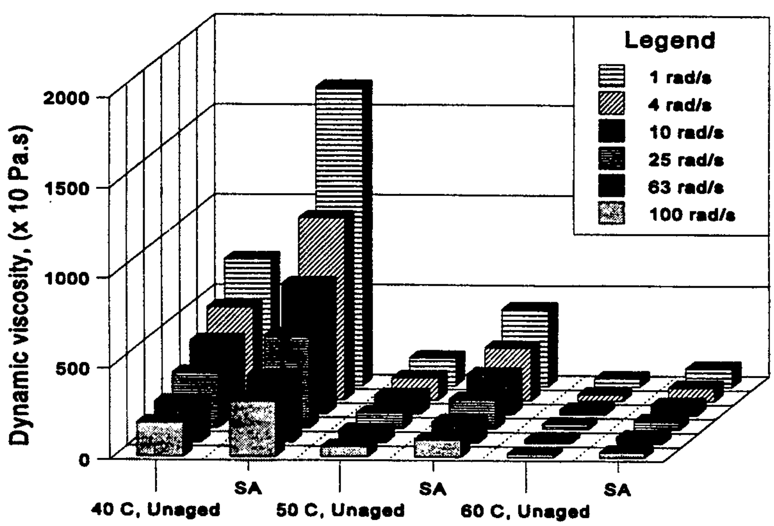


Fig. 3-28 Dynamic viscosity variation due to temperature and frequency for AR binder (AC-5 with 15%WRF30)

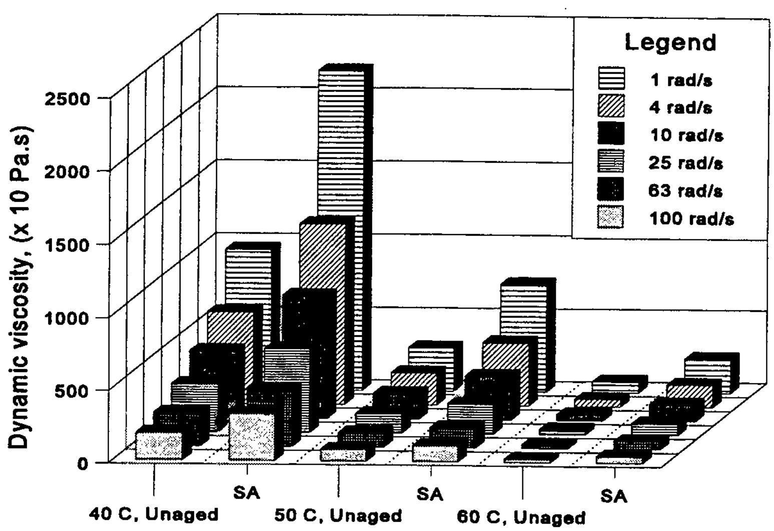


Fig. 3-29 Dynamic viscosity variation due to temperature and frequency for AR binder (AC-5 with 20%WRF30)

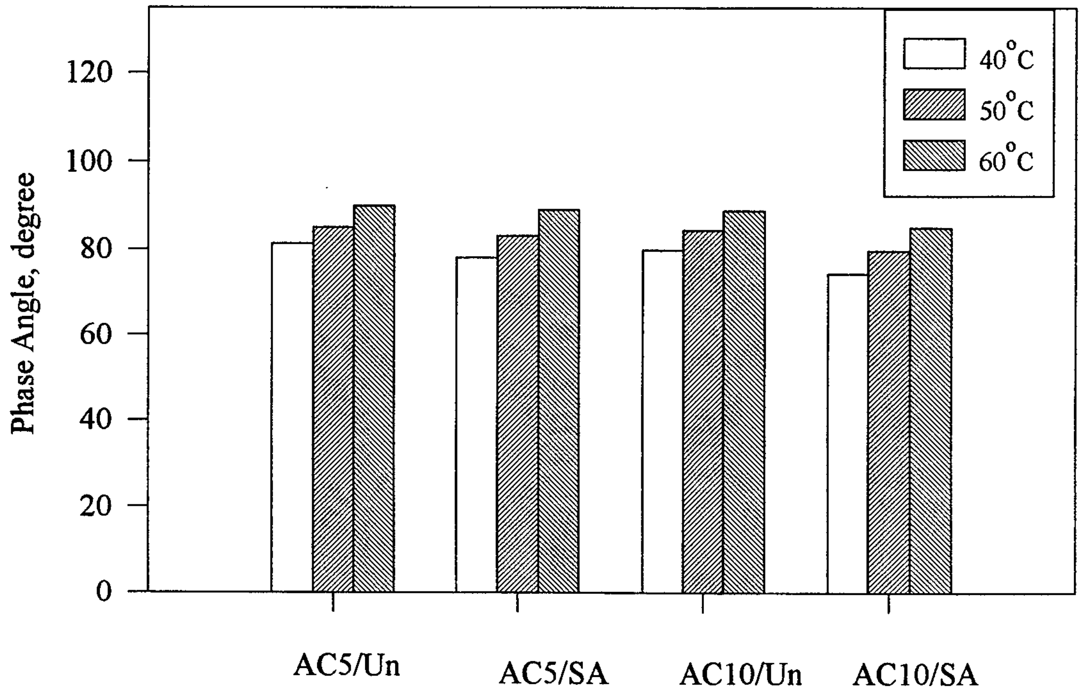


Fig. 3-30 Changes in phase angle due to short-term aging for AC5 and AC10 asphalt at different temperature

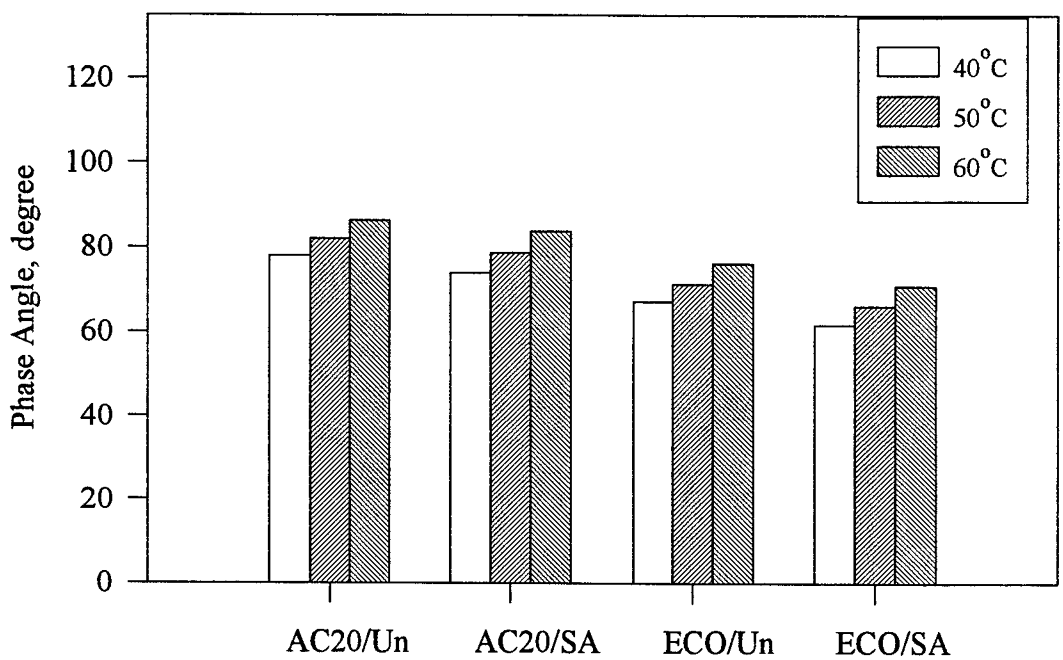


Fig. 3-31 Changes in phase angle due to short-term aging for AC20 and Ecoflex at different temperature

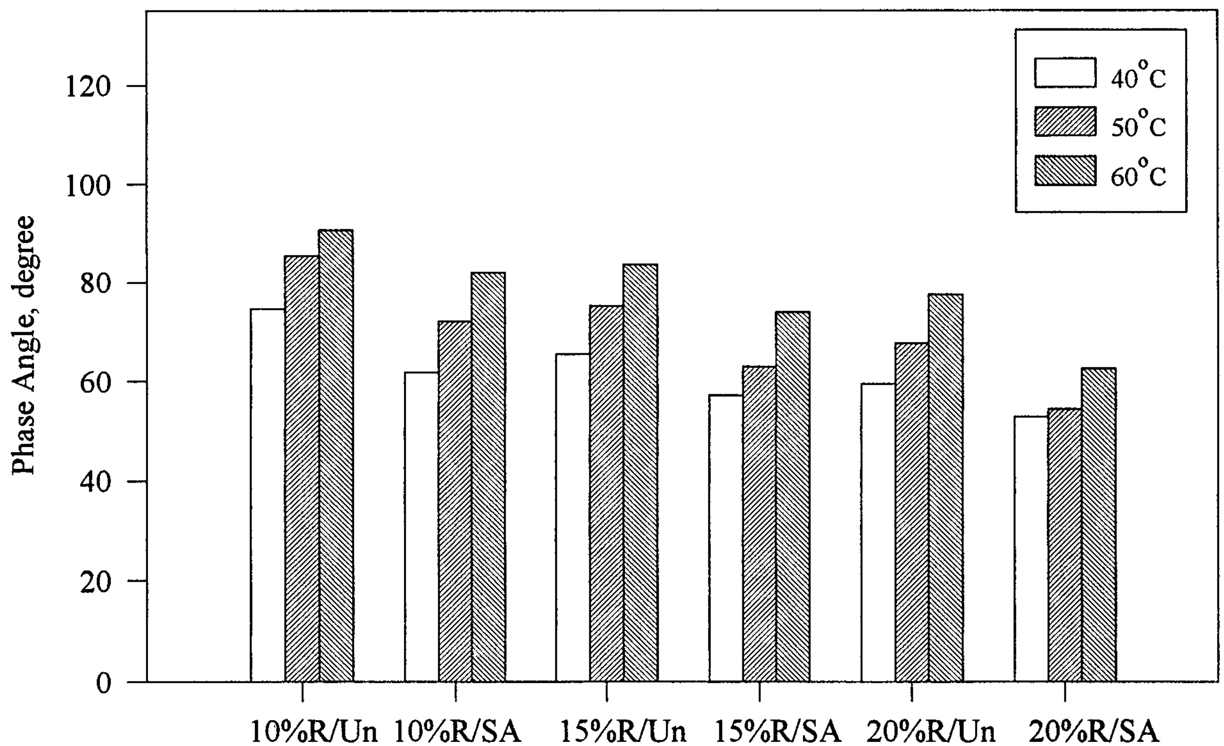


Fig. 3-32 Changes in phase angle due to short-term aging for asphalt-rubbers (AC5 base) at different temperature

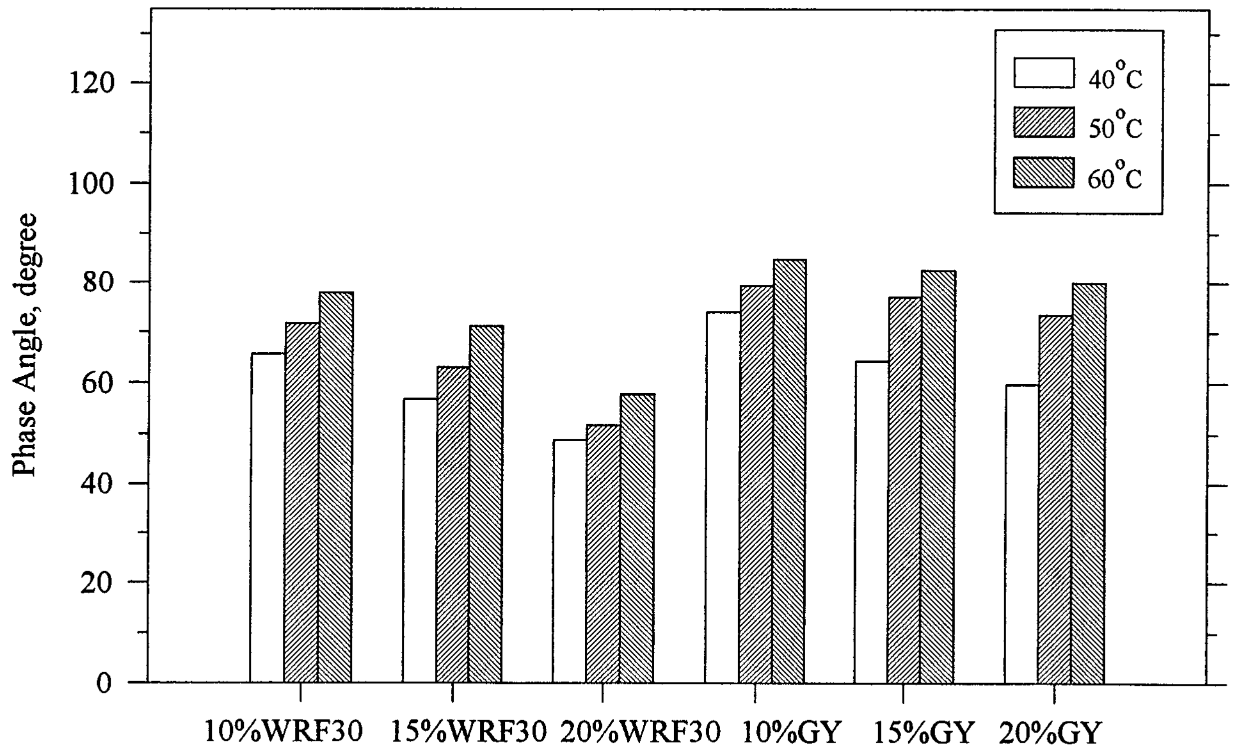


Fig. 3-33 Phase angles of asphalt-rubbers (AC10 base) at different temperature

1. The swelling test results indicated that the projected maximum percent weight increase, either for small or large specimen, seemed to be around 50 percent in the wet process. The maximum percent weight increase for the dry process was found to be 1.41% for a small rubber specimen and 1.93% for a large specimen.
2. The Brookfield rotational viscosity test results indicated that the viscosity of asphalt-rubber binder increases with an increase in CRM content and reaction period but decreases with an increase in temperature.
3. For unaged binders (AC-5 and AC-10), the magnitude of non-Newtonian behavior was reduced after short-term aging period. However, an opposite trend was observed for the asphalt-rubbers (AC-10 based), i.e., short-term aging seemed to promote non-Newtonian behavior.
4. The dynamic shear rheometer test results indicated that dynamic viscosity of short-term aged binders (either unmodified binder or asphalt-rubber) possess higher numbers than that of unaged binders. Also, increase in both frequency and temperature resulted in a decrease in the dynamic viscosity. Both complex modulus and phase angle increase with short-term aging for both unmodified asphalt and CRM modified asphalt. Increase in complex shear modulus indicates the stiffening effects; however, increase in phase angle represents more viscous behavior. Both effects are more pronounced for asphalt-rubber than unmodified binders.

6. For the values of G'/G" (inverse of tan δ), which implies the ratio of an elastic component to a viscous component, short-term aged binders showed higher numbers than those of unaged binders.

7. $G^*/\sin\delta$ of both unaged and short-term aged asphalt-rubber binder is greater than the minimum values of 1.00 kPa and 2.2 kPa, respectively, required by the Performance Graded Asphalt Binder Specification to possess the potential of resistance to rutting at a high temperature of 60°C.

CHAPTER IV

MIX DESIGN

Over the past two decades, the optimum asphalt content of the asphalt concrete paving mixtures has been determined from either Marshall or Hveem procedure. Each procedure uses a series of laboratory tests to select the optimum asphalt content. This selection is based upon satisfying the following objectives:

1. Limitimg permeability.
2 Providing room for additional traffic densification.
3. Insuring adequte strength for carrying traffic loads.
4. Resisting excessive permanent deformation
5. Providing adequate film thickness.

Test limits were selected subjectively for these objectives besed upon the experience of engineers and historical observations of pavement performance prior to 1950's.

In this chapter, details of Marshall mix design method for asphalt-rubber-aggregate mix will be presented. It consists of five sections, including materials used, specimen preparation procedures, Marshall test results, open graded friction course test results, and summary.

4.1 Materials Used

4.1.1 Aggregates

The aggregates used in this research were crushed limestones (No. 57 and 9-D per ODOT specification) purchased from a local aggregate supplier (Akron Crushed Limestone Co.). Aggregates were first oven dried before sieving to separate various particle sizes. The sieved aggregates were washed and then oven dried. The following sieve sizes were used: 3/8", No. 4, No. 8, No. 16, No. 50, and No. 200 sieve. Table 4-1 provides information on aggregate gradations used in preparing the rubber modified asphalt-aggregate mix according to the wet process. The aggregate gradation curves for wet process used in the present Marshall mix design, along with ODOT # 404 and the maximum density line were plotted in Fig. 4-1 and Fig. 4-2 for dense-graded and gap-graded, repectively. Table 4-2 shows the aggregate and CRM gradations used in preparing the rubber modified asphalt-aggregate mix according to the generic dry process. The aggregate gradations for dry process were plotted in Fig. 4-3. Two percent of CRM (by the weight of aggregate) was used for dense-graded as well as gap-graded mixture. Table 4-3, furnished by International Surfacing, Inc., shows the suggested specification of aggregate gradation for the open graded asphalt-rubber-aggregate mixtures.

Table 4-1 Aggregate gradations and FHWA requirement for different graded HMA

Sieve Size	FHWA Dense	ODOT #404	Dense JMF	Max. density	FHWA Gap	Gap JMF
1/2"	100	100	100	100	100	100
3/8"	90-100	90-100	90	87.9	78-92	86
No. 4	60-80	45-75	60	64.2	28-42	36
No. 8	40-60	-	40	46.9	15-25	22
No. 16	-	15-45	28	34.3	-	15
No. 30	18-30	-	18	25.3	5-15	10
No. 50	8-18	3-22	8	18.5	-	8
No. 200	2-8	0-8	2	9.9	3-7	5

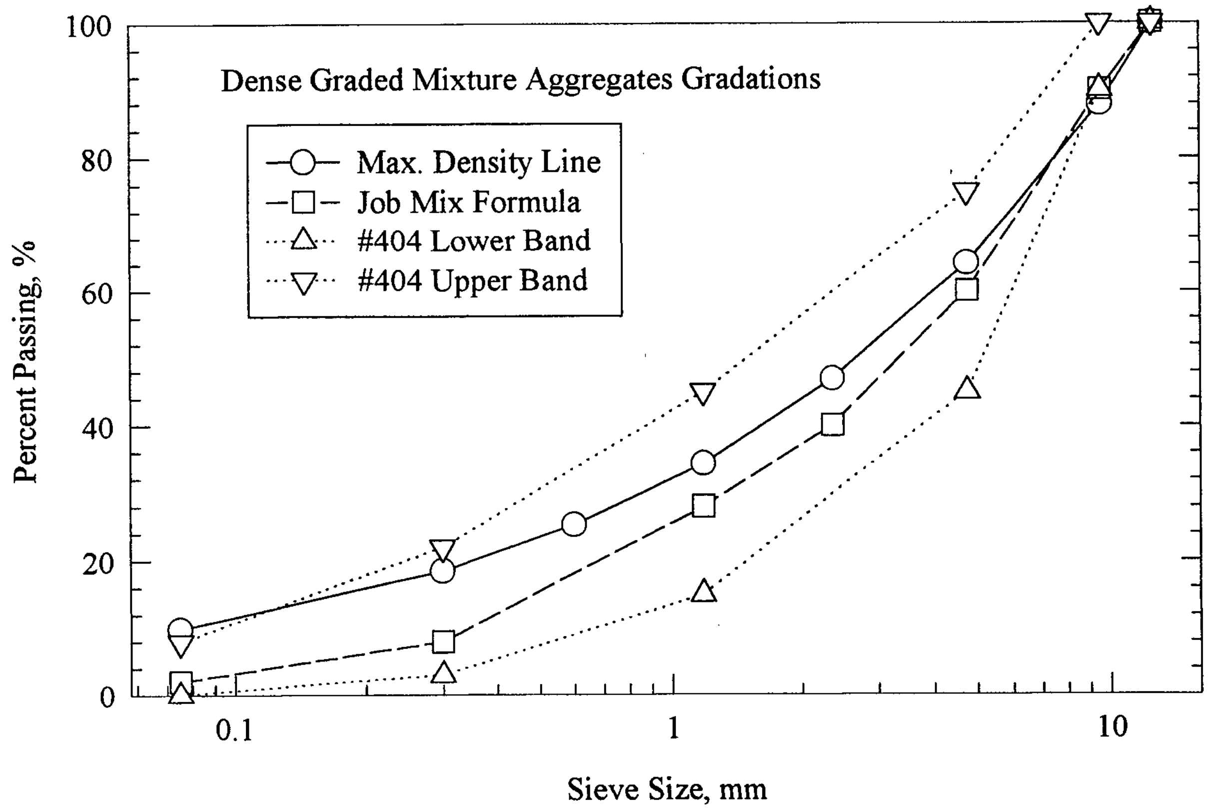


Fig. 4-1 Dense-graded Aggregate Gradations Used for Mix Design

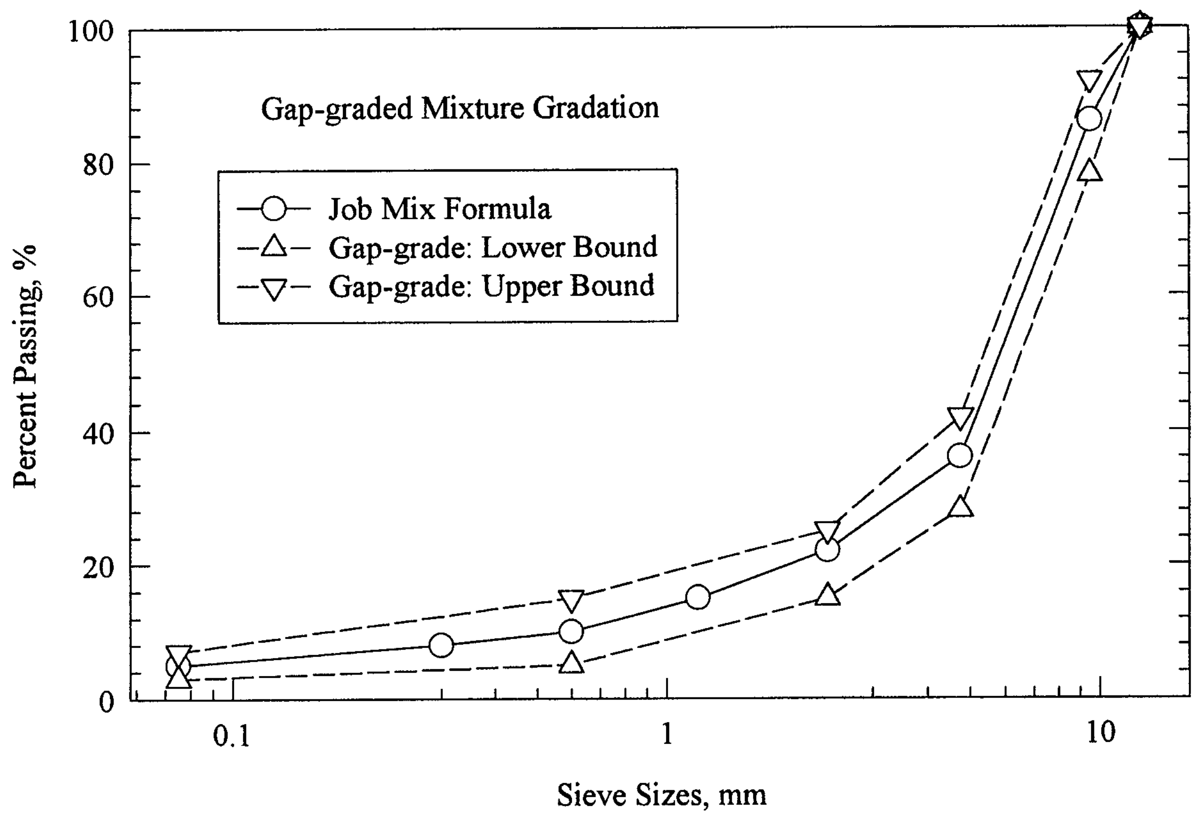


Fig. 4-2 Gap-graded aggregates gradation used in mix design

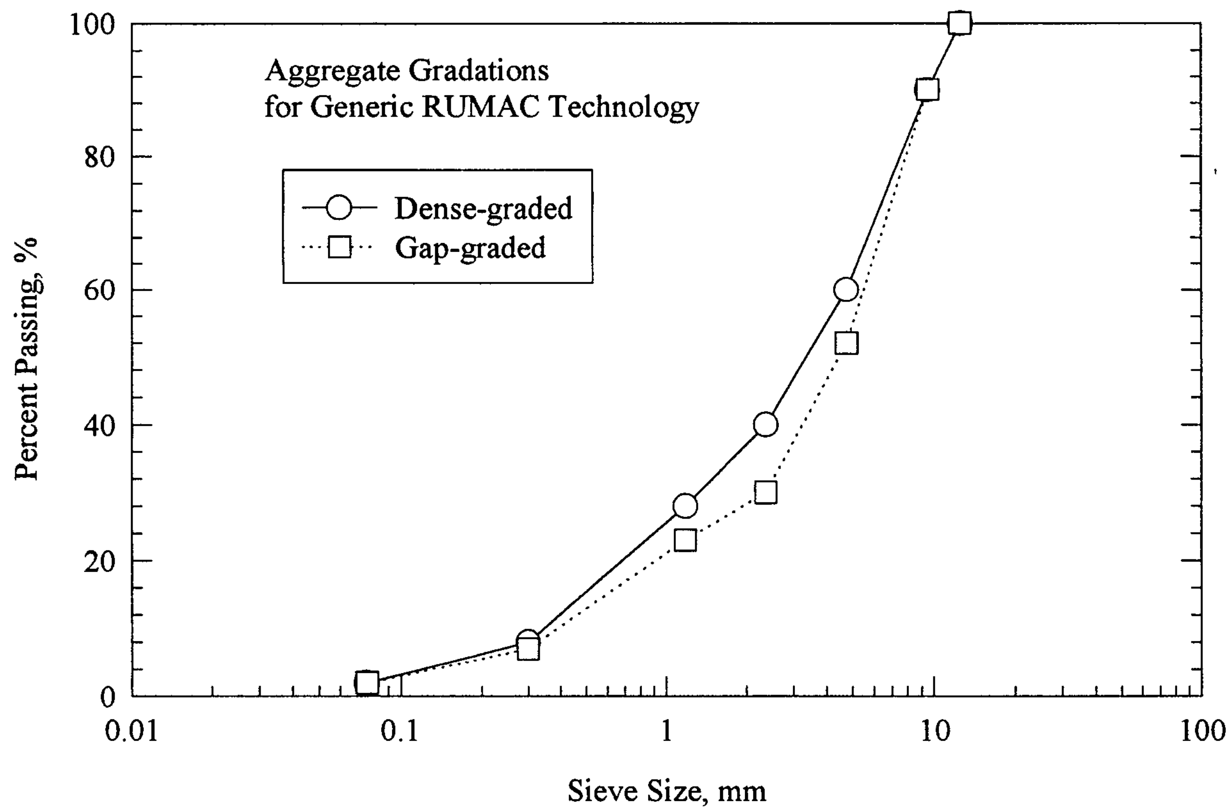


Fig. 4-3 Dense- and gap-graded aggregates gradation used in preparing Marshall specimens (generic dry process)

Table 4-2 Aggregate and CRM gradations of generic RUMAC mix design

Sieve size	ODOT # 404	Gap-RUMAC	CRM	Dense-RUMAC
1/2"	100	100	-	100
3/8"	90-100	90	-	90
No. 4	45-75	52	100	60
No. 8	-	30	85	40
No. 16	15-45	23	52	28
No. 30	-	-	27	-
No. 50	3-22	7	10	8
No. 200	0-8	2	-	2

Table 4-3 Suggested specification of aggregate gradation and job mix formula for open-graded asphalt-rubber -aggregate mixtures (International Surfacing, Inc., 1992)

Sieve Size	3/8" Max	1/2" Max	Job Mix Formula
3/4"	100	100	100
1/2"	100	95-100	100
3/8"	85-100	75-95	90
No. 4	25-55	20-45	40
No. 8	5-15	5-15	10
No. 30	0-10	0-10	5
No. 200	0-5	0-5	2

4.1.2 CRM

The majority of CRM used in this study were obtained from Baker Rubber Inc. (BRI) in Chambersberg, PA. Different sizes of CRM were available, including WRF 1/4", WRF 10, and WRF 30. Product description of BRI's CRM has been shown previously in Table 3-2. An ultra fine CRM was obtained from Goodyear Tire Rubber Co. in Cleveland, Ohio. The results of sieve analysis of each CRM used in this study have been shown previously in Table 3-3.

4.1.3 Asphalt Cement

AC-5, AC-10, and AC-20 supplied by Ashland Petroleum Co. in Canton, Ohio were used. Ecoflex, a proprietary rubberized asphalt manufactured by BITUMAR Inc. Quebec, Canada was also studied.

4.2 Specimen Preparation Procedures

This section presents the specimen preparation procedures for the Marshall mix design method as well as for open grade friction course mix design method. The Marshall mix design covers both wet process and dry process.

4.2.1 Wet Process

Marshall specimens were prepared according to ASTM D 1559 using 50-blow compactive effort. Dense-graded aggregates were mixed with AC20, Ecoflex, asphalt-rubber binder (AC5 modified by 15% of WRF 30) and asphalt-rubber binder (AC10 modified by 10% of WRF30). Also, Dense-graded Marshall mix design with continuous blending technology was included in this study. In addition, gap-graded aggregates were mixed with the same AC5+15%WRF30 asphalt-rubber binder to prepare specimens for Marshall test.

The asphalt-rubber binder and the aggregates were heated up to 350° F and 300° F, respectively. It was noted that actual mixing of asphalt-rubber with aggregate took about 1 minute and 30 seconds. Compaction was done by impact with 50 blows per side for a medium traffic volume.

Specimen Preparation Procedures

The mix design procedure used for wet process consists of the following steps:

- preparation of mixtures
    - weigh out ingredients (aggregates, CRM, and asphalt)
    - produce asphalt-rubber with 2 hour reaction (350^0 F)
    - heat aggregates and asphalt-rubber (300^0 F and 350^0 F)
    - wet mix for about 1 minute 30 second
- prepare compaction mold and hammer (heated to 300^0 F)
- compact 50 blows per side using the Marshall hammer
- cool down the specimen and remove it from the mold

4.2.2 Dry Process (Generic RUMAC)

Marshall tests were conducted according to ASTM D 1559 using 50-blow compactive effort. Both dense and gap-graded aggregate gradations were used to prepare the Marshall samples. Both aggregate and CRM gradation used in Marshall test were summarized previously in Table 4-2.

Specimen Preparation Procedures

The mix design procedure used for generic dry process consists of the following steps:

- preparation of mixtures
    - weigh out ingredients (aggregates, CRM, and AC20)

- heat aggregate at 350° F and AC20 at 325° F
- dry mix aggregate and CRM for 15 seconds
- wet mix for about 1 minute 30 second

- prepare the compaction mold and the hammer (heated to 300^0 F)
- compact 50 blows per side using the Marshall hammer
- cool down the specimen and remove it from the mold

4.2.3 Open-graded Friction Course (OGFC)

Specimen preparation procedures used for the open-graded friction course mix design was specified in AASHTO: T 176 "Standard Method of Test for Compactive Strength of Bituminous Mixtures".

4.2.3.1 Apparatus

The pertinent apparatus used for mix design of OGFC is briefly summarized below.

Molds: Molding cylinder (7" height and 4" ID) and top (8" height and 4" ID) and bottom (2" height and 4" ID) molding plungers are used.

Supports: Temporary prismic steel bar supports (1" x 1" x 4") are used to raise the molding cylinder during the compression operation.

Testing machine: Universal testing machine is used to apply a vertical compressive pressure of 2000 psi.

Specimen Preparation Procedures

- Cool down the laboratory prepared mixture up to the molding temperature (255 ± 5^0 F) as quickly as possible after mixing (designed mixing temperature was 300^0 F).

- Place approximately one-half of the mixture in the molding cylinder.
- With the bottom plunger in place and molding cylinder supported temporarily on the two steel bars, the mixture shall be spaded vigorously twenty-five times with a preheated spatula with fifteen of the blows being delivered around the inside of the mold to reduce honeycomb, and the remaining ten at random over the mixture.
- Transfer the remaining half of the mixture to the molding cylinder and repeat a similar spading action.
- Compress the mixture between the top and bottom plunger under an initial load of about 150 psi to set the mixture against the sides of the mold. The support bars shall be removed to permit full double-plunger action and the entire molding load of 2000 psi is applied for 120 second.
- After removal from the mold, specimens are cured 24 hour at 140^0 F and thereafter brought to test temperature.

4.3 Mix Design Approaches

Marshall mix design method with medium traffic was adopted in this study. For open graded friction course mix design, FHWA Technical Advisory T5040.31 was used.

4.3.1 Marshall Mix Design

The Instron model 8500 material testing system with 20-kips capacity load cell was used to measure the stability and flow values of the Marshall mixtures. About 400 data points were recorded for each load versus deformation curve.

Properties that were evaluated from the Marshall tests were air void, voids in mineral aggregate (VMA), voids filled with asphalt (VFA), unit weight, stability, and flow value of Marshall sample described in the Asphalt Institute's Manual Series No. 2. For each binder content studied, three Marshall specimens were compacted and tested. Therefore, a total of fifteen specimens were prepared for each set of testing. Theoretical maximum specific gravity (ASTM D 2041) used in air void calculation was obtained from each binder content.

4.3.2 Marshall Mix Design Criteria

Wet Process

Two modifications in design criteria should be used for asphalt-rubber dense-graded HMA. First, due to the increased viscosity, elasticity, and softening point of the asphalt-rubber, HMA mixtures tend to experience less compaction and densification from traffic after construction. Therefore, for the dense-graded mixture containing the asphalt-rubber binder, the design air void level can be set at the low end of 3 to 5 percent range.

The second modification in the analysis of Marshall test results for determining the design binder content is that the maximum flow value can be raised to 24 for light traffic, 22 for medium traffic, and 20 for heavy traffic due to the higher binder contents that are typically required, and due to the flatter slope of the load versus deformation curve from the Marshall stability test results.

Dry Process

Since there were no standard mix design criteria available for dry process, all conventional Marshall design parameters were shown in Table 4-4.

Table 4-4 Marshall mix design criteria (MS-2 6th edition, 1993)

Marshall Criteria	Light Traffic		Medium Traffic		Heavy Traffic	
	Min	Max	Min	Max	Min	Max
Compaction	35 blows		50 blows		75 blows	
Stability	750 lb		1200 lb		1800 lb	
Flow	8	18	8	16	8	14
VMA, %	14*		14*		14*	
Air void, %	3	5	3	5	3	5
VFA, %	70	80	65	78	65	75

*For 4% air voids with 1/2 in. nominal maximum particle size

4.3.3 Open-graded Friction Course Mix Design Method

The modified physical properties of high CRM content asphalt-rubber permit its use in a variety of manners as OGFC. Due to the increased viscosity of asphalt-rubber, binder contents of up to approximately 10 percent can be used without experiencing excessive drain off. The higher binder contents produce thicker binder films which increase mixture aging resistance and durability.

The asphalt-rubber used in OGFC mix design is identical to those used in the wet process: AC5 with 15% of WRF30. The recommended gradation for asphalt-rubber-aggregate mixtures (International Surfacing Inc., 1992) and JMF have been discussed previously in Table 4-3.

Determination of AR Binder Content

Specific steps for determining the optimum AR binder content can be summarized as follows.

- Step 1: Determine the surface content K_c of the aggregate using the FHWA T-5050.31 procedure (oil soaking and drain off).
- Step 2: Calculate the required asphalt cement using the following formula: $(AC)_{JMF} = (2.0 K_c + 4.0) * 2.65/S.G_{AGG}$ where, $S.G_{AGG}$: specific gravity of aggregate.
- Step 3: Determine the base asphalt-rubber content by dividing the percentage of asphalt from step 2 by the fractional asphalt cement content of the asphalt-rubber. This provides an asphalt cement content in the asphalt-rubber which is equivalent to that determined in step 2.

4.4 Results of Marshall Mix Design

4.4.1 Wet Process

AC5 with 15% of WRF 30 and AC10 with 10%WRF30 were chosen as an asphalt-rubber binder for dense-graded asphalt-rubber-aggregate mixtures. AC5 with 15% of WRF30 was also chosen as an asphalt rubber for gap-graded asphalt-rubber-aggregates mixture.

(i) Conventional Dense-graded HMA with AC20

A four percent design air void for medium design traffic was used in selecting optimum asphalt cement content. Also, all other design criteria specified in the Asphalt Institute MS-2 were satisfied. Fig. 4-4 to 4-9 shows the Marshall mix design results for the conventional dense-graded mixture with AC20. The pertinent test results and optimum asphalt contents (by weight of

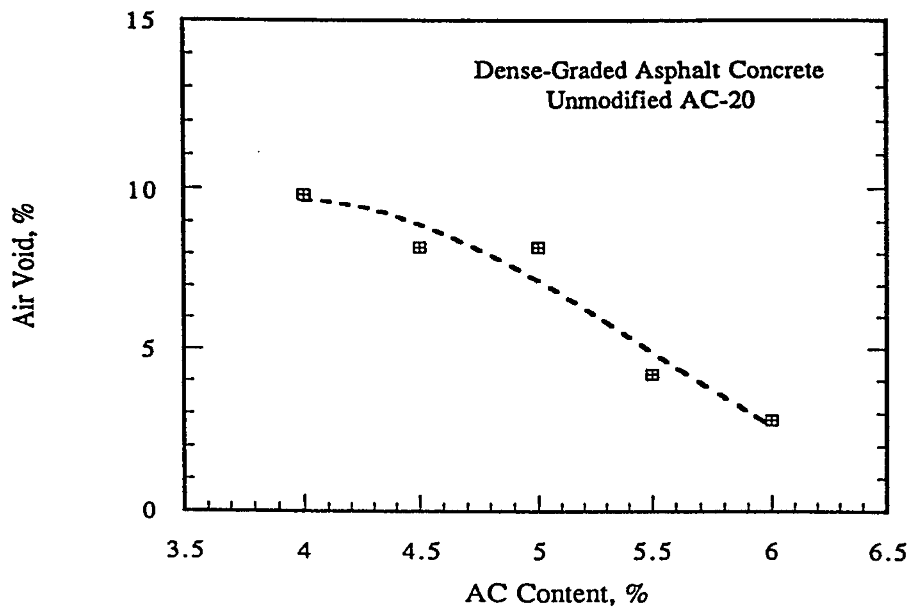


Fig. 4-4 Air void versus AC content for dense-graded HMA with AC20

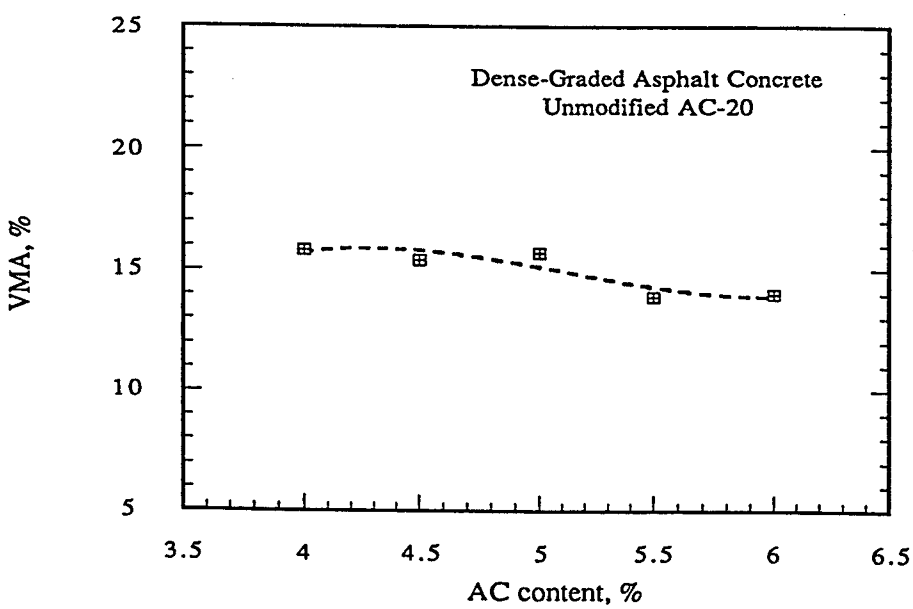


Fig. 4-5 VMA versus AC content for dense-graded HMA with AC20

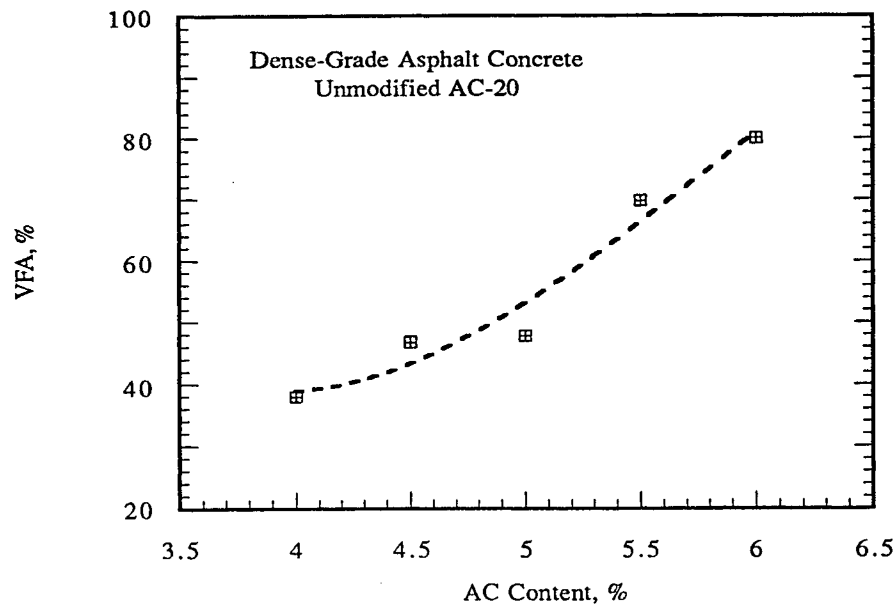


Fig. 4-6 VFA versus AC content for dense-graded HMA with AC20

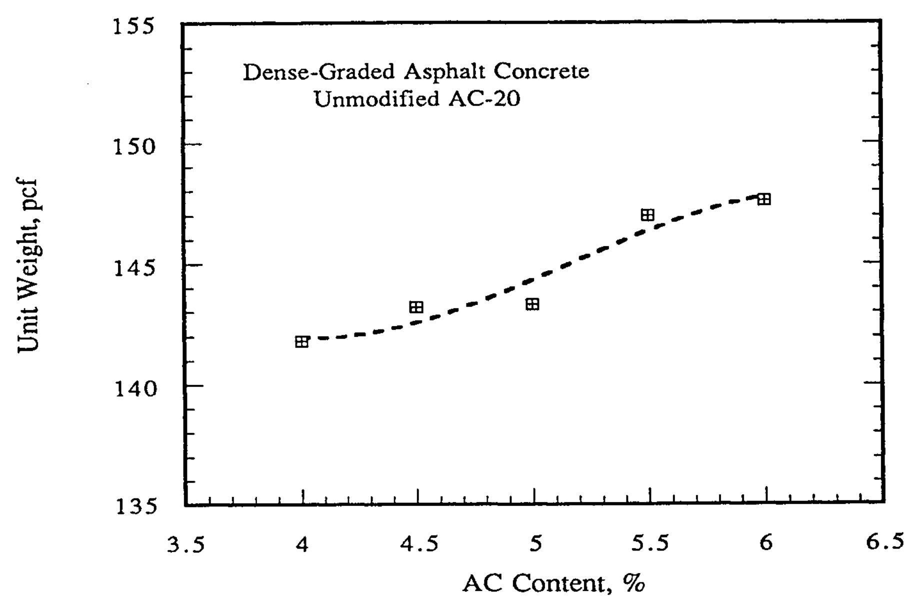


Fig. 4-7 Unit weight versus AC content for dense-graded HMA with AC20

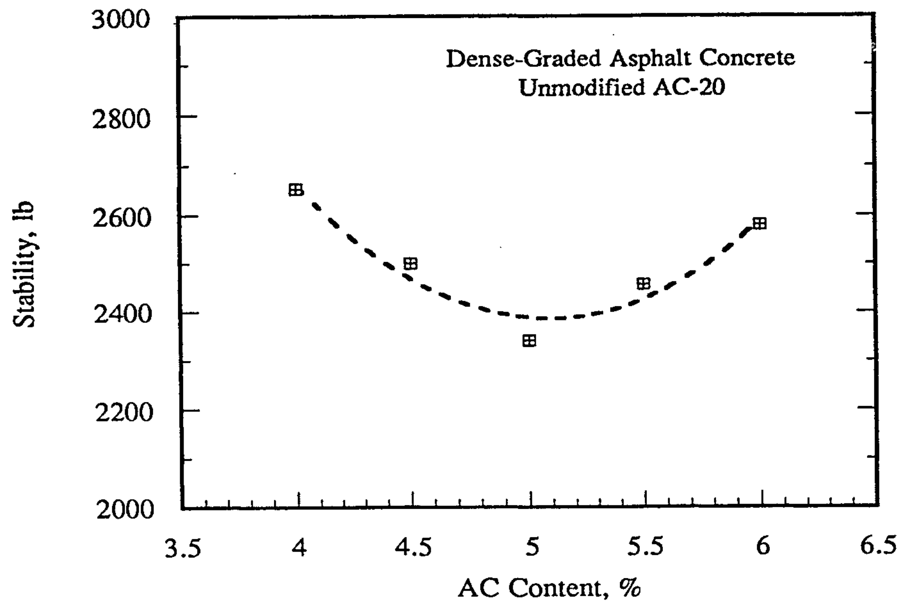


Fig. 4-8 Stability versus AC content for dense-graded HMA with AC20

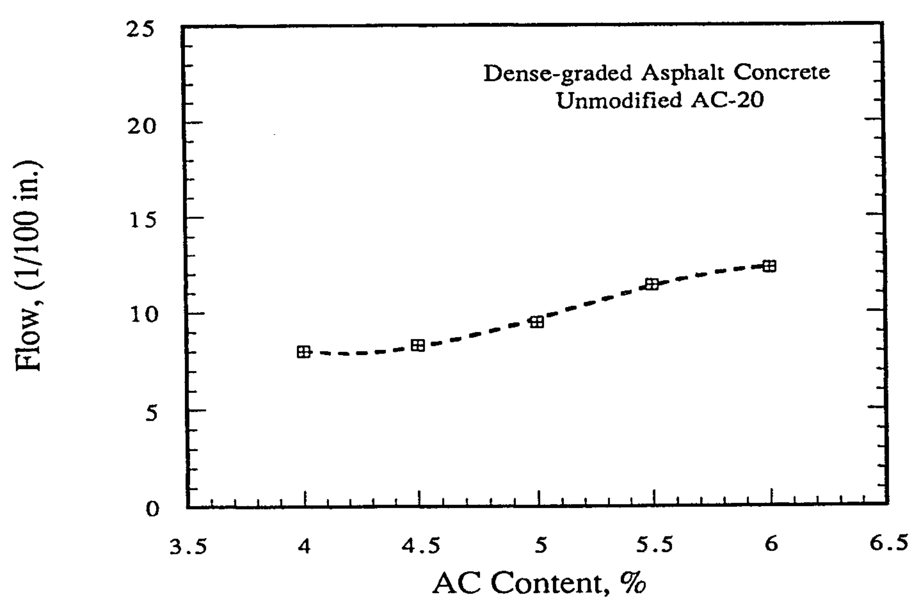


Fig. 4-9 Flow versus AC content for dense-graded HMA with AC20

mix) are shown in Table 4-5. Note that the values reported in Table 4-5 represent an average of three test specimens. Optimum asphalt cement content was determined as 5.7 percent. At the four percent air voids, the mixture properties are summarized in Table 4-6.

Table 4-5 Average of Marshall test data for dense-graded HMA with AC20

Binder Content	Air Void %	VMA %	VFA %	Unit Wt. pcf	Stability lb	Flow 0.01 in
Dense-graded with AC20: Optimum content (5.7%)						
4.0 %	9.8	15.8	38.0	141.8	2652	8.0
4.5 %	8.2	15.4	46.8	143.2	2500	8.3
5.0 %	8.2	15.7	47.8	143.3	2341	9.5
5.5 %	4.2	13.9	69.8	147.0	2457	11.4
6.0 %	2.8	14.0	80.0	147.6	2580	12.3

Table 4-6 Marshall mixture properties at optimum AC content

(Conventional dense-graded HMA with AC20)

Optimum Asphalt Content, %	5.7
VMA, %	14
VFA, %	70
Unit Weight, pcf	147.3
Stability, lb	2470
Flow, 0.01 in.	11.8

(ii) Dense-graded HMA with Ecoflex

Fig. 4-10 to 4-15 shows the Marshall test results for the dense-graded mix with Ecoflex. A four percent design air void with medium design traffic was used in selecting the optimum asphalt cement content. The other conventional Marshall design criteria were all satisfied. Test results

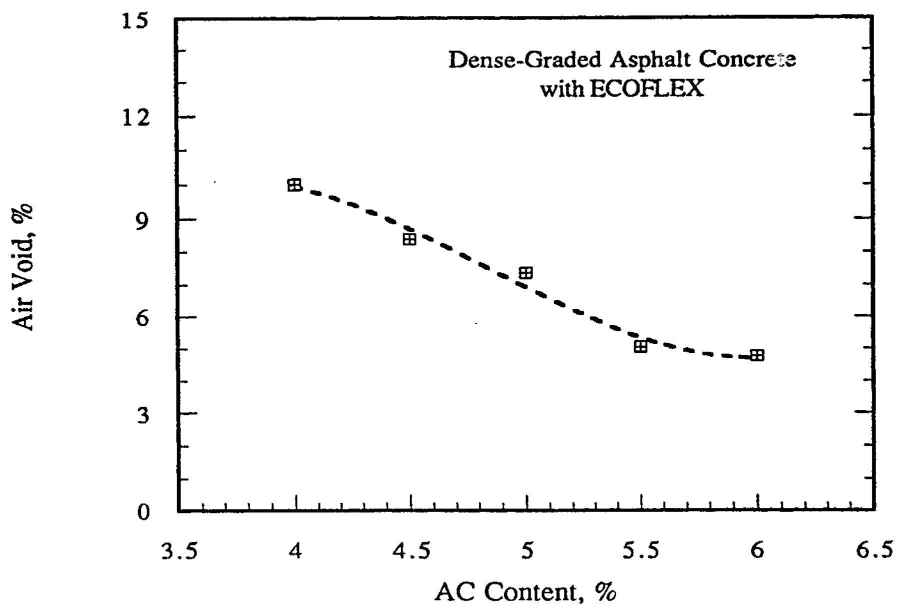


Fig. 4-10 Air void versus binder content for dense-graded HMA with Ecoflex

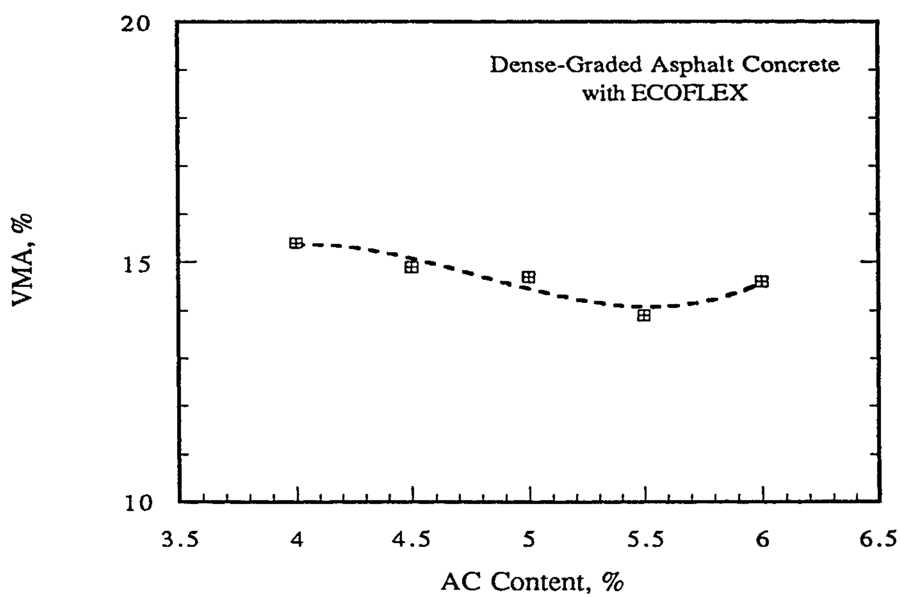


Fig. 4-11 VMA versus binder content for dense-graded HMA with Ecoflex

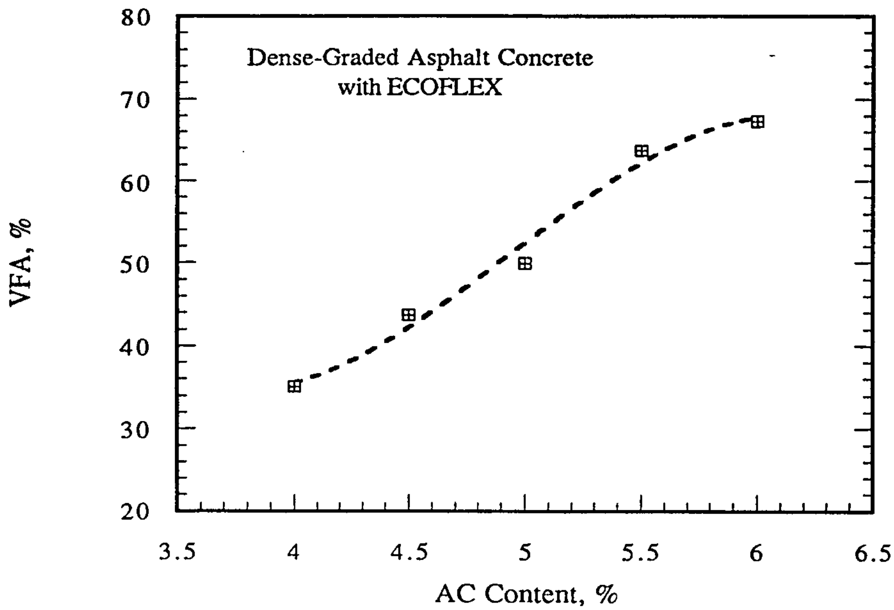


Fig. 4-12 VFA versus binder content for dense-graded HMA with Ecoflex

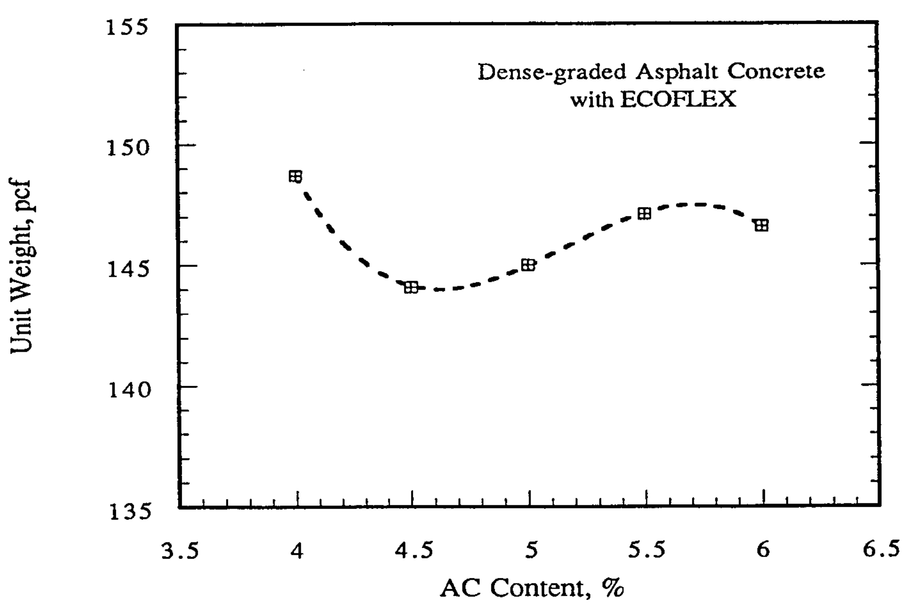


Fig. 4-13 Unit weight versus binder content for dense-graded HMA with Ecoflex

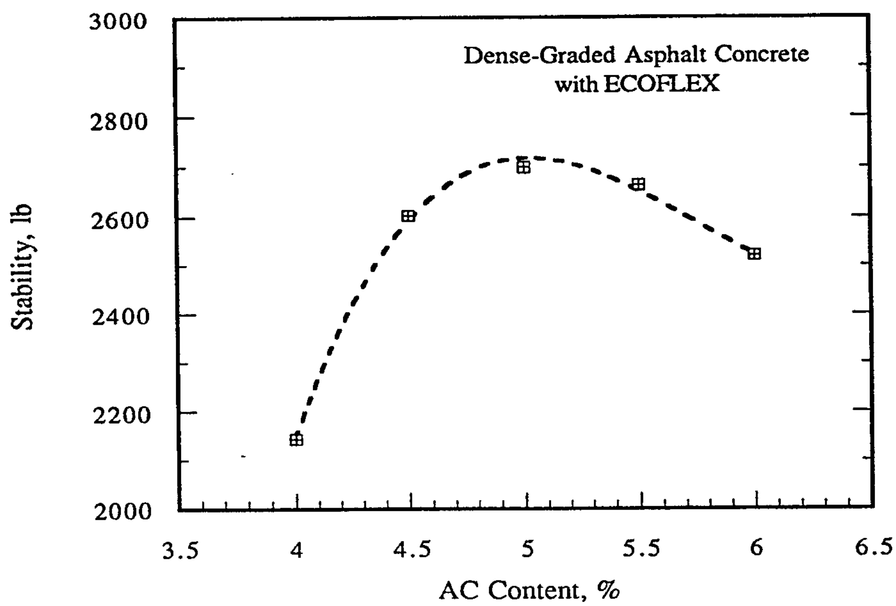


Fig. 4-14 Stability versus binder content for dense-graded HMA with Ecoflex

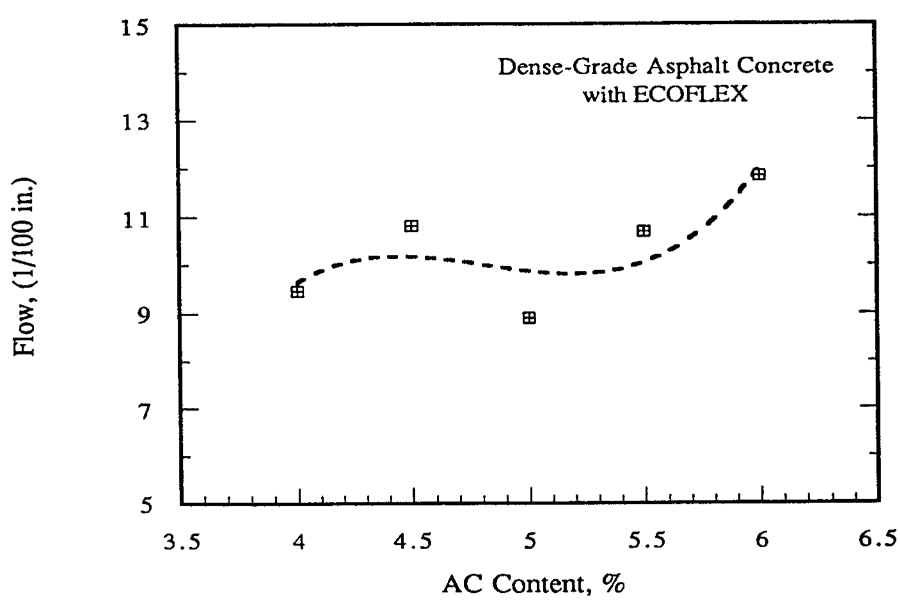


Fig. 4-15 Flow versus binder content for dense-graded HMA with Ecoflex

are summarized in Table 4-7. Optimum binder content was determined as 7.0 percent. At four percent air voids, the mixture properties are summarized in Table 4-8.

Table 4-7 Average of Marshall test data for dense-graded HMA with Ecoflex

Binder Content	Air Void %	VMA %	VFA %	Unit Wt. pcf	Stability lb	Flow 0.01 in
Dense-graded with Ecoflex: Optimum content (7.0%)						
4.5 %	8.39	14.9	43.7	144.1	2602	9.6
5.0 %	7.36	14.7	49.9	145.0	2699	7.8
5.5 %	5.05	13.9	63.7	147.1	2664	10.2
6.0 %	4.77	14.6	67.3	146.6	2520	10.7
7.0 %	4.00	17.4	77.0	144.0	2292	13.6

Table 4-8 Marshall mixture properties at optimum binder content (Ecoflex)

Optimum Binder Content, %	7.0
VMA, %	17.4
VFA, %	77.0
Unit Weight, pcf	144.0
Stability, lb	2292
Flow, 0.01 in.	13.6

(iii) Dense-graded Asphalt-Rubber Concrete: AC5 with 15% of WRF 30

Fig. 4-16 to 4-21 show the Marshall mix design results for the dense-graded asphalt-rubber-aggregate mixture (AC5+15%WRF30). A four percent design air void criterion was used along with the modified Marshall criterion (maximum flow of 22 for medium traffic). Test results are collected in Table 4-9. Optimum asphalt-rubber content was determined as 7.2 percent. At four percent air voids, the mixture properties are summarized in Table 4-10.

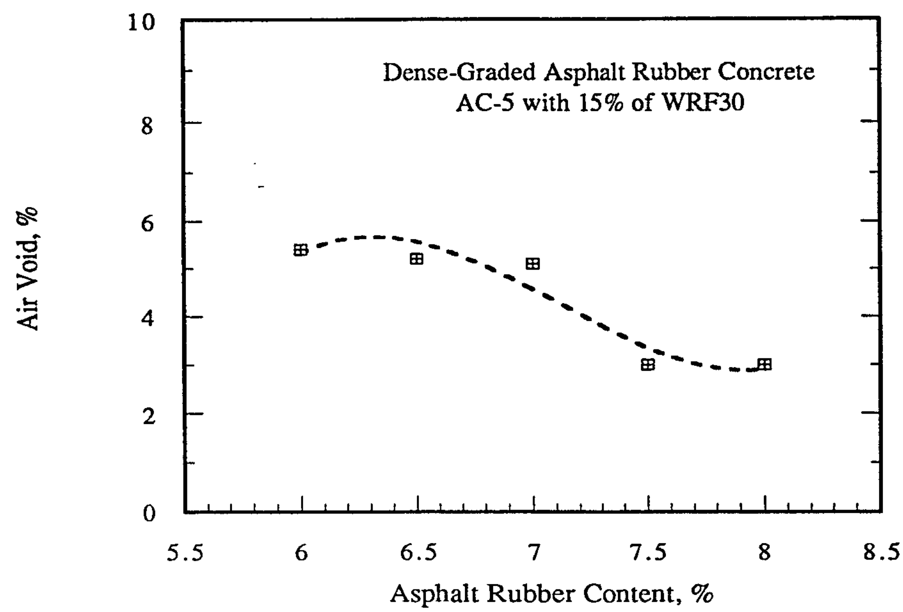


Fig. 4-16 Air void versus binder content for dense-graded HMA with AC5+15%WRF30

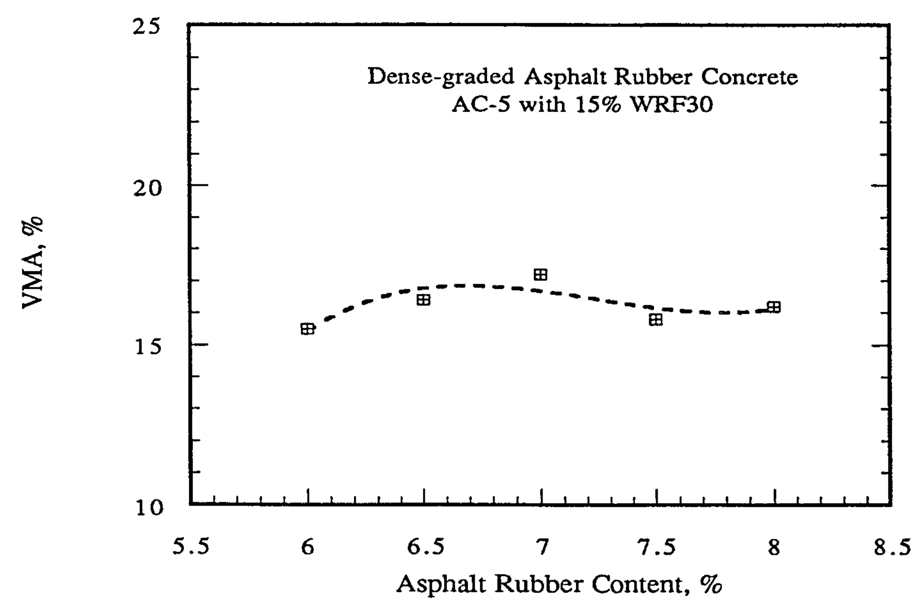


Fig. 4-17 VMA versus binder content for dense-graded HMA with AC5+15%WRF30

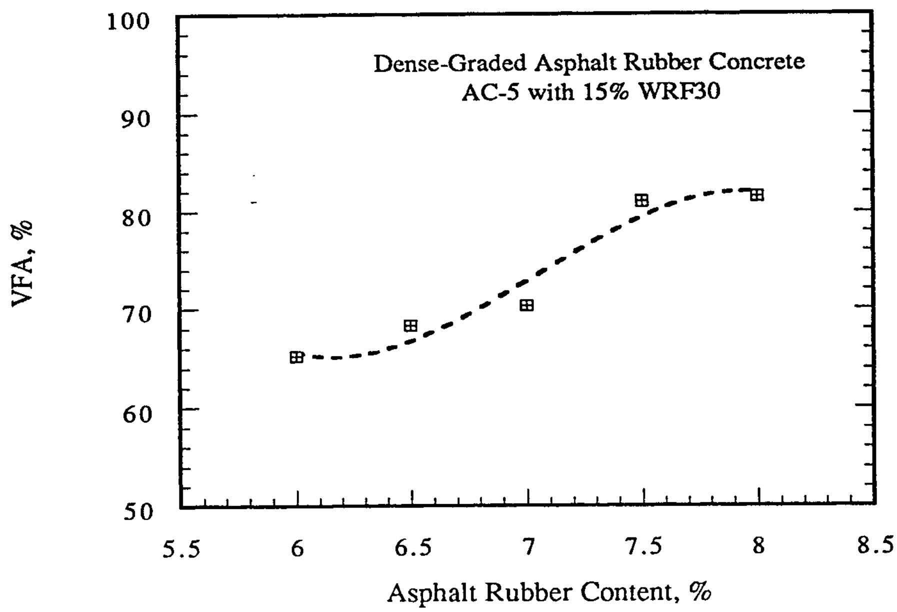


Fig. 4-18 VFA versus binder content for dense-graded HMA with AC5+15%WRF30

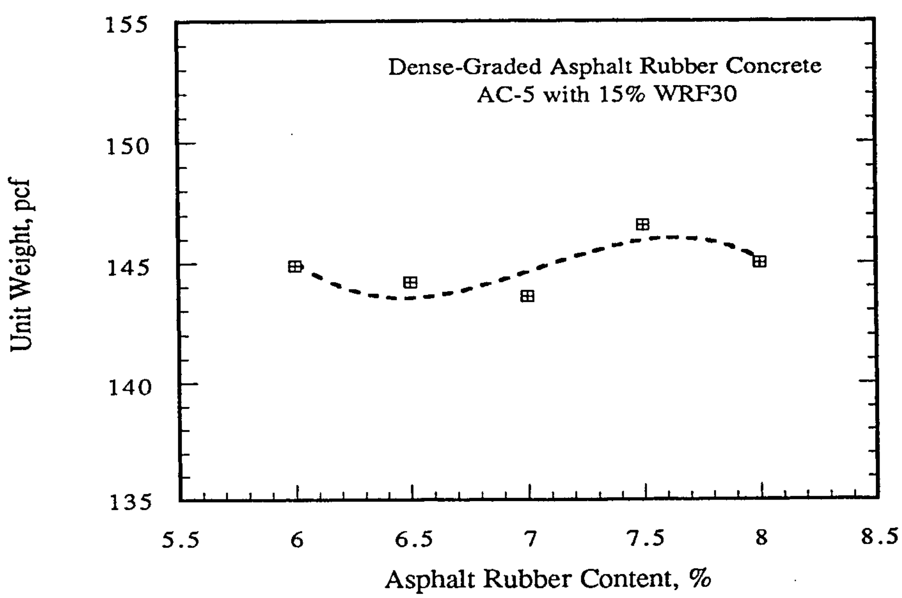


Fig. 4-19 Unit Weight versus binder content for dense-graded HMA with AC5+15%WRF30

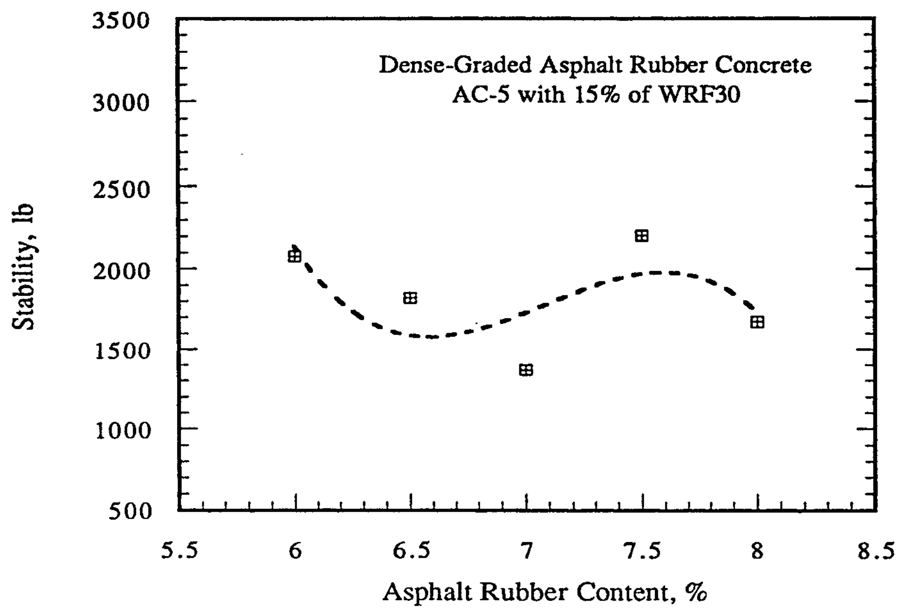


Fig. 4-20 Stability versus binder content for dense-graded HMA with AC5+15%WRF30

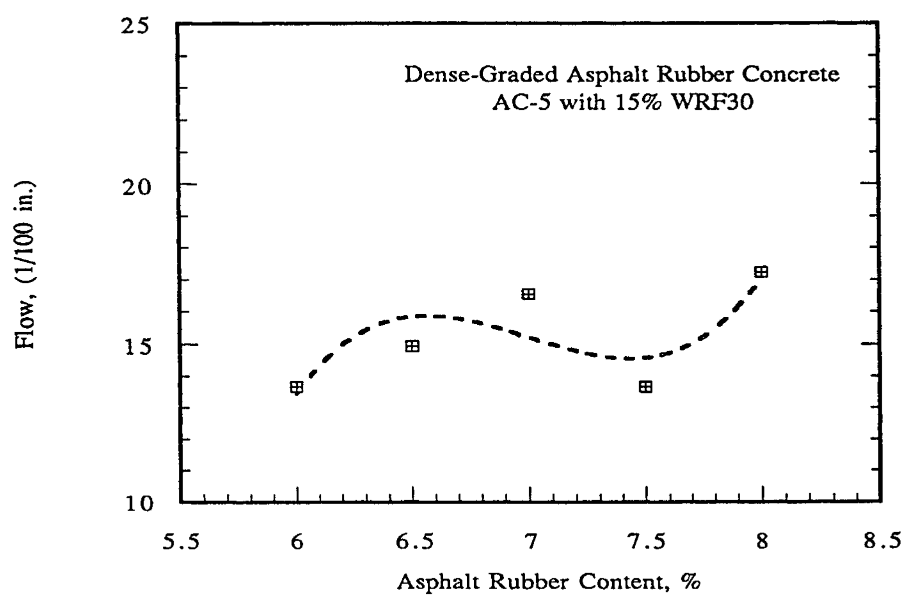


Fig. 4-21 Flow versus binder content for dense-graded HMA with AC5+15%WRF30

Table 4-9 Average of Marshall test data for dense-graded HMA with asphalt-rubber AC5+15%WRF30

Binder Content	Air Void %	VMA %	VFA %	Unit Wt. pcf	Stability lb	Flow 0.01 in
Dense-graded with AC5/15%WRF30: Optimum content (7.2%)						
0.06	5.4	15.5	65.2	144.9	2076	10.7
0.065	5.2	16.4	68.3	144.2	1820	13.2
0.07	5.1	17.2	70.3	143.6	1369	13.7
0.075	3	15.8	81	146.6	2203	12.5
0.08	3	16.2	81.5	145	1671	15.1

Table 4-10 Marshall mixture properties at optimum AR content (AC5/15%WRF30)

Optimum AR Content, %	7.2
VMA, %	16.5
VFA, %	75.0
Unit Weight, pcf	145.1
Stability, lb	1850
Flow, 0.01 in.	15.0

(iv) Dense-graded Asphalt-Rubber Concrete: AC5 with 10% of GY

(Continuous Blending Technology: Florida Process)

Fig. 5-22 to 4-27 present the Marshall mix design test results for the dense-graded mix using continuous blending technology. Ten percent of GY (Ultrafine CRM) with AC5 (AC5+10%GY) was used for Marshall mix design. The design criterion of four percent airvoid is also used here. Test results are shown in Table 4-11. At the four percent air voids, the mixture properties are summarized in Table 4-12.

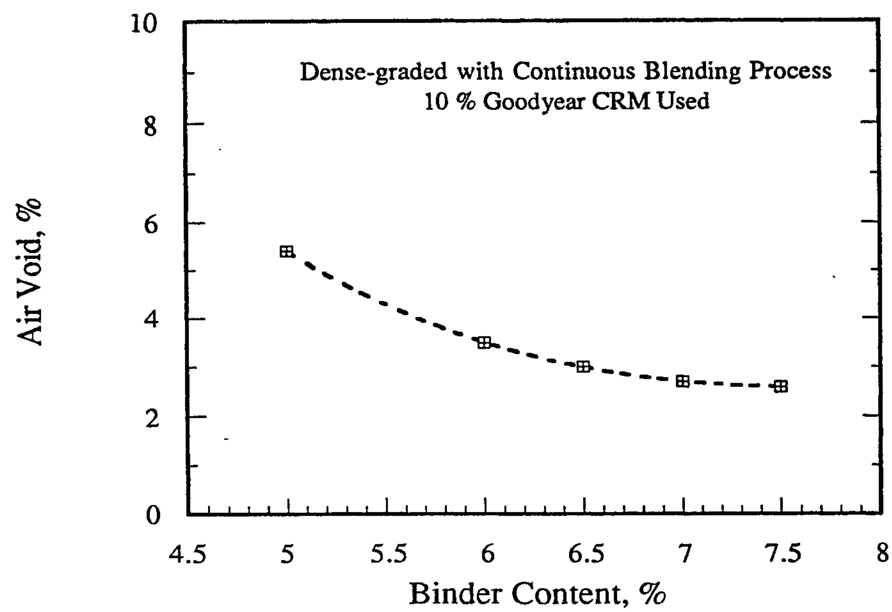


Fig. 4-22 Air void versus binder content for dense-graded HMA with AC5/10%GY

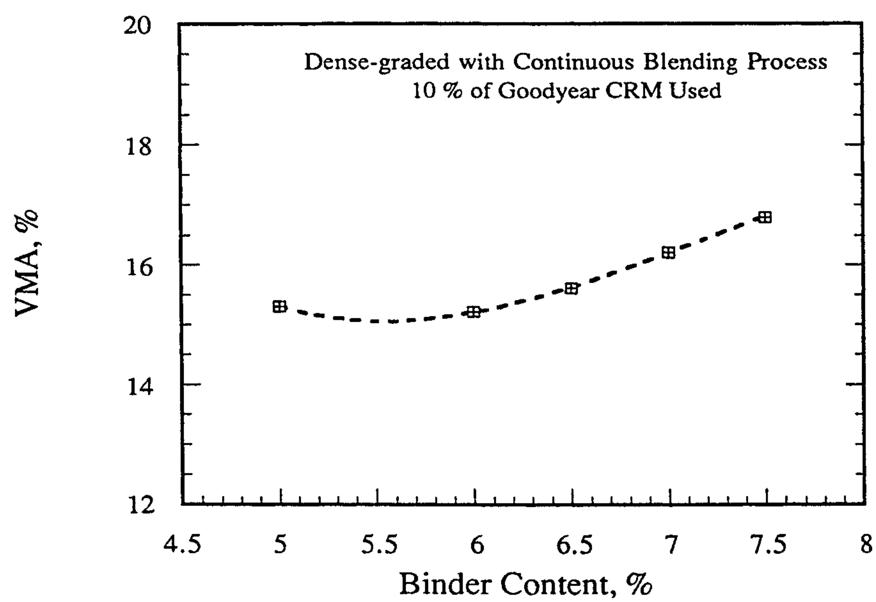


Fig. 4-23 VMA versus binder content for dense-graded HMA with AC5/10%GY

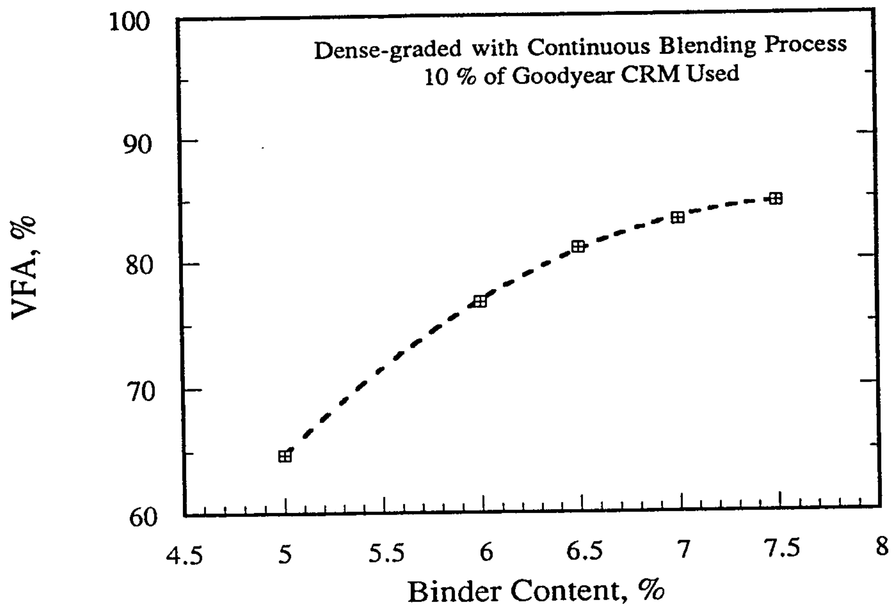


Fig. 4-24 VFA versus binder content for dense-graded HMA with AC5/10%GY

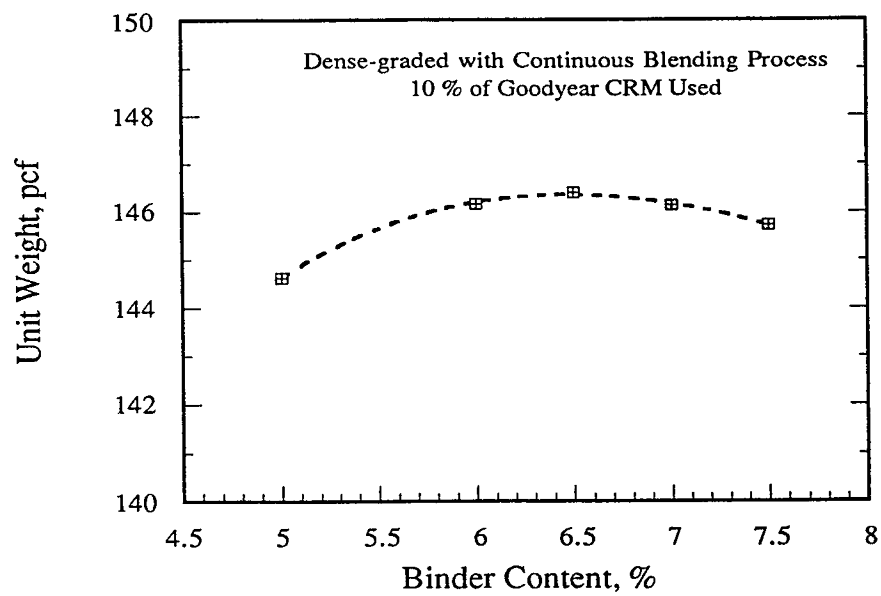


Fig. 4-25 Unit Weight versus binder content for dense-graded HMA with AC5/10%GY

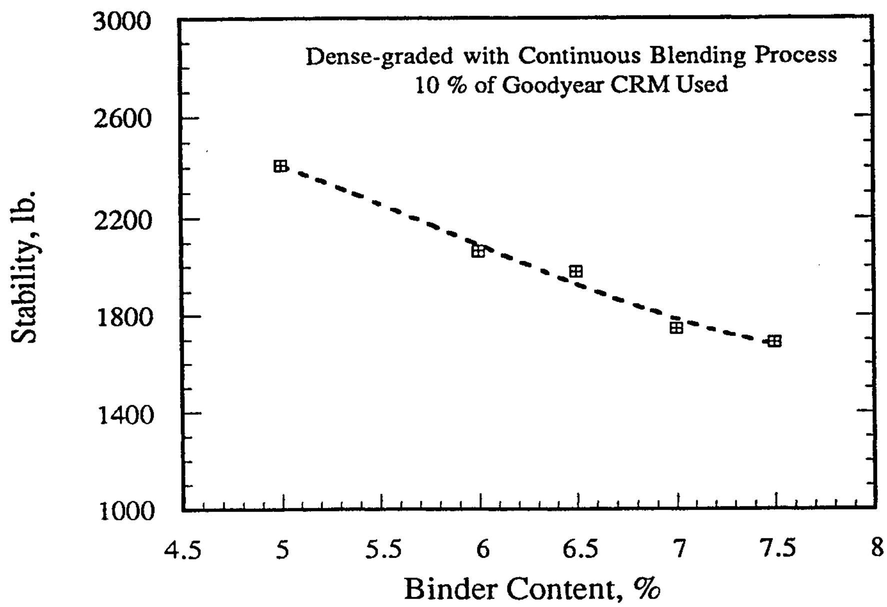


Fig. 4-26 Stability versus binder content for dense-graded HMA with AC5/10%GY

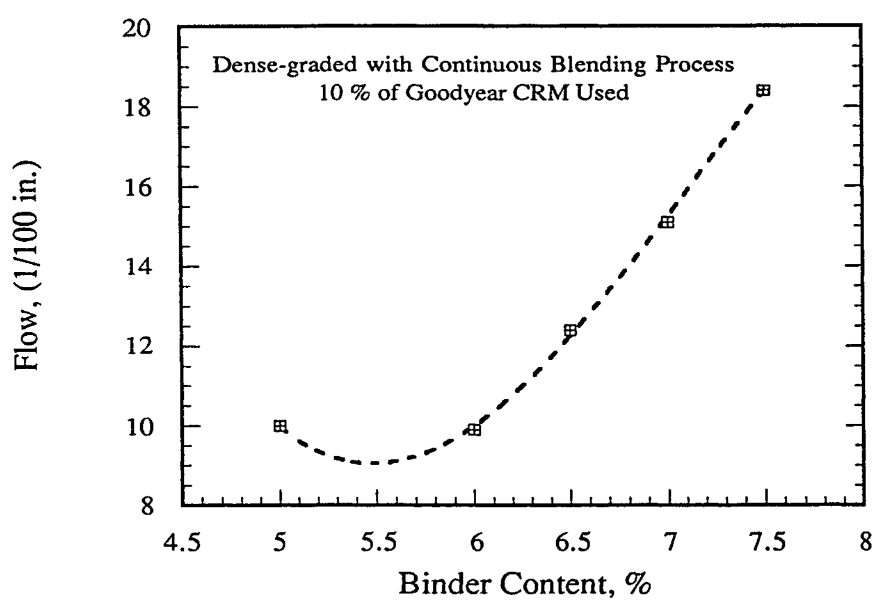


Fig. 4-27 Flow versus binder content for dense-graded HMA with AC5/10%GY

Table 4-11 Average of Marshall test data for dense-graded HMA with AC5/10%GY

Binder Content	Air Void %	VMA %	VFA %	Unit Wt. pcf	Stability lb	Flow 0.01 in
Dense-graded with AC5/10%GY: Optimum content (5.6%)						
5.0 %	5.4	15.3	64.7	144.63	2408	10.0
6.0 %	3.5	15.2	76.7	146.18	2064	9.9
6.5 %	3.0	15.6	81.0	146.40	1980	12.4
7.0 %	2.7	16.2	83.3	146.14	1747	15.1
7.5 %	2.6	16.8	84.7	145.73	1688	18.4

Table 4-12 Marshall mixture properties at optimum binder content (AC5/10%GY)

Optimum Binder Content, %	5.6
VMA, %	15.3
VFA, %	73
Unit Weight, pcf	145.8
Stability, lb	2210
Flow, 0.01 in.	9

(v) Dense-graded Asphalt-Rubber Concrete: AC10 with 10%WRF30

The Marshall mix design results for the dense-graded asphalt-rubber mixture (AC10+10%WRF30) were plotted in Fig. 4-28 to 4-33 and summarized in Table 5-13. A 3.5% airvoid design criterion was used along with modified Marshall criterion (maximum flow of 22 for medium traffic). Optimum binder content was determined as 8.3%. At 3.5% airvoid, the mixture properties are summarized in Table 4-14.

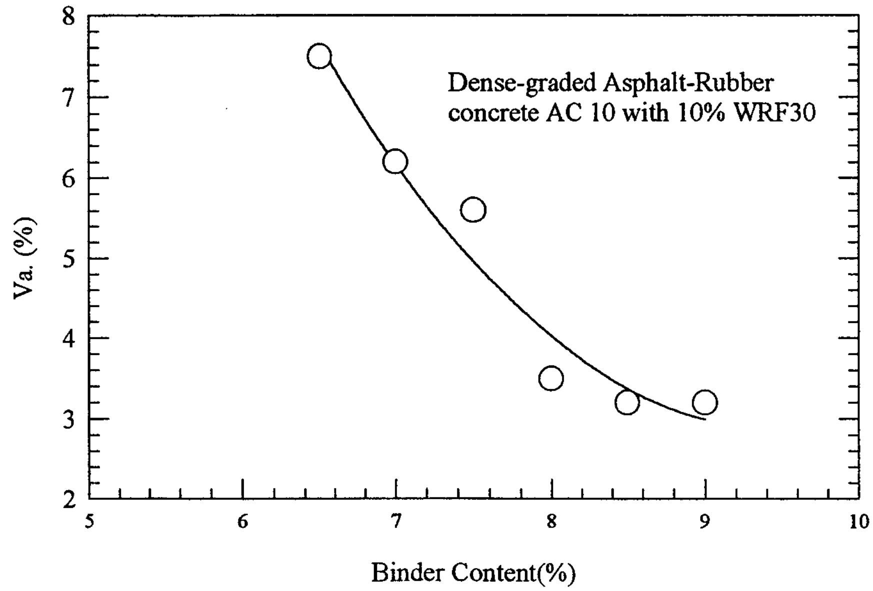


Fig. 4-28 Va. vs. AR content for dense-graded HMA with AC 10/10% WRF30

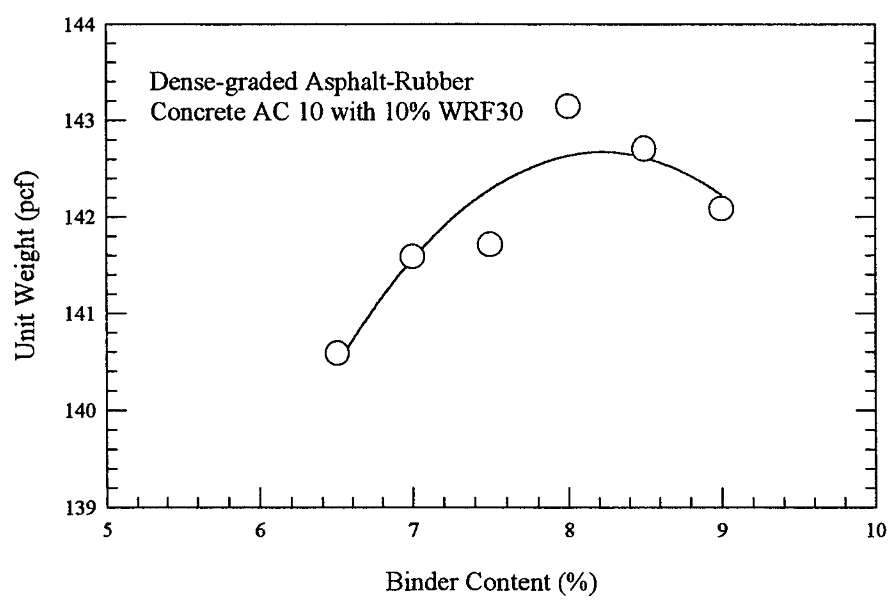


Fig. 4-29 Unit weight vs. AR content for dense-graded HMA with AC10/10% WRF30

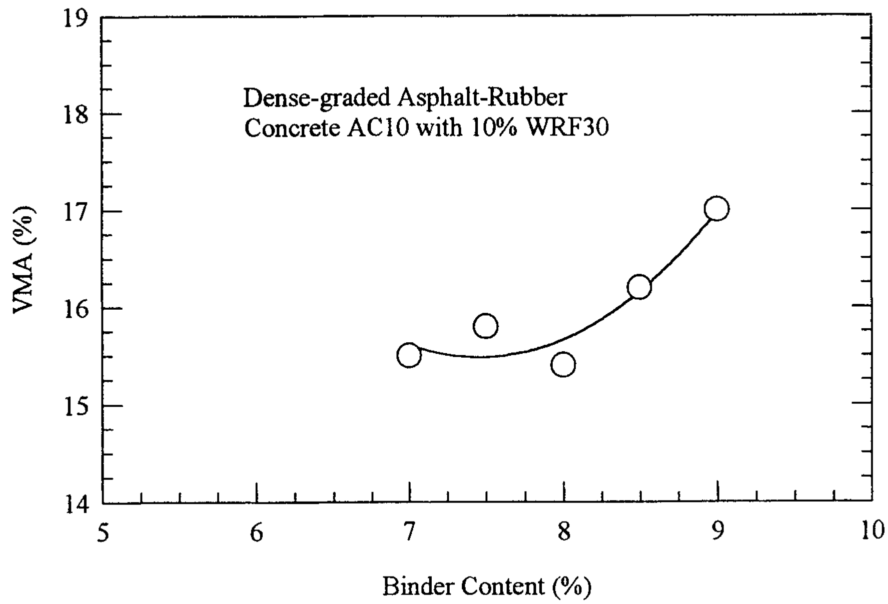


Fig. 4-30 VMA vs. AR content for dense-graded HMA with AC10/10% WRF30

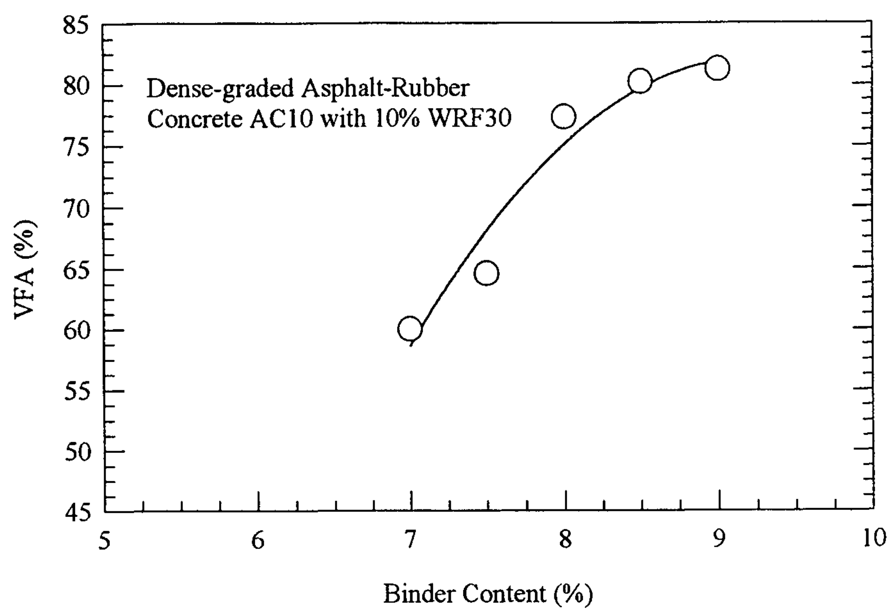


Fig. 4-31 VFA vs. AR content for dense-graded HMA with AC10/10% WRF30

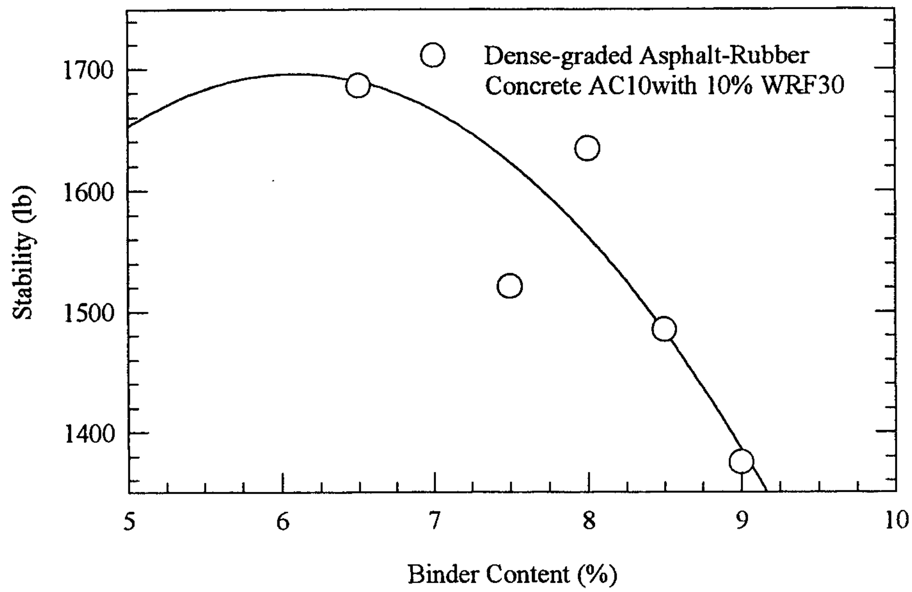


Fig. 4-32 Stability vs. AR content for dense-graded HMA with AC 10/10% WRF30

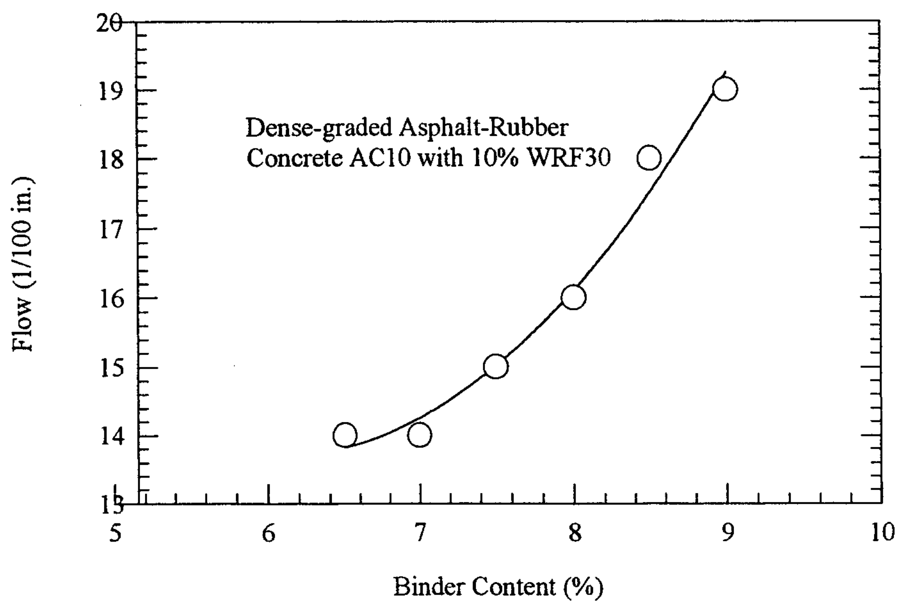


Fig. 4-33 Flow vs. AR content for dense-graded HMA with AC10/10% WRF30

Table 4-13 Average of Marshall test data for dense-graded HMA with asphalt-rubber AC10+10%WRF30

Binder content %	Air void %	VMA %	VFA %	Unit Wt. Pcf	Stability lb	Flow 0.01 in
Dense-graded with AC10/10%WRF30: Optimum binder content 8.3%						
6.5	7.5	15.6	51.9	140.6	1686	14
7	6.2	15.5	60.0	141.6	1711	14
7.5	5.6	15.8	64.6	141.7	1521	15
8	3.5	15.4	77.3	143.1	1634	16
8.5	3.2	16.2	80.2	142.7	1485	18
9	3.2	17.0	81.2	142.1	1375	19

Table 4-14 Marshall mixture properties at optimum AR content (AC10+10%WRF30)

Optimum AR content, %	8.3
VMA, %	16.1
VFA, %	77
Unit Wt. Pcf	142.7
Stability, lb	1520
Flow, 0.01in.	17

(vi) Gap-graded Asphalt-Rubber Concrete: AC5 with 15 % of WRF 30

Marshall mix design tests for gap-graded mix with AC5/15%WRF30 are conducted with a four percent airvoid criterion. Test results are shown in Fig. 4-34 to 4-39 and summarized in Table 4-15. Optimum binder content was determined to be 8.4 percent. At the optimum binder content, the mixture properties are summarized in Table 4-16. It can be seen that all other Marshall criteria have been met as well.

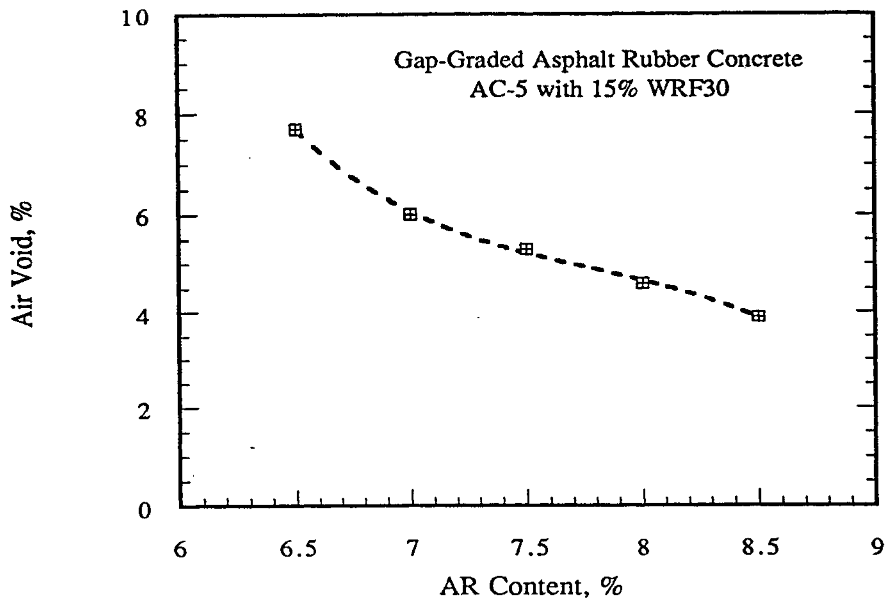


Fig. 4-34 Air void versus binder content for gap-graded HMA with AC5+15%WRF30

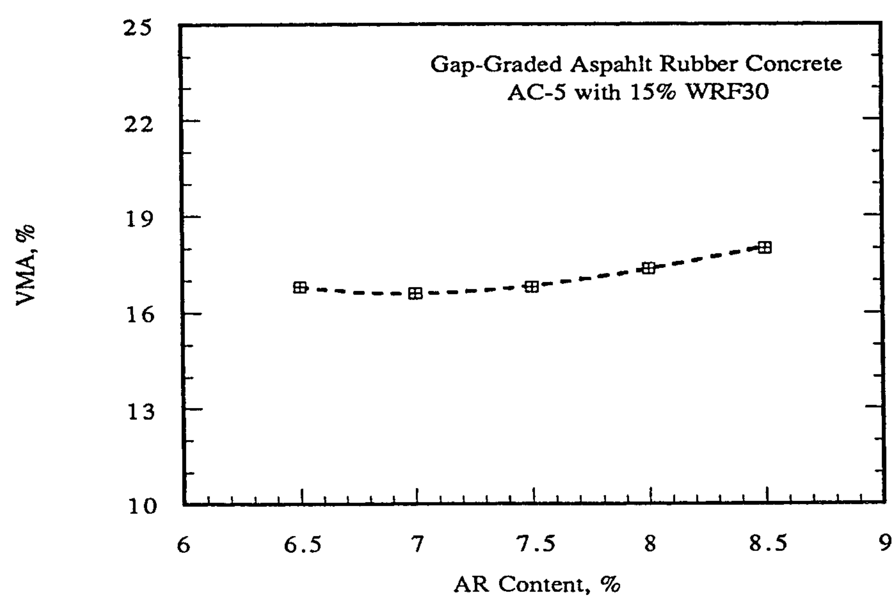


Fig. 4-35 VMA versus binder content for gap-graded HMA with AC5+15%WRF30

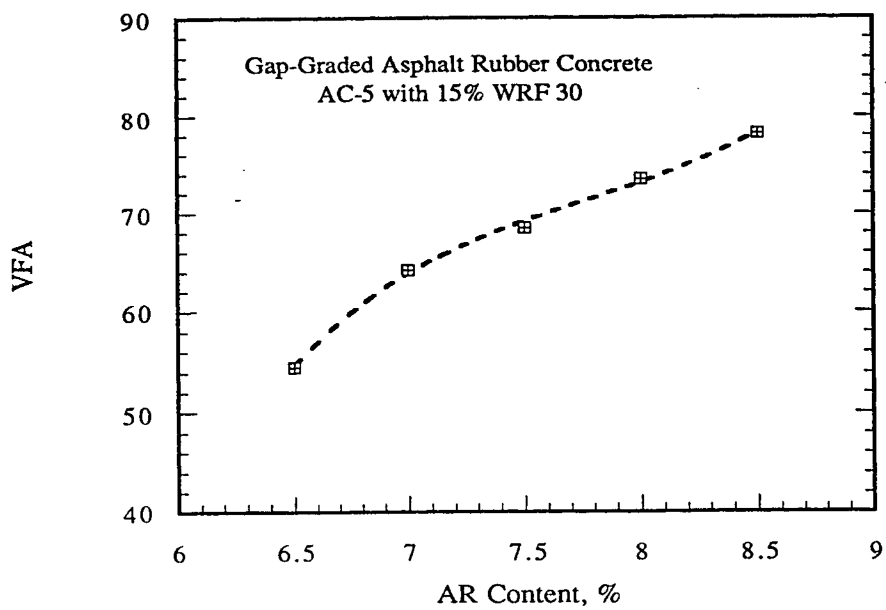


Fig. 4-36 VFA versus binder content for gap-graded HMA with AC5+15%WRF30

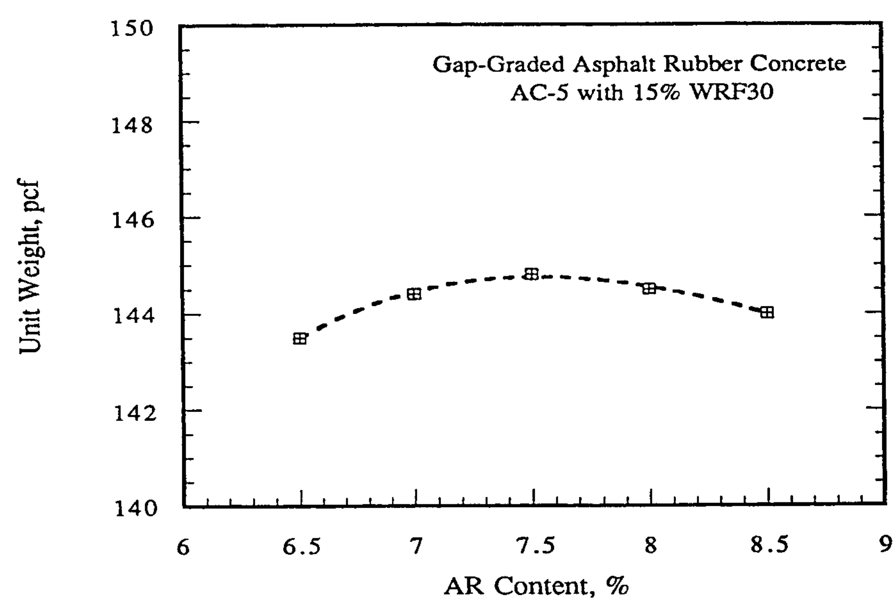


Fig. 4-37 Unit weight versus binder content for gap-graded HMA with AC5+15%WRF30

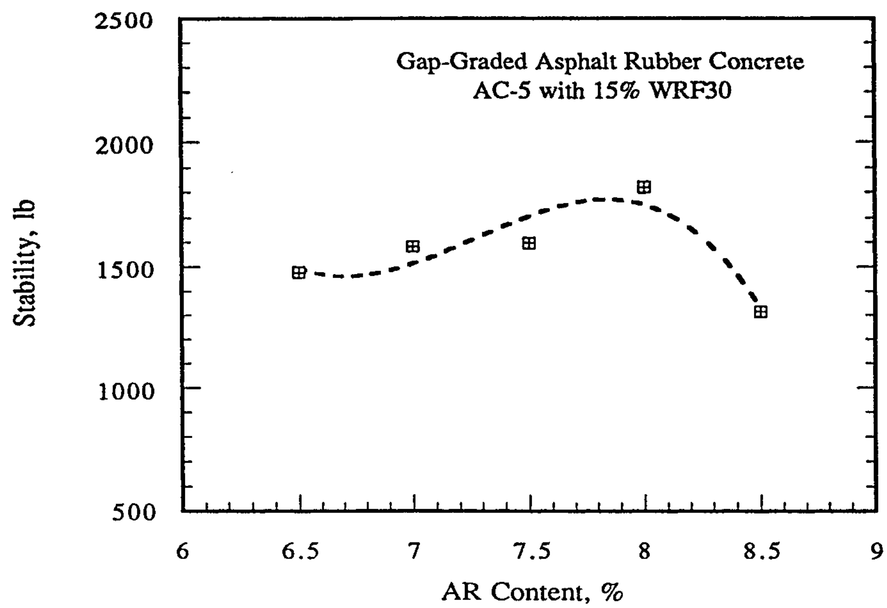


Fig. 4-38 Stability versus binder content for gap-graded HMA with AC5+15%WRF30

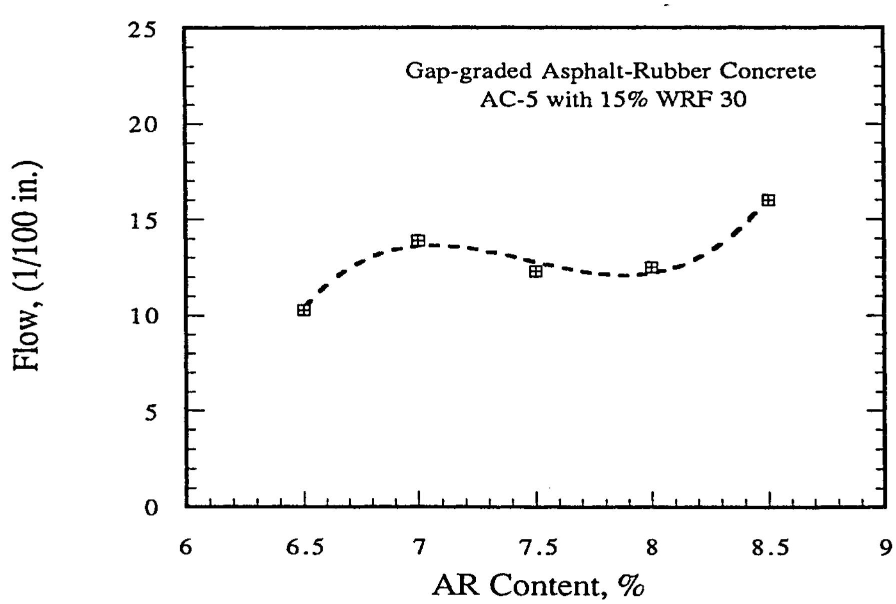


Fig. 4-39 Flow versus binder content for gap-graded HMA with AC5+15%WRF30

Table 4-15 Average of Marshall test data for gap-graded HMA with (AC5/15%WRF30)

Binder Content	Air Void %	VMA %	VFA %	Unit Wt. pcf	Stability lb	Flow 0.01 in
Gap-graded with AC5/15%WRF30: Optimum content (8.4%)						
0.065	7.7	16.8	54.5	143.5	1478	10.3
0.07	6	16.6	64.2	144.4	1582	13.9
0.075	5.3	16.8	68.5	144.8	1597	12.3
0.08	4.6	17.36	73.5	144.5	1821	12.5
0.085	3.9	18	78.2	144	1315	16

Table 4-16 Marshall mixture properties at optimum AR content
(AC5/15%WRF30, gap-graded gradation)

AR Content, %	8.4
VMA, %	17.8
VFA, %	77.4
Unit Weight, pcf	144.2
Stability, lb	1480
Flow, 0.01 in.	15.0

4.4.2 Generic Dry Process

Marshall mix design tests were conducted on both dense-graded and gap-graded aggregate gradations used for the generic dry process. Two percent of CRM (by the weight of aggregate) was used for the dense-graded as well as the gap-graded mixture. The gradation of CRM used in this process was shown previously in Table 4-2. Three percent of CRM was only used for gap-graded mixture. AC-20 was used as the asphalt cement binder.

(i) Dense-graded Generic RUMAC

Fig. 4-40 to Fig. 4-45 show the Marshall test results for the dense-graded RUMAC with 2%CRM. Four percent air void was the only criterion used for determining the optimum AC content, which was determined as 9.5 percent by the weight of aggregate plus CRM. The average of the test result and the optimum binder contents (by weight of aggregate) of each mix as determined by the Marshall tests are summarized in Table 4-17. The mixture properties at optimum binder content are summarized in Table 4-18.

Table 4-17 Average of test data from Marshall mix design (generic dry process) (dense-graded gradation with 2 % CRM)

Binder Content	Air void	VMA %	VFA %	Unit weight	Stab. lb	Flow 0.01 in.
Dense-graded with 2% CRM: Optimum AC content (9.5%)						
6.5%	10.7	22.5	52.4	133.72	1090	25.7
7.5%	8.5	22.3	61.9	135.16	1057	25.9
8.5%	7.2	23.2	69.0	137.59	1056	26.7
9.5%	3.9	22.4	82.6	137.38	1034	25.5
10.5%	1.3	22.2	94.1	139.03	1156	29.5

Table 4-18 Marshall mixture properties at optimum AC content (generic dry process)

AC Content, %	9.5
VMA, %	22.6
VFA, %	82.5
Unit Weight, pcf	137.9
Stability, lb	1045
Flow, 0.01 in.	26

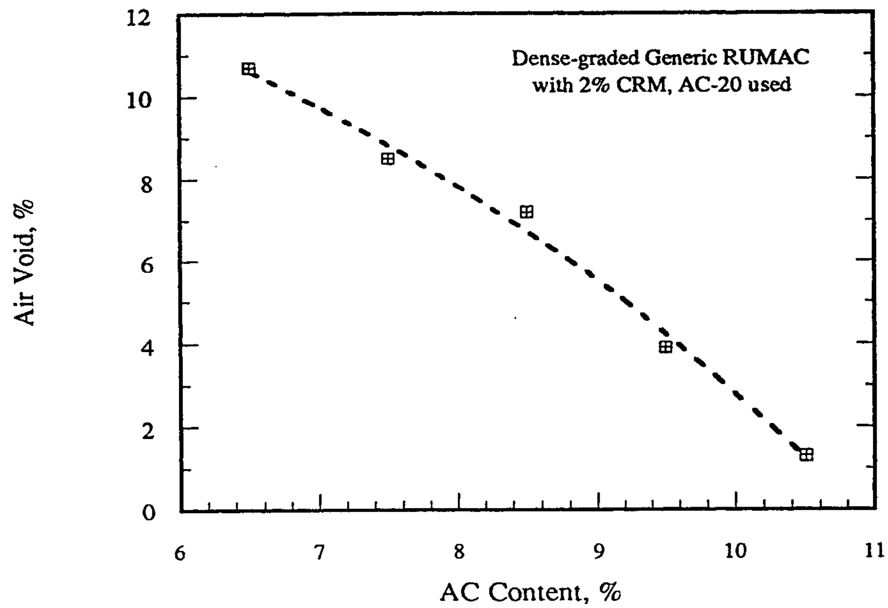


Fig. 4-40 Air void versus AC content for dense-graded HMA with 2%RUMAC

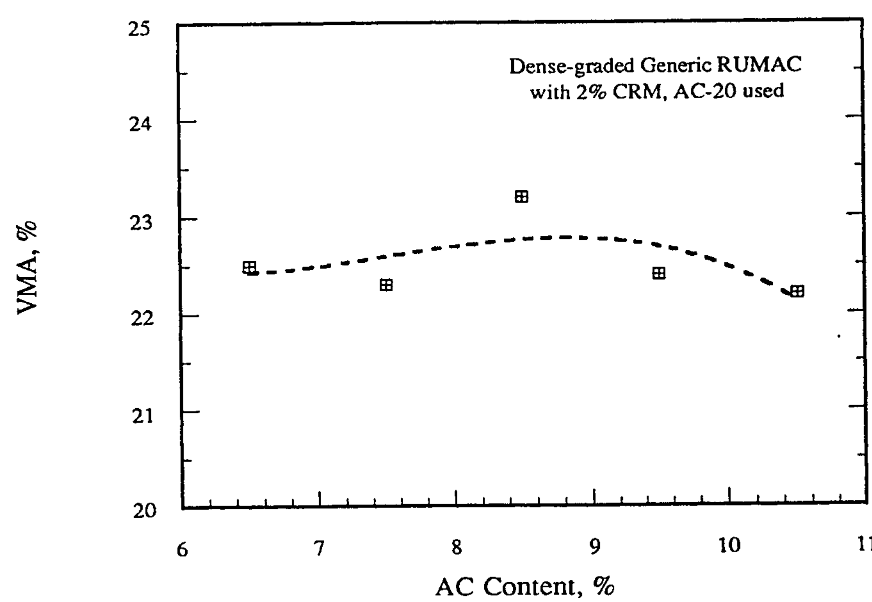


Fig. 4-41 VMA versus AC content for dense-graded HMA with 2%RUMAC

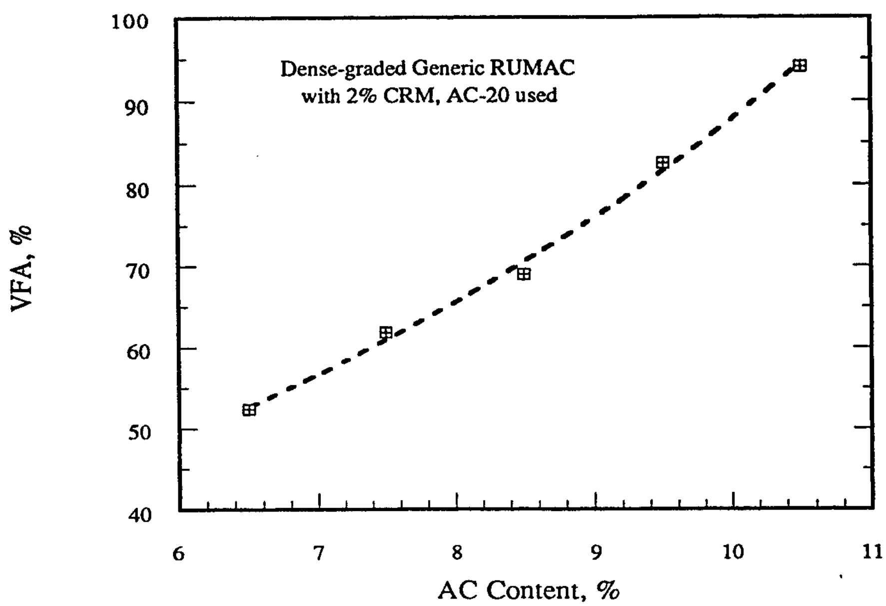


Fig. 4-42 VFA versus AC content for dense-graded HMA with 2%RUMAC

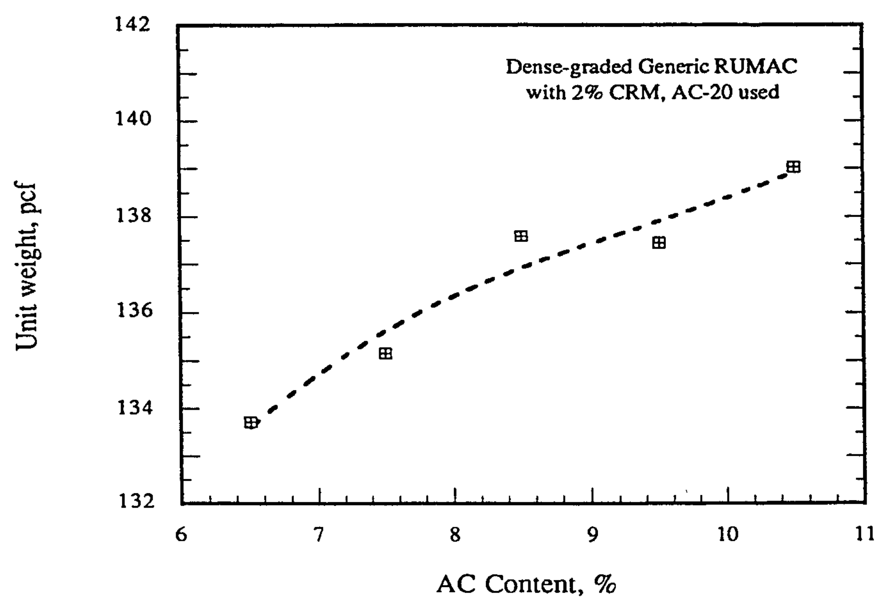


Fig. 4-43 Unit weight versus AC content for dense-graded HMA with 2%RUMAC

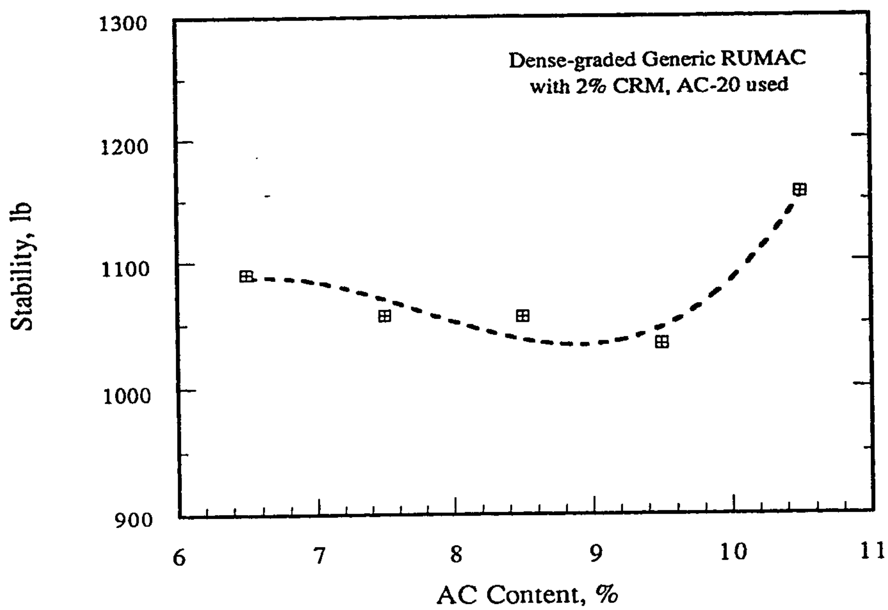


Fig. 4-44 Stability versus AC content for dense-graded HMA with 2%RUMAC

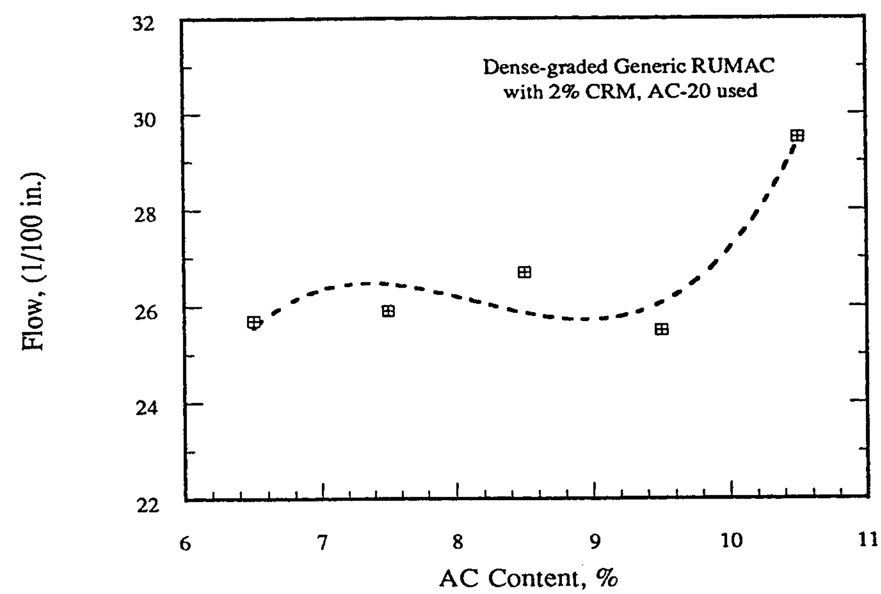


Fig. 4-45 Flow versus AC content for dense-graded HMA with 2%RUMAC

(ii) Gap-graded Generic RUMAC

Both 2 and 3 percent of CRM contents by the weight of aggregate were used for the gap-graded generic RUMAC. Fig. 4-46 to Fig. 4-51 show the Marshall test results for the gap-graded generic RUMAC with 2%CRM. Similarly, Fig. 4-52 to Fig.4-57 show the Marshall test results for the gap-graded generic RUMAC with 3%CRM. Optimum AC content was determined as 9.7 percent and 8.0 percent by weight of aggregates for the mix of 2% and 3% CRM, respectively, based on 4 percent air void criterion. The average of test results and the optimum binder content (by weight of aggregates) of each mix are presented in Table 4-19 and Table 4-21, respectively The pertinent mix properties of the mix containing optimum AC content are summarized in Table 4-20 and Table 4-22 for 2% and 3% CRM, respectively.

Table 4-19 Average of test data from Marshall design (generic dry process) (gap-graded gradation with 2 % CRM)

Binder Content	Air void %	VMA %	VFA %	Unit weight	Stab. lb	Flow 0.01 in
Gap-graded with 2% CRM: Optimum AC content (9.7%)						
6.5%	11.3	22.7	50.2	133.22	1048	30.4
7.5%	9.9	22.7	56.4	134.47	1013	28.9
8.5%	7.2	21.7	66.8	137.53	1231	32.0
9.5%	4.5	21.7	79.3	138.80	1176	28.7
10.5%	2.9	22.6	87.2	138.42	982	27.0

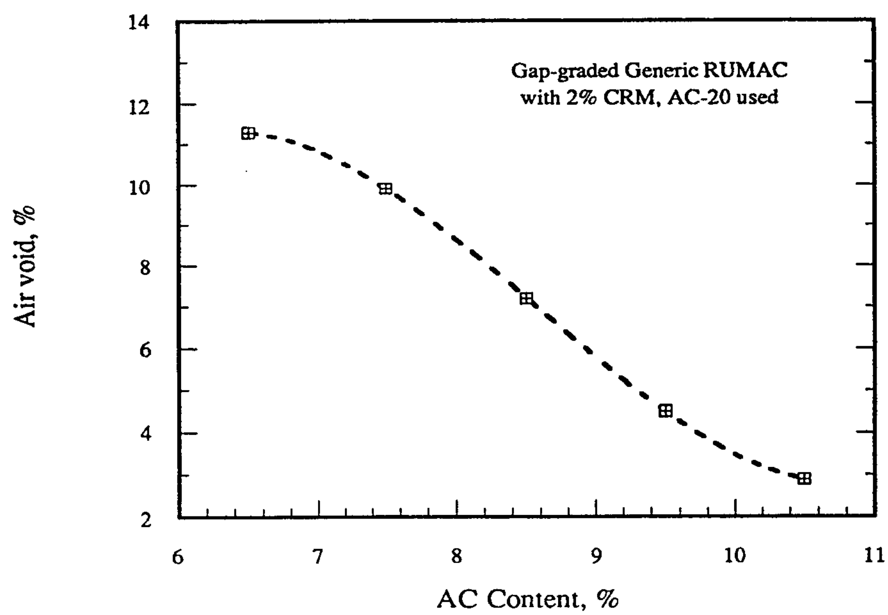


Fig. 4-46 Air void versus AC content for gap-graded HMA with 2%RUMAC

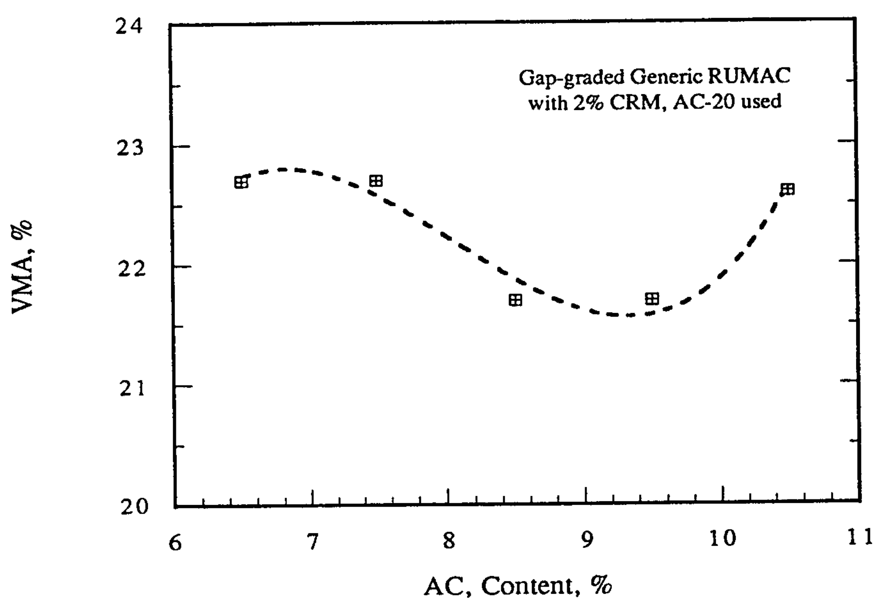


Fig. 4-47 VMA versus AC content for gap-graded HMA with 2%RUMAC

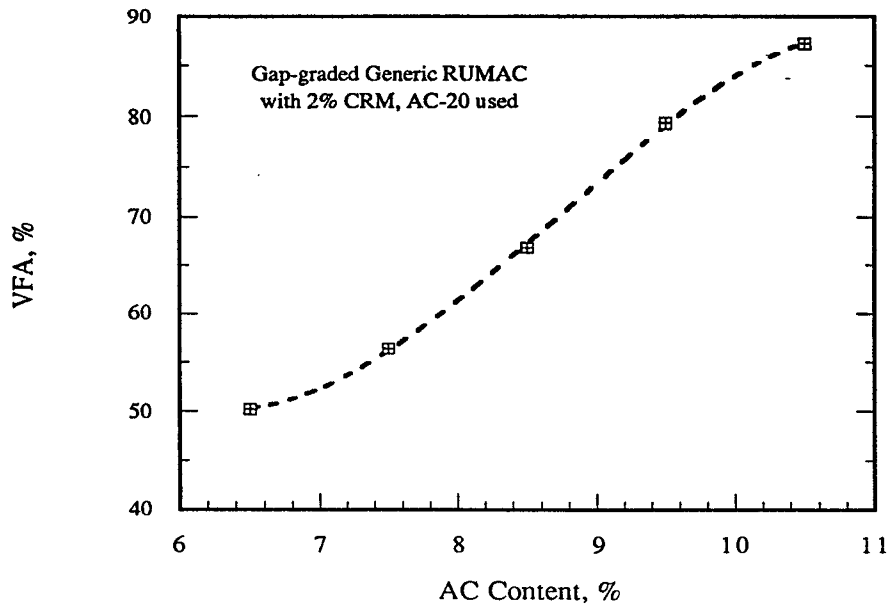


Fig. 4-48 VFA versus AC content for gap-graded HMA with 2%RUMAC

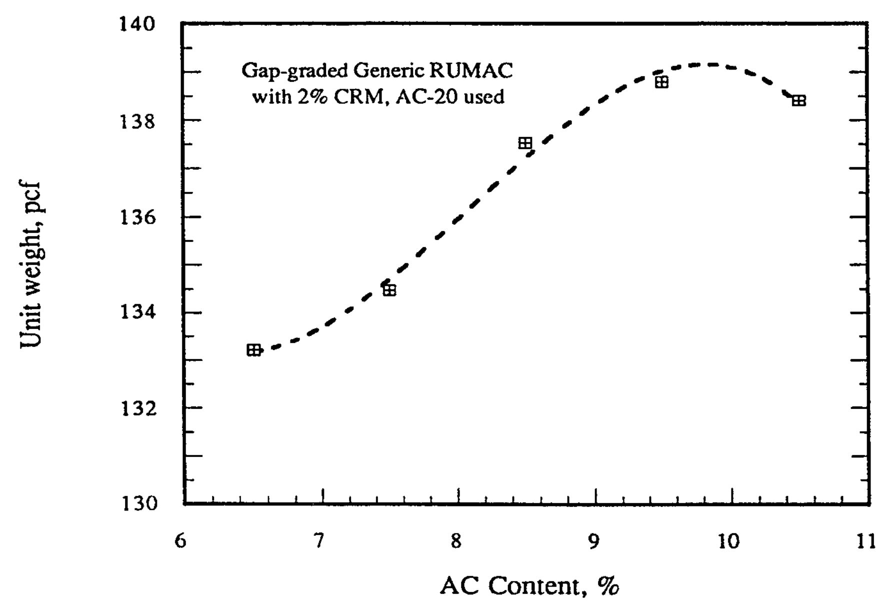


Fig. 4-49 Unit weight versus AC content for gap-graded HMA with 2%RUMAC

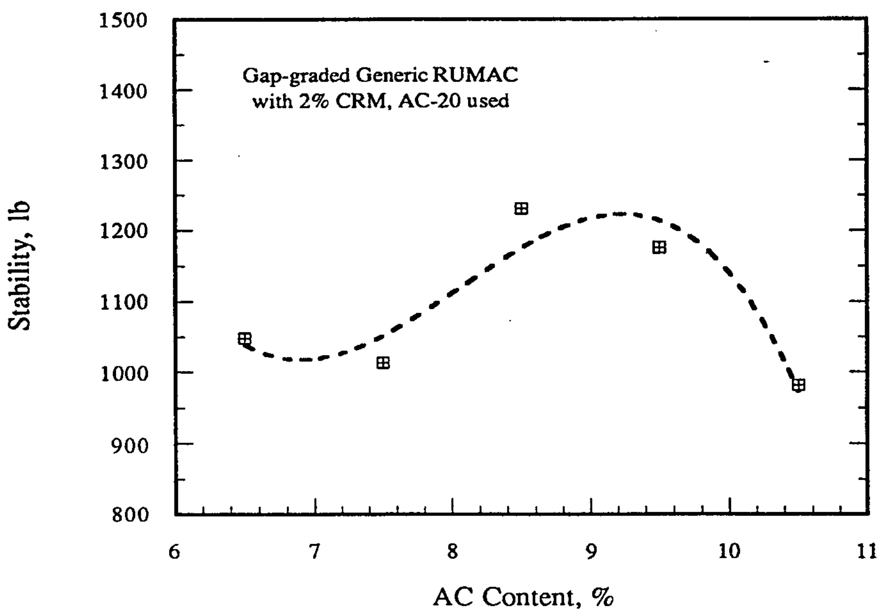


Fig. 4-50 Stability versus AC content for gap-graded HMA with 2%RUMAC

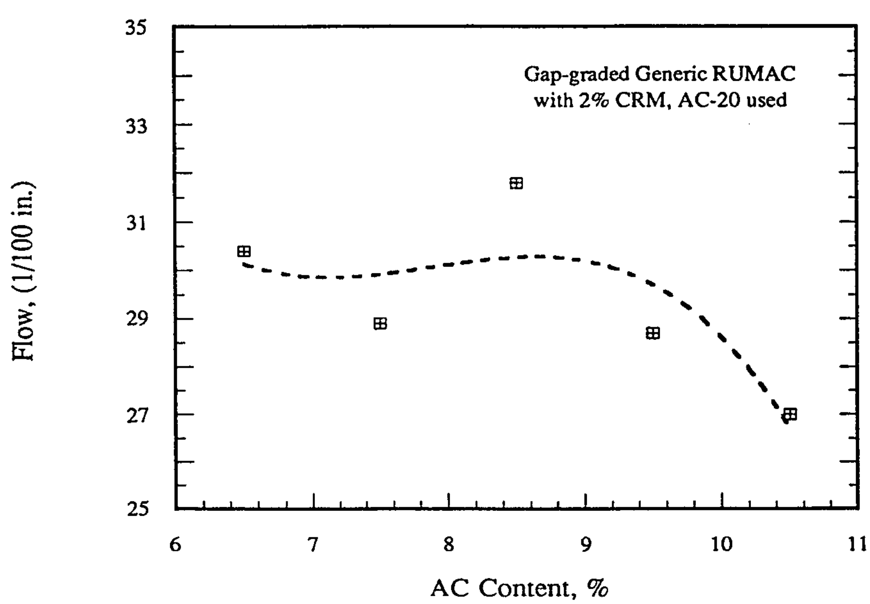


Fig. 4-51 Flow versus AC content for gap-graded HMA with 2%RUMAC

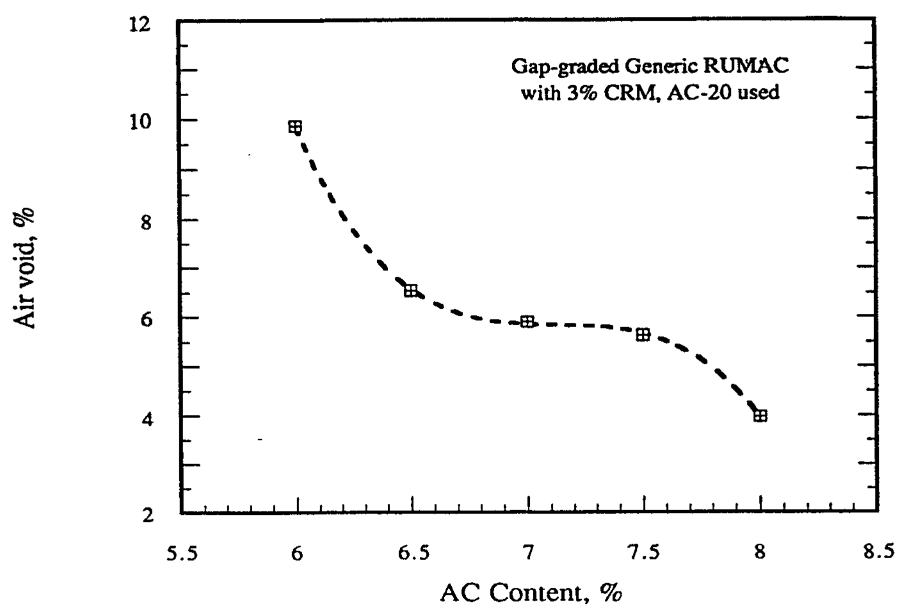


Fig. 4-52 Air void versus AC content for gap-graded HMA with 3%RUMAC

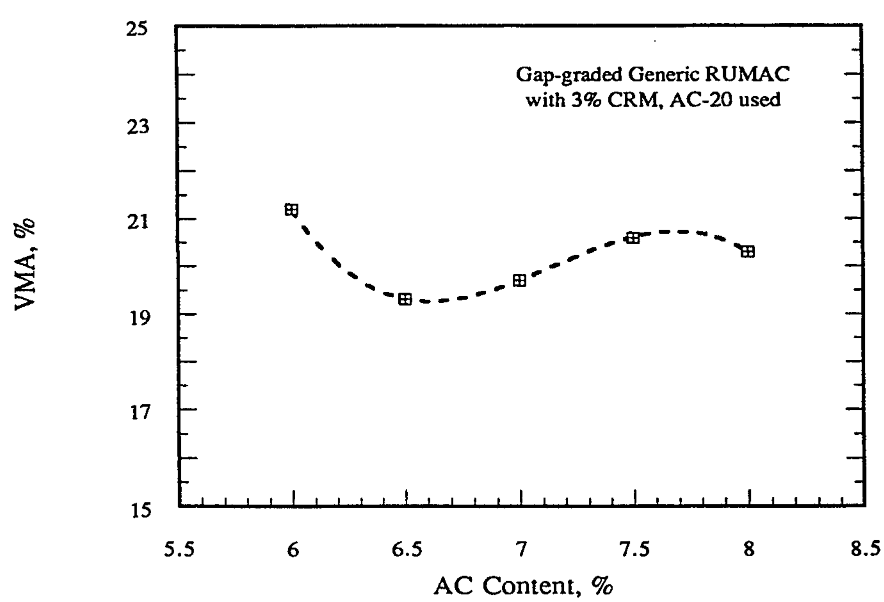


Fig. 4-53 VMA versus AC content for gap-graded HMA with 3%RUMAC

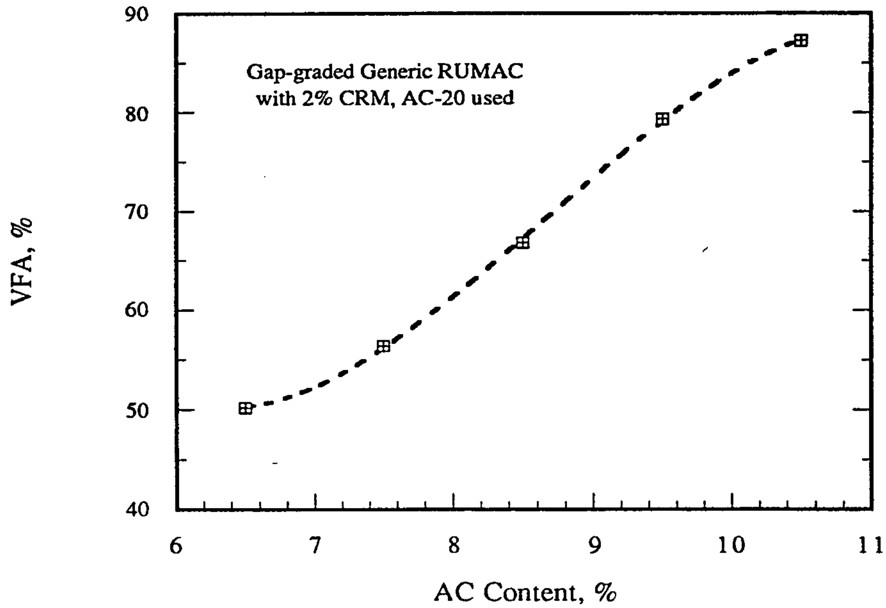


Fig. 4-54 VFA versus AC content for gap-graded HMA with 3%RUMAC

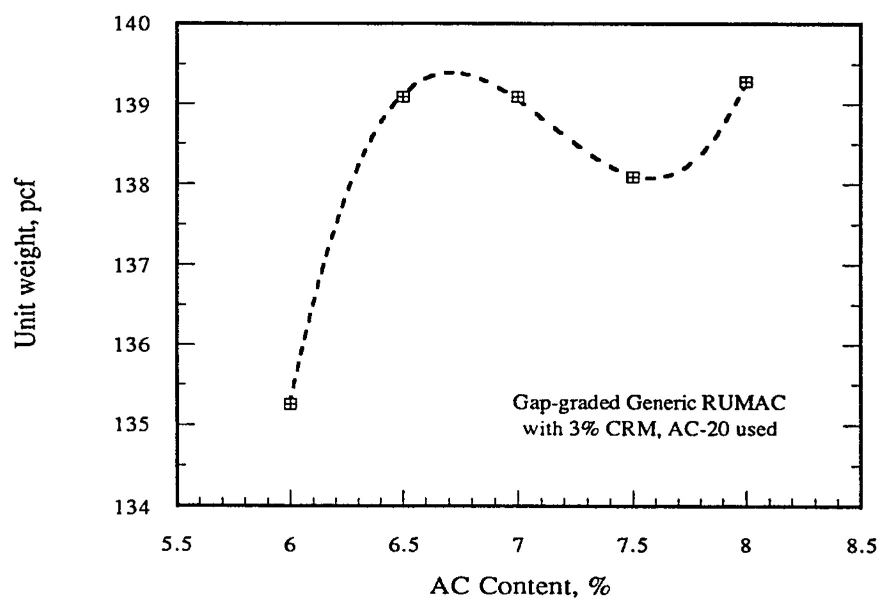


Fig. 4-55 Unit weight versus AC content for gap-graded HMA with 3%RUMAC

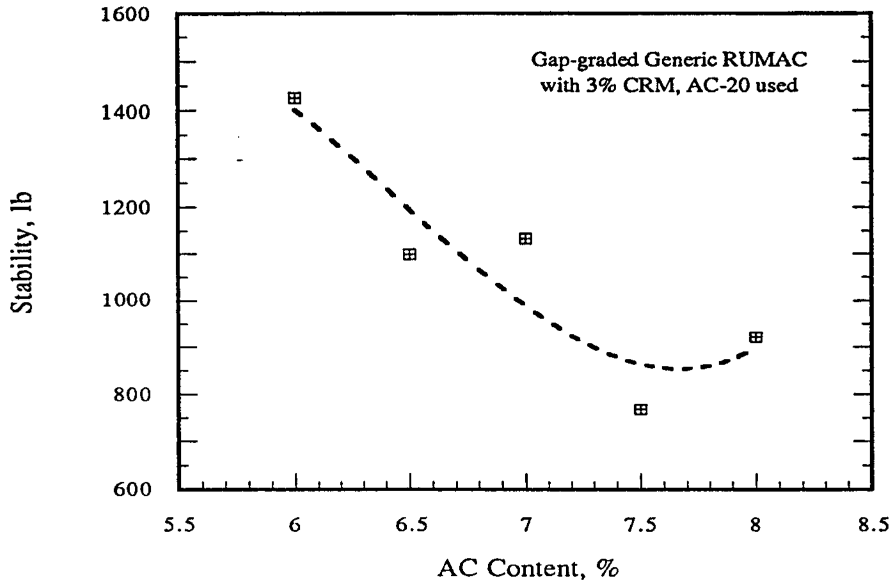


Fig. 4-56 Stability versus AC content for gap-graded HMA with 3%RUMAC

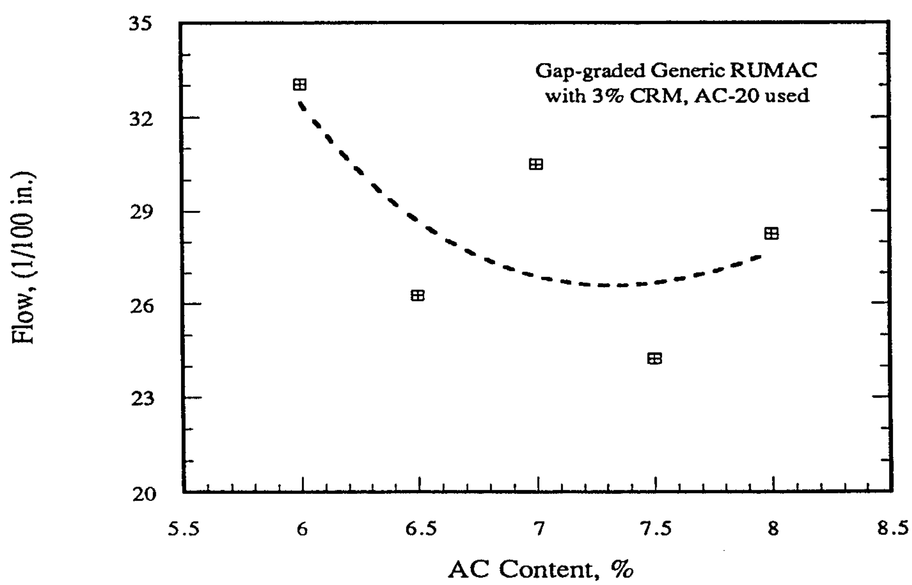


Fig. 4-57 Flow versus AC content for gap-graded HMA with 3%RUMAC

Table 4-20 Marshall mixture properties at optimum AC content (generic dry process) (gap-graded gradation with 2 % CRM)

AC Content, %	9.7
VMA, %	21.7
VFA, %	80.0
Unit Weight, pcf	139.0
Stability, lb	1190
Flow, 0.01 in.	29.4

Table 4-21 Average of test data from Marshall design (generic dry process) (gap-graded gradation with 3 % CRM)

Binder Content	Va %	VMA %	VFA %	Unit weight	Stab. lb	Flow 0.01 in.
Gap-graded with 3% CRM: Optimum AC content (8.0%)						
6.0%	9.87	21.2	53.6	135.3	1425	33.1
6.5%	6.54	19.3	66.2	139.1	1099	26.3
7.0%	5.90	19.7	70.1	139.1	1133	30.5
7.5%	5.63	20.6	72.7	138.1	768	24.3
8.0%	3.98	20.3	80.4	139.3	922	28.7

Table 4-22 Marshall mixture properties at optimum AC content (generic dry process) (gap-graded gradation with 3 % CRM)

AC Content, %	8.0
VMA, %	20.3
VFA, %	80.4
Unit Weight, pcf	139.3
Stability, lb	922
Flow, 0.01 in.	28.7

4.5 Result of open grade friction course mix design

Open grade friction course mix design with asphalt-rubber binder was performed in accordance with FHWA Technical Advisory T5040.31. The asphalt-rubber binder and aggregate used for this mix are identical to those used in the dense-graded wet mix.

Surface capacity of the predominant (plus No. 4 sieve) aggregate fraction was determined in accordance with oil soak test (AASHTO T 270). Percent oil retained (POR) was calculated as 3.67% using Eq. 4-1.

$$POR = \frac{SG_a}{2.65} \times \frac{(B-A)}{A} \times 100(\%) \qquad (4\text{-}1)$$

where,

SG_a = apparent specific gravity of the predominant aggregate

A = oven dry weight of the sample

B = coated weight of the sample

Surface constant value (K_c) and optimum AC content were determined as 1.567 and 6.92% according to the equations listed below.

$$K_c = 0.1 + 0.4\,(POR)$$

$$(AC)_{JMF} = (2.0\,K_c + 4.0)^* \; 2.65/SG_a$$

Optimum asphalt-rubber binder content, air void, and VMA were calculated as 8.2%, 9%, and 22.7%, respectively. Mix at temperature of 300^0 F was seen to perform well without showing excessive drainoff.

Calculations of IRS (index of retained strength) according to ASTM D 1075 provide the numerical index of resistance of bituminous mixtures to the detrimental effect of water. The IRS was calculated as the percentage of the original strength that is retained after the specific immersion period: 24 hr at 140^0 F plus 2 hr at 77^0 F. The IRS of the OGFC mix with optimum AC content was determined as 55.4 percent. The pertinent results of the compressive strength test between dry and immersed specimens are summarized in Table 4-23.

Table 4-23 Result of compressive strength test on OGFC mixtures

Specimen	Before immersion	After immersion
Max. Load, lb	2100, 2130, 2650	1130, 1170, 1630
Average, lb	2293	1310
Compressive Strength, psi	182.5	101.1
IRS, %	55.4	

4.6 Summary

The optimum binder contents of different processes and technologies as determined by the Marshall procedure and corresponding criteria are summarized in Table 4-24.

Table 4-24 Summary of the mix design for different processes and technologies

Mix Design Method	Optimum Binder content, % (by weight of mix)
DG/wet/AC20	5.7
DG/wet/Ecoflex	7.0
DG/wet/AR (AC5/10%GY)	5.6
DG/wet/AR (AC5/15%WRF 30)	7.2
GG/wet/AR (AC5/15%WRF 30)	8.4
DG/wet/AR (AC10/10%WRF 30)	8.3
DG/dry/2% CRM	9.5*
GG/dry/2% CRM	9.7*
GG/dry/3% CRM	8.0*
OG/AC5/15% WRF 30	8.2

DG: dense-graded
GG: gap-graded
OG: open-graded
*: By weight of aggregates

CHAPTER V

MECHANICAL PROPERTIES OF RUBBER MODIFIED ASPHALT CONCRETE MIXTURE

There are numerous laboratory tests available for evaluating the performance characteristics of the asphalt-aggregate mix. In this study, seven types of tests were used: (i) Indirect Tensile Strength, (ii) Resilient Modulus Test, (iii) low temperature thermal cracking resistance using TSRST, (iv) Incremental Creep Test, (v) loaded wheel track test, (vi) water sensitivity test, and (vii) fatigue test. Most the tests were conducted using the ASTM testing standards, if applicable. The TSRST and the incremental creep test, however, were conducted in accordance with the AASHTO provisional testing procedures.

5.1 Indirect Tensile Strength Test

The indirect tensile strength for various asphalt mixtures was determined following the SHRP Protocol 07. A four inch diameter Marshall specimen was loaded in compression along the diametral axis at a fixed deformation rate (2 inch/min.) until failure occurs.

Calculation of the indirect tensile strength was based on the following equation:

$$S_t = \frac{50.127 \times P_0}{t} \times \left[\sin[\frac{1455.313}{D}] - [\frac{12.7}{D}] \right] \tag{5-1}$$

where

S_t = indirect tensile strength, kPa

P_0 = maximum load sustained by the specimen, N

t = specimen thickness, mm

D = specimen diameter, mm

5.1.1 Specimen Preparation

Marshall specimens were used for indirect tensile strength test. To simulate the aging process of the asphalt-rubber-aggregate mixtures, the method described by Von Quintus et al. (1988) was followed. The short-term aging was accomplished by subjecting the loose mixture of asphalt and aggregate to the elevated temperature of 275°F (135°C) for 8 hours before compaction. After the short-term aging process, the compacted specimen is stored in the forced draft oven at the temperature of 140°F (60°C) for two days and then at the temperature of 225°F (107°C) for additional five days.

5.1.2 Results of Indirect Tensile Tests

A typical load versus time curve of the indirect tensile test is shown in Fig. 5-1. Table 5-1 presents the results (including the aging effect) of the indirect tensile tests of various mixtures. Table 5-2 gives a summary of the indirect tensile strength measured at room temperature of the different mixes under short-term amd long-term aging processes.

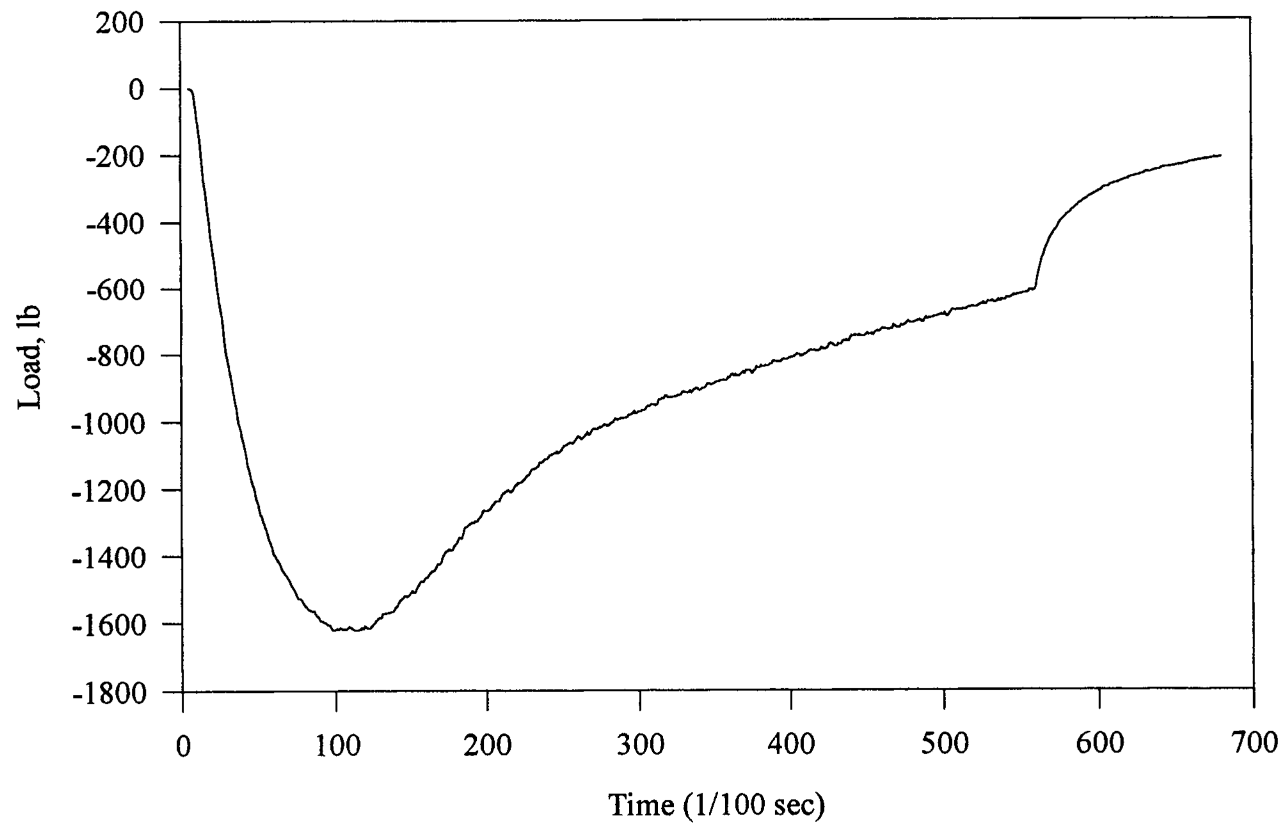
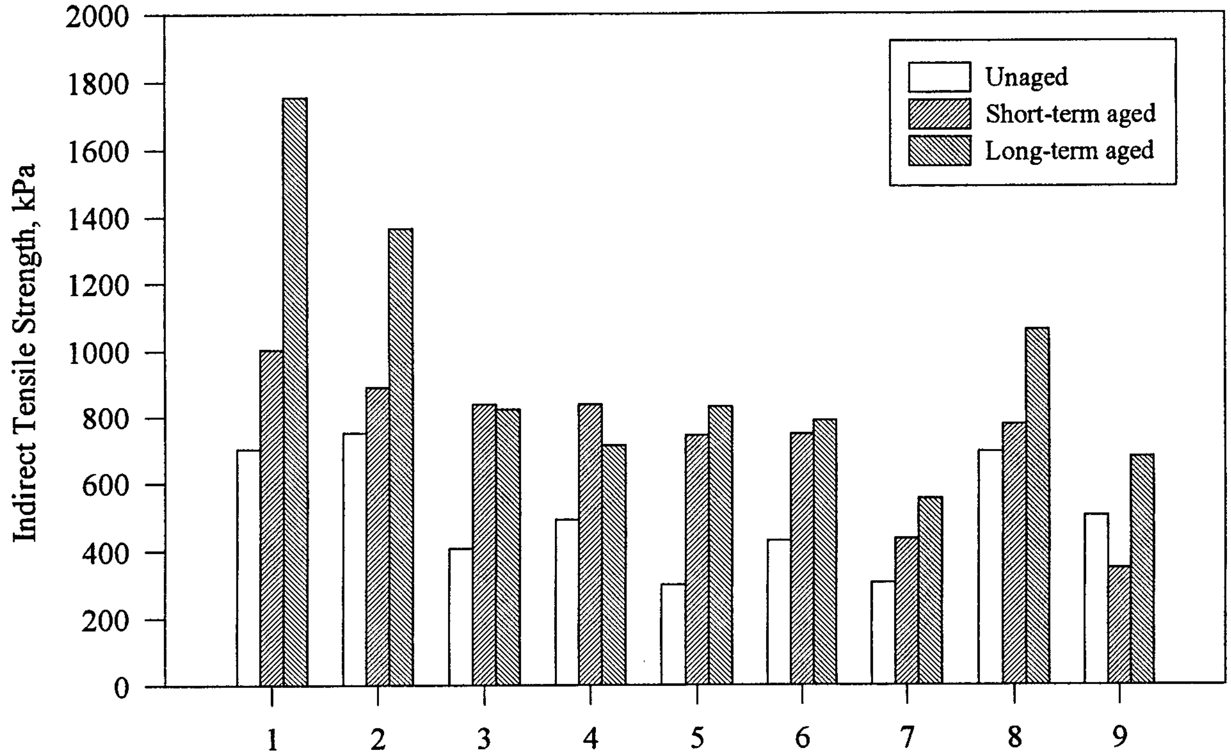


Fig 5-1 A typical load versus time curve in indirect tensile test

Fig. 5-2 were plotted to show direct comparison between various mixes and the effect of aging on the indirect tensile strength. The general observation of the indirect tensile test results is that the mixture prepared with CRM modified binder tend to exhibit lower tensile strength compared to the control mix. Both the control mix and the CRM modified mixes tend to increase the indirect tensile strength due to the aging effect, except for the continuously blended mix which shows a decreased indirect tensile strength due to the short-term aging effect. There is no significant discernable difference in the aging behavior between the control mix and the CRM modified mixes.

Table 5-1 Results of indirect tensile test for various mixtures

Specimen Designation		Max. Load (N)	Displacement at Max. Load (mm)	Indirect Tensile Strength (kPa)
Control Mix AC20	Unaged	7201	1.4989	701.3
	Short-term aged	10306	1.4605	1003.7
	Long-term aged	18014	0.955	1754.3
Ecoflex Dense-graded	Unaged	7820	1.5342	750.1
	Short-term aged	9238	1.1481	890.0
	Long-term aged	14025	0.95	1366.1
AC5+15% WRF30, Wet process, Dense-graded	Unaged	4177	1.971	406.9
	Short-term aged	8594	1.1085	837.2
	Long-term aged	8545	2.0904	823.3
AC5+15% WRF30 Wet process Gap-graded	Unaged	5137	2.1615	493.1
	Short-term aged	8642	1.3843	837.9
	Long-term aged	7588	1.7805	713.0
AC5+15% WRF30 Wet Process Open-graded	Unaged	2237	3.5484	211.7
	Short-term aged	NA	NA	NA
	Long-term aged	NA	NA	NA
AC20 Dry process Dense-graded with 2%CRM	Unaged	3065	4.7676	298.6
	Short-term aged	7624	1.717	742.7
	Long-term aged	8531	2.86	831.0
AC20 Dry process Gap-graded with 2%CRM	Unaged	4412	3.8785	429.6
	Short-term aged	7673	2.0396	747.5
	Long-term aged	8100	2.7556	788.9
AC20 Dry process Gap-graded with 3%CRM	Unaged	3332	4.9555	305.5
	Short-term aged	4782	3.302	435.1
	long-term aged	5956	3.4239	554.4
AC10+10% WRF30 Wet process Dense-graded	Unaged	6994	0.81	693
	Short-term aged	7710	NA	777
	long-term aged	10549	NA	1063
AC5+10%GY Continuous blending Dense-graded	Unaged	5134	NA	504
	Short-term aged	3548	NA	349
	long-term aged	6898	NA	678



1--Control mix AC20
2--Ecoflex, Dense-graded
3--AC5+15%WRF30, Wet process, Dense-graded
4--AC5+15%WRF30, Wet process, Gap-graded
5--AC20, Dry process, Dense-graded with 2%CRM
6--AC20, Dry process, Gap-graded with 2%CRM
7--AC20, Dry process, Gap-graded with 3%CRM
8--AC10+10%WRF30, Wet process, Dense-graded
9--AC5+10%GY, Continuous blending, Dense-graded

Fig. 5-2 The effect of aging on the indirect tensile strength of various mixes

Table 5-2 Summary of the Aging Effect on the Indirect Tensile Test Results

Mixtures	Indirect Tensile Strength (kPa)			Ratio	
	Unaged	Short-Term Aged	Long-Term Aged	Short-Term over Unaged	Long-Term over Unaged
Control Mix, AC20	701.3	1003.7	1754.3	1.43	2.50
Ecoflex, Dense-graded	750.1	890.0	1366.1	1.19	1.82
AC5+15%WRF30 Wet process, Dense-graded	406.9	837.2	823.3	2.06	2.02
AC5+15%WRF30 Wet process, Gap-graded	493.1	837.9	713.0	1.70	1.45
AC5+15%WRF30 Wet process, Open-graded	211.7	NA	NA	NA	NA
AC20, Dry process, Dense-graded with 2%CRM	298.6	742.7	831.0	2.49	2.78
AC20, Dry process, Gap-graded with 2%CRM	429.6	747.5	788.9	1.74	1.84
AC20, Dry process, Gap-graded with 3%CRM	305.5	435.1	554.4	1.42	1.81
Dense-graded, Wet process (AC10+10%WRF30)	693	777	1063	1.12	1.53
Dense Grade, Continuous Blending, (AC5+10% GY)	504	349	678	0.70	NA

5.2 Resilient Modulus Test

5.2.1 Introduction

It is well known that most asphalt paving materials are not purely elastic materials, but more like visco-elastic or visco-plastic materials. That means permanent deformation develops with each load application. As the number of load repetition increases, the incremental plastic strain due to each load repetition decreases (of course, the accumulated plastic strain increases). After 100 to 200 repetitions, the strain is practically all recoverable. The elastic modulus based on the recoverable strain under repeated load is called the resilient modulus M_R, which is defined as

$$M_R = \frac{\sigma_d}{\varepsilon_r} \tag{5-2}$$

in which σ_d is the applied deviatoric stress, which is the axial stress in an unconfined compression test, and ε_r is the recoverable strain.

5.2.2 Previous Findings

Researchers (Hoyt et al., 1987) from Texas A&M University conducted the resilient modulus tests on asphalt-rubber concrete. The aggregate used in their study was a blend of crushed limestone and field sand to meet Federal Aviation Administration (FAA) aggregate grading specification for pavement with a bituminous surface course that will accommodate aircraft with gross weight of 60,000 lb or more and with tire pressure of 100 psi or more. The maximum particle size of the aggregate was ½ in. (100% passing the ½ in. sieve and some retained on the 3/8 in. sieve).

In their resilient modulus tests, three temperatures were used: 33^0 F, 77^0 F, and 104^0 F. The specimens tested include the AC-10 based control mix as well as asphalt-rubber concrete with low, medium, and high binder content. From their resilient modulus test results, the resilient modulus of the control mix exhibits a higher value (about 30 percent more) than that of the asphalt-rubber concrete mix at 33^0 F. The resilient modulus test results conducted at other two temperatures showed about the same resilient modulus values for the control mix and asphalt-rubber concrete.

Al-Abdul-Wahhab and Al-Amri (1991) conducted resilient modulus tests on specimens that were prepared from Marshall compaction procedure. The aggregate used in the mix design was limestone, which was used extensively in road projects in Saudi Arabia. Two aggregate gradations, G1 and G2, as per Saudi Arabian Ministry of Communications specification, were selected for preparation of the required laboratory specimens. The CRM used in their study was obtained from Arabian International Tire Retreaders. The CRM was in the form of fine ground shreds produced by the mechanical grinding of truck tire treads at ambient temperatures. CRM used in the wet process consisted of the material passing sieve No. 20 and retained on sieve No. 200. The G2 aggregates were used in making three percent RUMAC mix.

The specimens were placed in the dynamic diametral test apparatus with a seating load of 4.5 kg . A dynamic load of 68 kg was applied and after 100 load repetitions, the load applied and the horizontal elastic deformation were used to compute the resilient modulus value. Specimens were tested at two temperatures, 25°C and 40°C. Results of their tests are summarized in Table 5-3

Table 5-3 Resilient modulus test results, ksi (Al-Abdul-Wahhab and Al-Armi, 1991)

Mix type	CRM content %	M_R at 25° C ksi	M_R at 40° C ksi
G1	0	964	283.33
	10	1103	360.33
	20	710.33	273
	30	575	255
G2	0	1267.33	510
	10	1699.67	630.33
	20	1096	370
	30	805.67	283.33
3% RUMAC	0	449.67	145.33

As can be seen in Table 5-3, with increasing test temperatures, mixes tended to soften and lose strength accompanied with reduction in resilient modulus ranging from 60% to 75% for the specimens tested. Mixes with 3% rubber aggregate (RUMAC) seemed to be more sensitive to temperature increases in terms of stiffness lose. On the other hand, mixes with 10% CRM showed an improved modulus compared to the control mixes, which seems to be opposite to that observed for 20% and 30% CRM mixes.

5.2.3 SHRP Resilient Modulus Test Protocol

The SHRP test protocol provides the test procedures for the determination of the resilient modulus (M_R) of hot mix asphalt concrete using repeated load indirect tensile test techniques. The SHRP test procedure was partially based on the test standard ASTM D 4123, indirect tension test for resilient modulus of bituminous mixtures. A schematic diagram of typical resilient modulus test apparatus with two horizontal LVDTs is shown in

Fig. 5-3. The vertical deformation is measured using the LVDT located inside the actuator of the Instron testing machine.

Two separate resilient moduli are suggested in the SHRP test procedure. One, termed instantaneous resilient modulus, is calculated using the recoverable horizontal deformation measured during the unloading portion of one load-unload cycle. The other one, termed total resilient modulus, is calculated using the total recoverable horizontal deformation which includes both instantaneous recoverable and time-dependent continuing recoverable deformation during the unload or rest period portion of one cycle.

In normal resilient modulus test, a repeated compressive stress of a fixed magnitude with a duration of 0.1 second and a rest period of 0.9 second is recommended. The indirect tensile strength determined from the tests were used for selecting the load level for the resilient modulus testing. Load levels corresponding to tensile stress levels of 30, 15, and 5 percent of the tensile strength measured at 25^0 C, were used in conducting the resilient modulus tests at the temperatures of 5, 25, and 40^0 C, respectively. At each temperature, ten percent of the established load level was used as the contact load to maintain a positive contact between loading strips and the specimen during the resilient modulus testing. Fig. 5-4 shows a typical repeated load versus time plot for the resilient modulus test. As can be seen, the specimen is subjected to a repeated compressive stress (90 percent of the total stress applied) and a constant stress (10 percent of the total stress applied). The instantaneous and total resilient (recoverable) vertical and horizontal deformation responses of the specimen are measured and used to calculate both instantaneous and total resilient moduli (M_{Ri} and M_{Rt} respectively). Eqs. 5-3 and 5-4 are

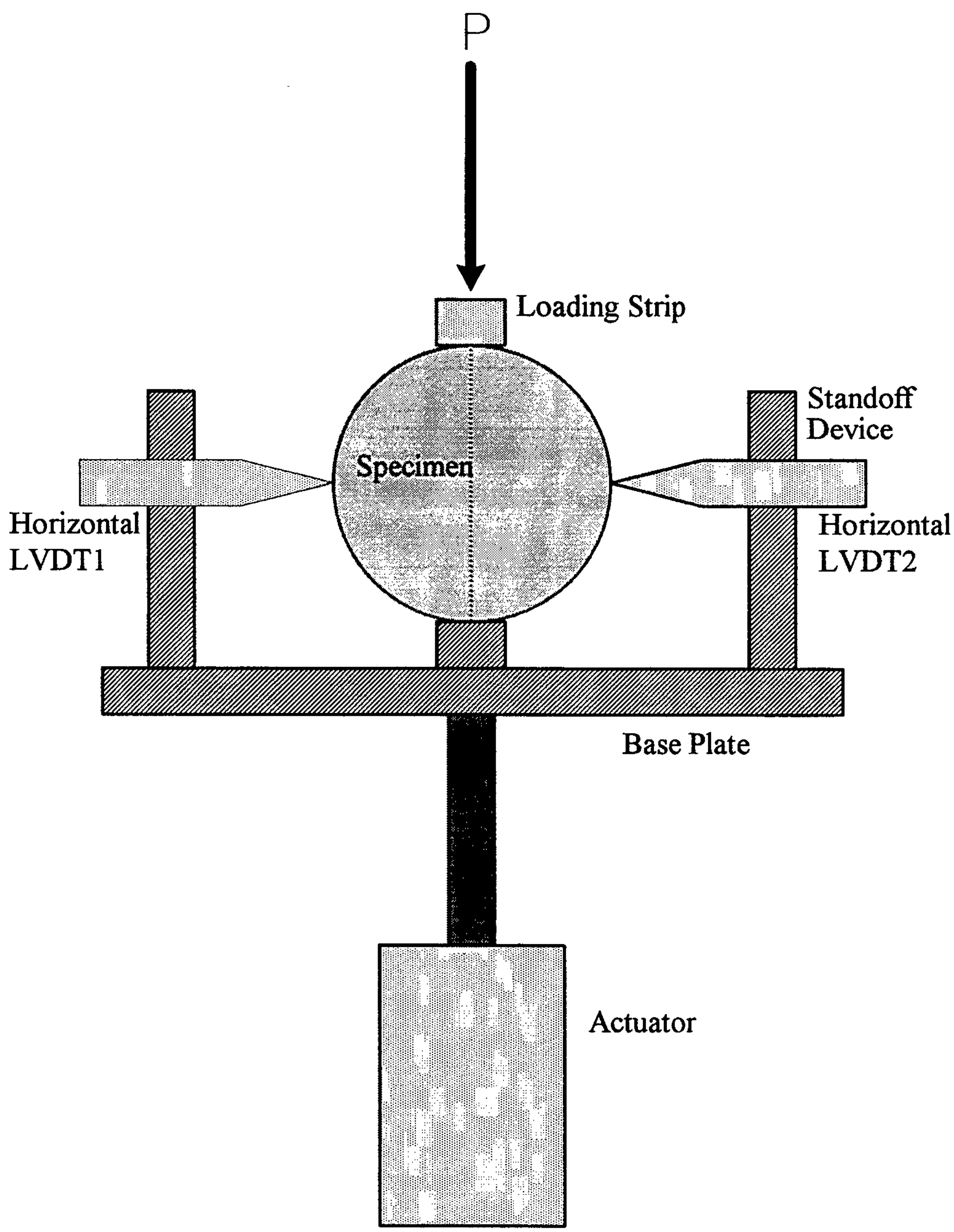


Fig. 5-3 Positioning of horizontal LVDTs and illustration of correct specimen alignment

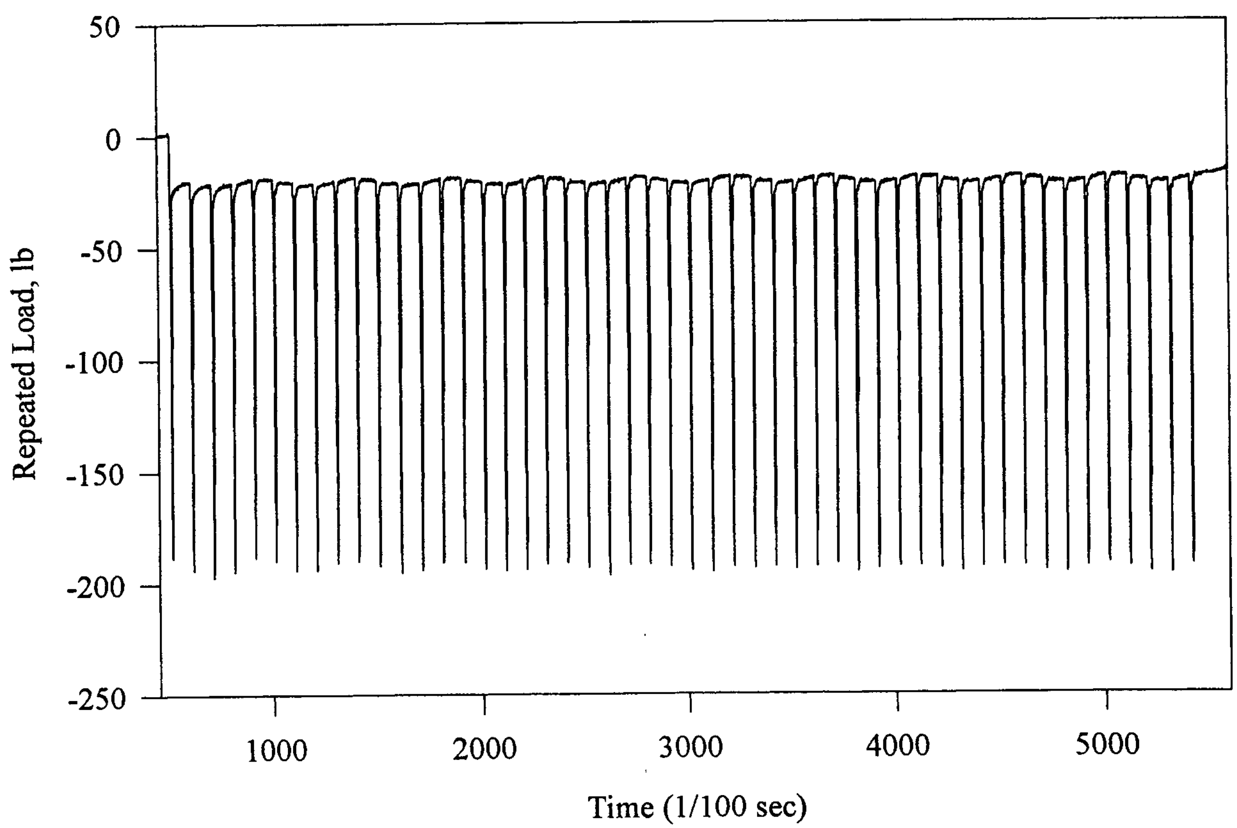


Fig. 5-4 Repeated load versus time relationship used in resilient modulus test

used in the calculation.

$$M_{Ri} = P \times \frac{\mu_{Ri} + 0.25}{t \times \Delta H_i}, \qquad M_{Rt} = P \times \frac{\mu_{Rt} + 0.25}{t \times \Delta H_t} \tag{5-3}$$

where;

M_{Ri} = instantaneous resilient modulus of elasticity, MPa

M_{Rt} = total resilient modulus of elasticity, MPa

P = repeated load, N

μ_{Ri} = instantaneous resilient Poisson's ratio

μ_{Rt} = total resilient Poisson's ratio

$$\mu_{Ri} = 3.59 \times \frac{\Delta H_i}{\Delta V_i} - 0.27, \qquad \mu_{Rt} = 3.59 \times \frac{\Delta H_t}{\Delta V_t} - 0.27 \tag{5-4}$$

where;

ΔH_i = instantaneous recoverable horizontal deformation, mm

ΔV_i = instantaneous recoverable vertical deformation, mm

ΔH_t = total recoverable horizontal deformation, mm

ΔV_t = total recoverable vertical deformation, mm

t = thickness of specimen, mm

Measurement of Deformations

In calculating resilient modulus, two types of deformation measurement are needed. One is instantaneous deformation and the other one is total deformation. As illustrated in Fig. 5-5, instantaneous deformations are determined from the intersection

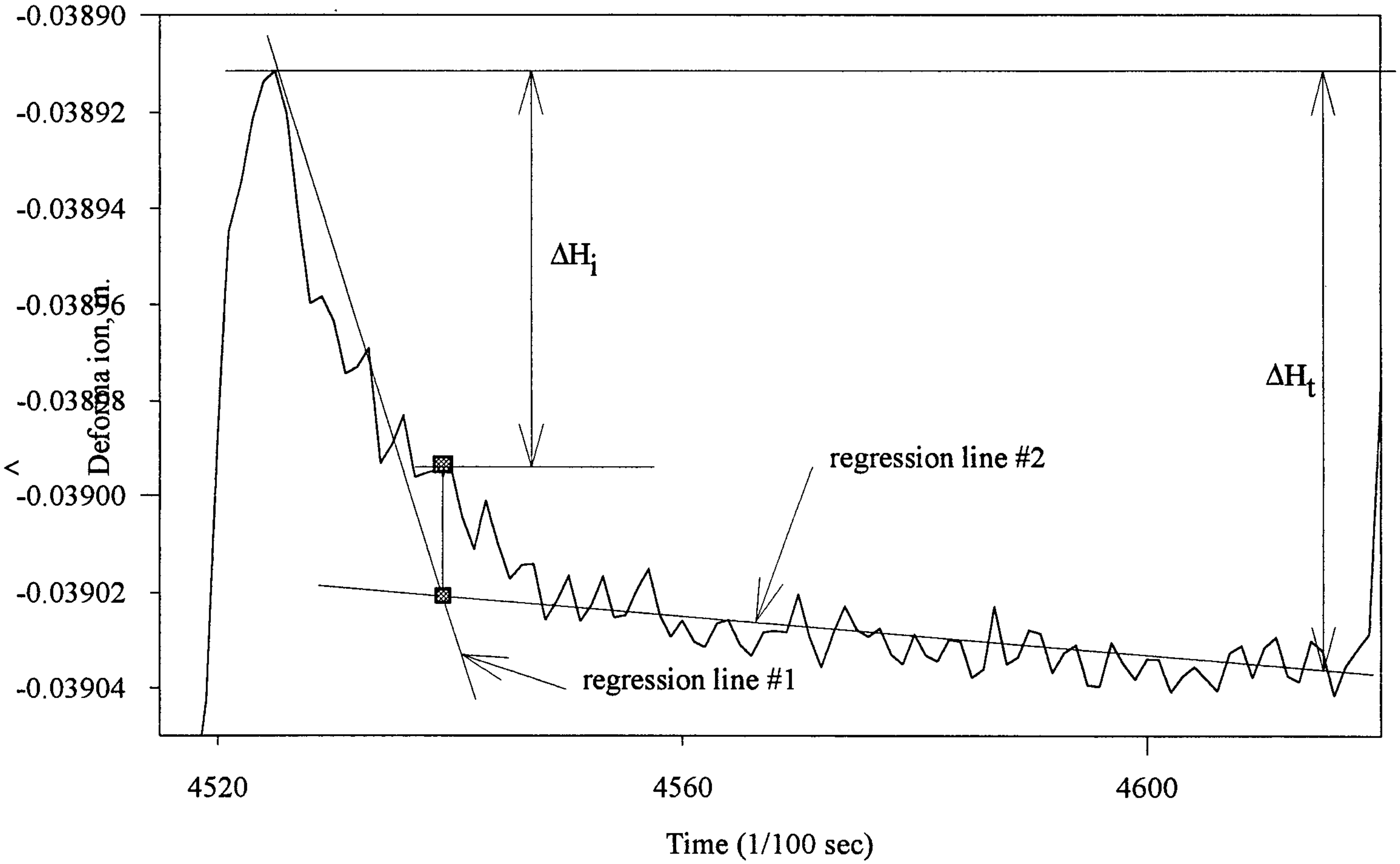


Fig. 5-5 Theoretical determination of instantaneous and total deformation

(intercept) of two regression lines and a vertical projection from the intercept to the actual unloading portion of the deformation versus time plot. The first regression line is essentially an extension of the linear portion of the unloading curve (regression line #1), and is based on all data points after the maximum or peak deformation occurs and before the specimen has rebounded by 75 percent of the total deformation. The second regression line is based on all data in the last 0.75 seconds of the one second loading and rest cycle (regression line #2). From the point of interception, a vertical line is generated which extends upwards to the actual unloading portion of the deformation curve. Total deformations are obtained at the end of one load-unload rest period cycle, as determined as the average deformation value from the last 75 percent of the cycle. Figs. 5-6, 5-7, and 5-8 show the typical trace of deformations from vertical and two horizontal LVDTs, respectively, during resilient modulus test. Fig. 5-9 gives a magnified picture of the deformation from one LVDT.

5.2.4 Specimen Preparation

Specimen preparation procedure and aging process are the same as those for the indirect tensile test.

5.2.5 Results of Resilient Modulus Test

The results of resilient modulus test at three temperatures on the various unaged mixtures are collected in Table 5-4.

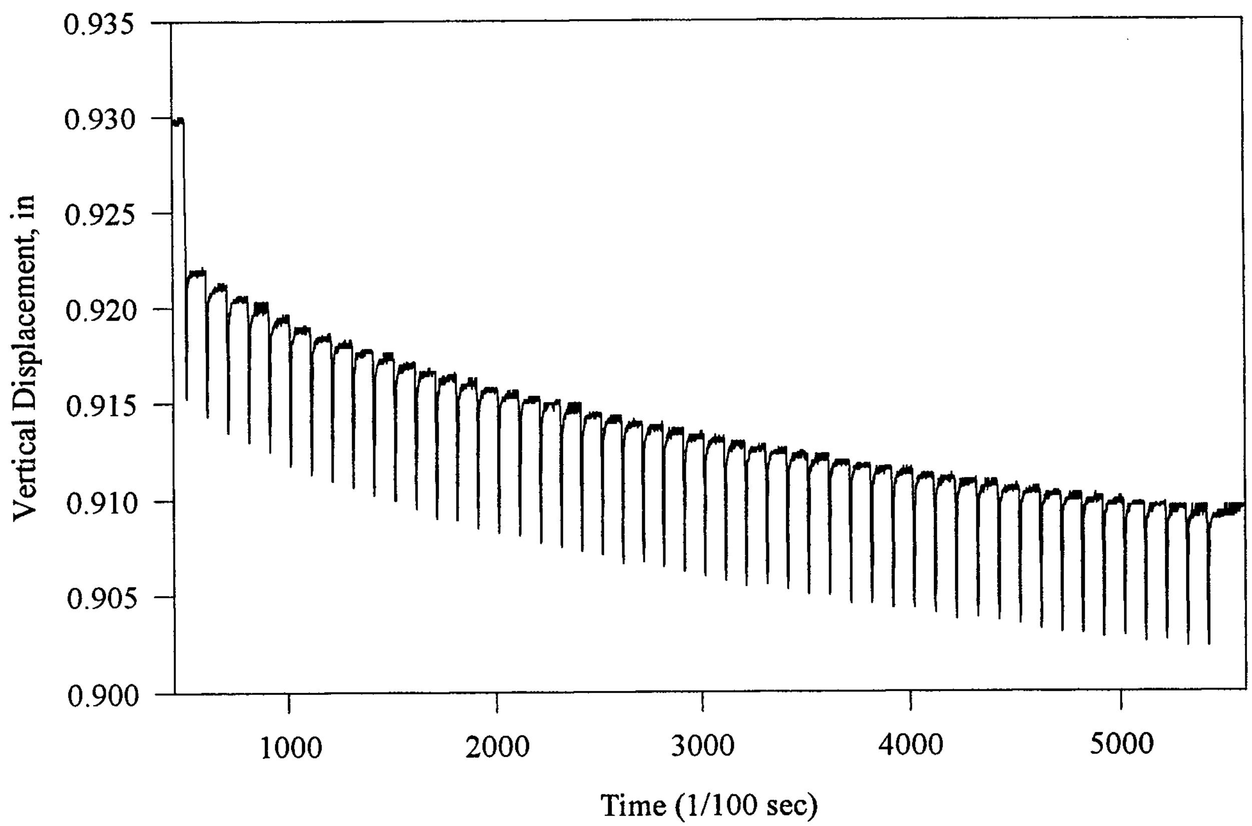


Fig. 5-6 Vertical displacement versus time relationship of resilient modulus test

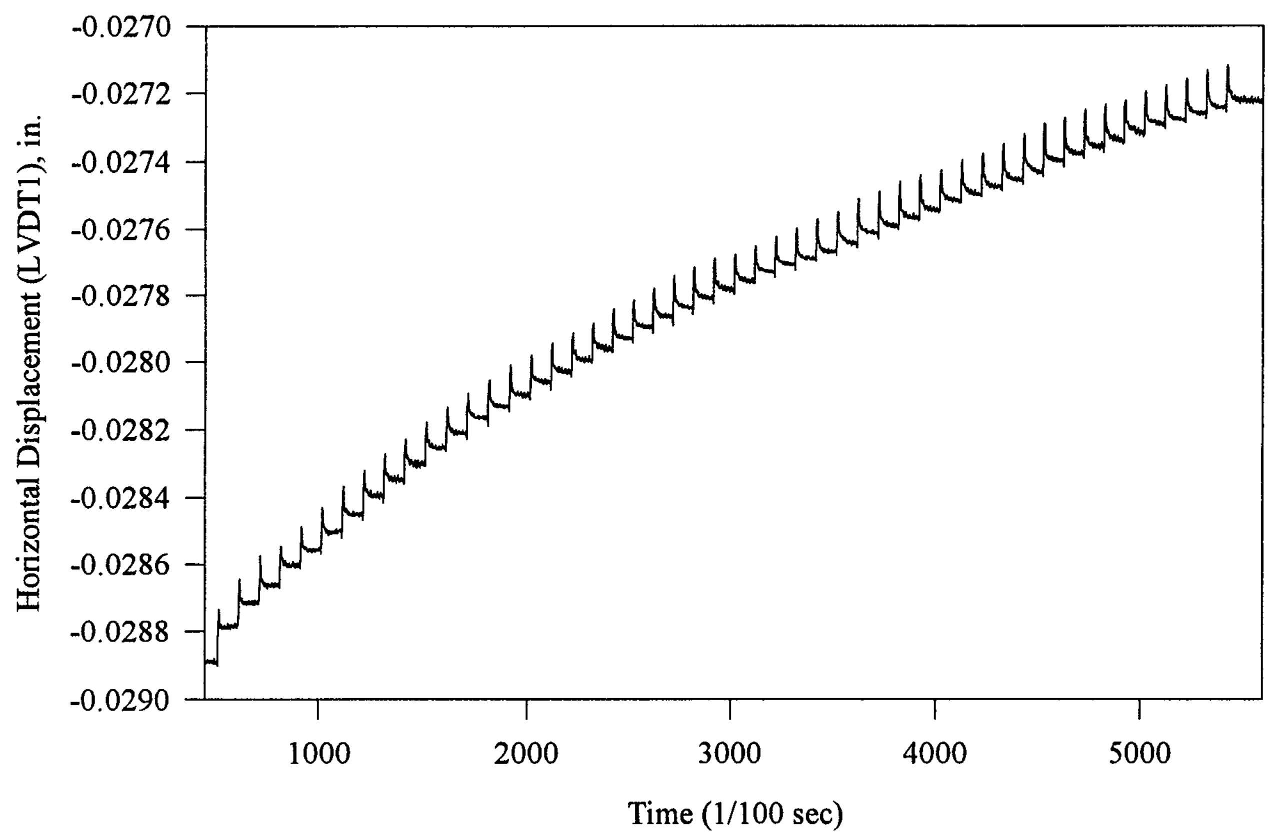


Fig. 5-7 Horizontal displacement (LVDT1) versus time relationship in resilient modulus test

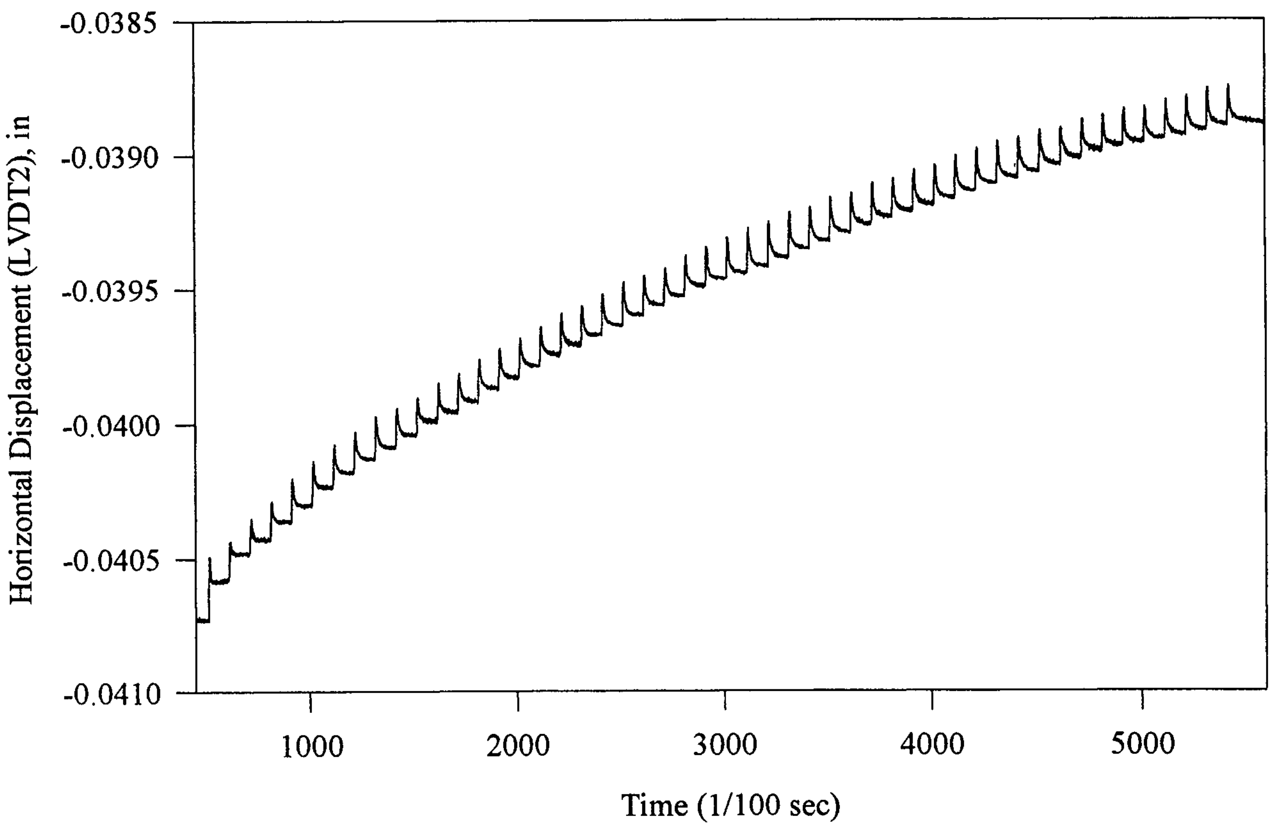


Fig. 5-8 Horizontal displacement (LVDT2) versus time relationship in resilient modulus test

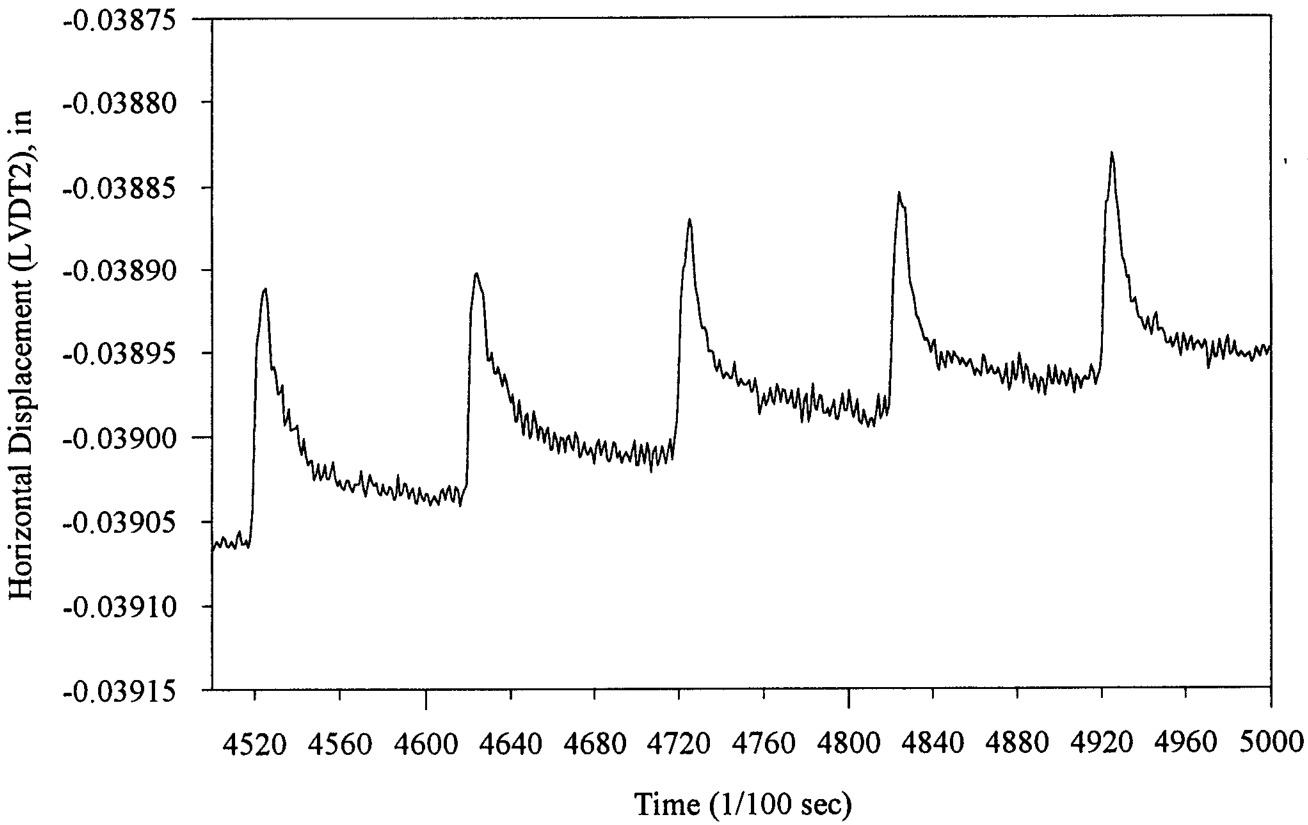


Fig. 5-9 Details of horizontal displacement (LVDT2) versus time relationship in resilient modulus test

Table 5-4 Results of resilient modulus test for various mixtures at three different temperatures

Mixes	Resilient Modulus (MPa)					
	5°C		25°C		40°C	
	M_{RI}	M_{RT}	M_{RI}	M_{RT}	M_{RI}	M_{RT}
Control Mix, AC20	NA	9559	NA	4979	NA	1531
AC5+15%WRF30 Wet process, Dense-graded	NA	7069	NA	1421	NA	1138
AC20, Dry process, Dense-graded with 2%CRM	1060	932	255	232	192	171
AC20, Dry process, Gap-graded with 2%CRM	791	754	191	181	192	173
AC5+15%WRF30 Wet process, Gap-graded	NA	3372	NA	2800	NA	fail
AC5+10%GY Continuous blending Dense-graded	NA	5903	NA	3655	NA	1290
AC10+10%WRF30 Wet process Dense-graded	4762	3735	2670	2007	1211	780

Fig. 5-10 shows the variations of the total resilient moduli for the unaged mixtures at the three temperatures. It can be seen that the mixes prepared with wet process and the continuous blending process show higher resilient modulus than the dry mixes. The AC10 based asphalt mixture seems to possess less temperature susceptibility than the others.

The effect of aging on the resilient moduli for the various mixes are shown in the Table 5-5(a) and Table 5-5(b). Fig. 5-11 to Fig 5-14 were plotted to give a graphic description of the effect of aging (only four mixtures were presented due to the

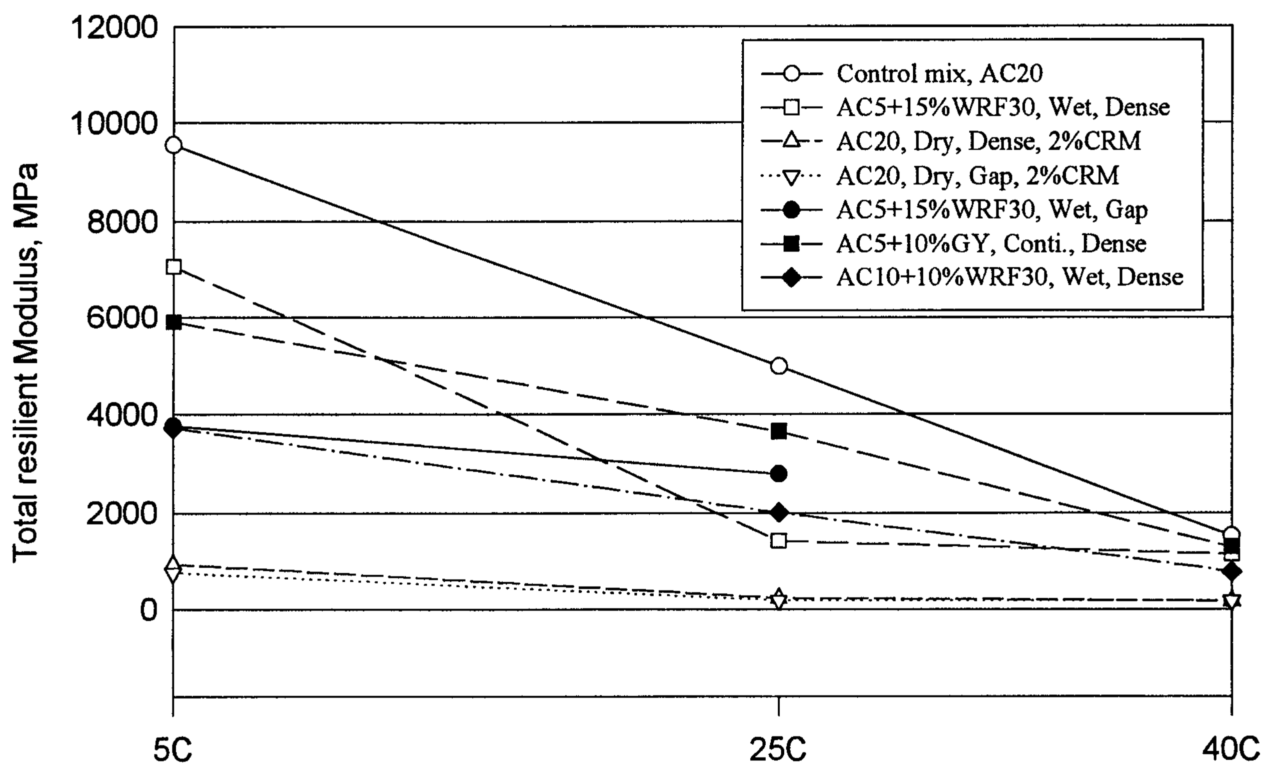


Fig. 5-10 The total resilient modulus vs. temperature for various unaged mixtures

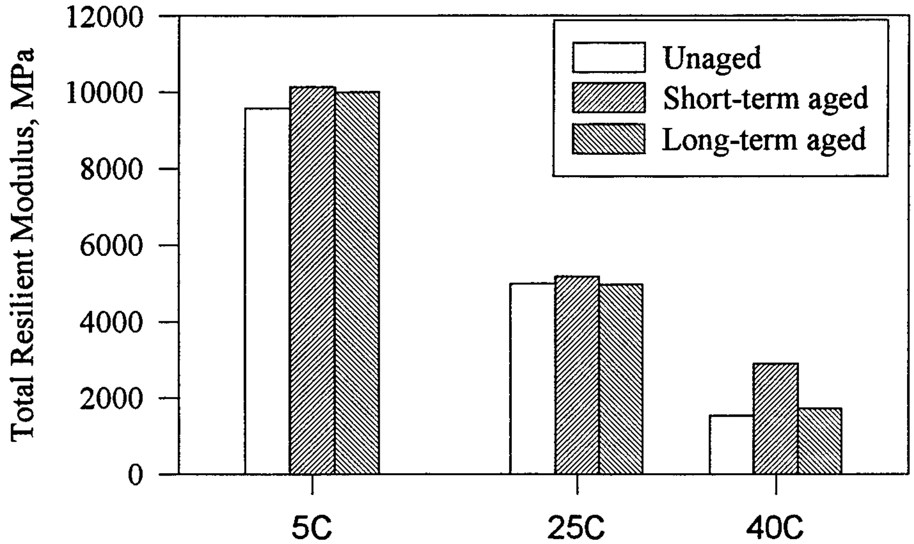


Fig 5-11 Aging effect on the total resilient modulus of control mix AC20

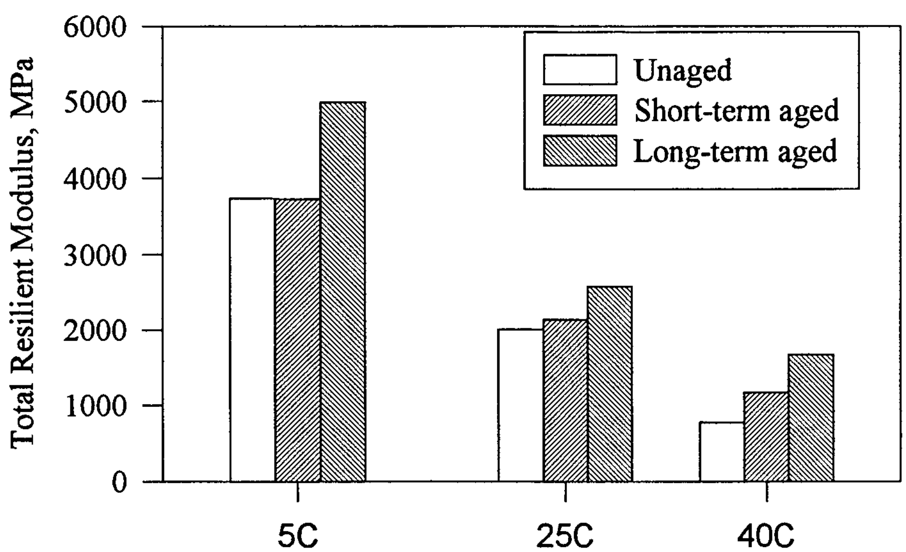


Fig. 5-12 Aging effect on the total resilient modulus of dense-graded mix with AC10+10%WRF30

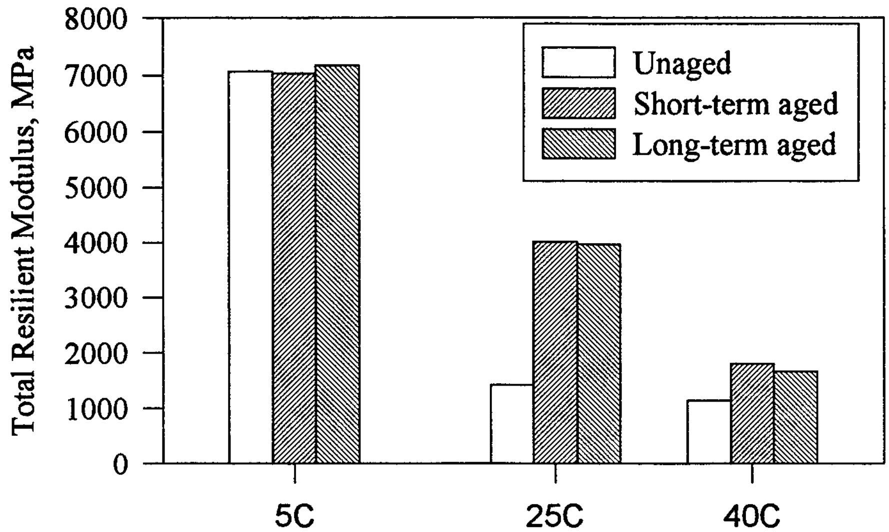


Fig. 5-13 Aging effect on the total resilient modulus of dense-graded mix with AC5+15%WRF30

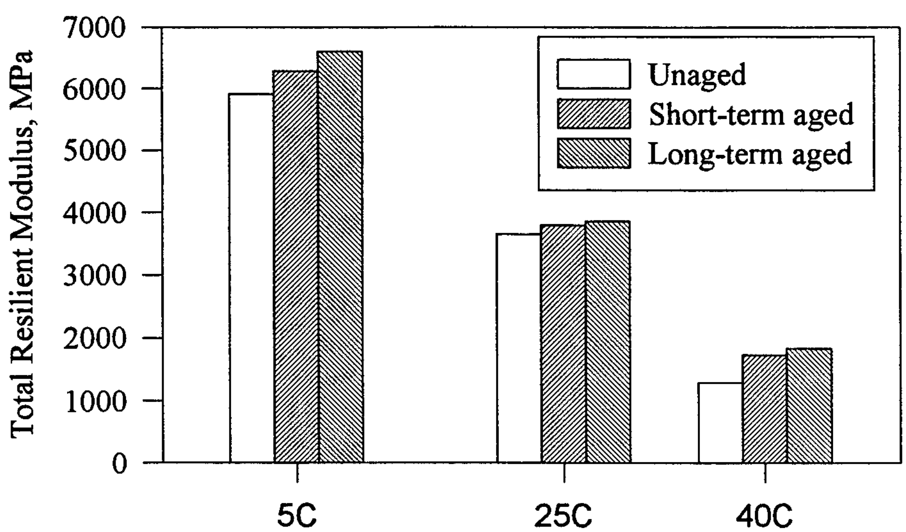


Fig. 5-14 Aging effect on the total resilient modulus of continuous blenging dense-graded mix with AC5+10%GY

incompleteness of data). The aging behavior for the CRM modified mix does not seem to differ drastically from the conventional control mix. Both short-term and long-term aging increase the resilient modulus of the control mix and the CRM modified mixes due to the hardening process.

Table 5-5(a) Summary of the Effect of Aging on the Resilient Moduli

Mixtures	Total Resilient Modulus (MPa)								
	Unaged			Short-Term Aged			Long-Term Aged		
	5°C	25°C	40°C	5°C	25°C	40°C	5°C	25°C	40°C
Control Mix, AC20	9559	4979	1531	10138	5172	2903	10000	4966	1724
Dense Grade, Wet Process, AC10+10%WRF30	3735	2007	780	3725	2134	1169	4983	2575	1672
Dense Grade, Wet Process, AC5+15%WRF30	7069	1421	1138	7034	4014	1807	7172	3960	1669
Dense Grade, Continuous Blending AC5+10%GY	5903	3655	1290	6283	3793	1724	6600	3862	1828
Gap Grade, Wet Process, AC5+15%WRF30	3772	2800	fail	3331	1379	fail	4483	1586	fail
AC20, Dry process Dense-graded with 2%CRM	932	232	171	NA	414	352	NA	422	237
AC20, Dry process, Gap-graded with 2%CRM	754	181	173	NA	344	175	NA	182	199

Table 5-5(b) Summary of the Effect of Aging on the Resilient Moduli

Mixtures	Aging Ratio					
	Short-term aged			long-term aged		
	5°C	25°C	40°C	5°C	25°C	40°C
Control Mix, AC20	1.065	1.041	1.82	1.051	1.00	1.087
Dense Grade, Wet Process, AC10+10%WRF30	1.00	1.06	1.50	1.33	1.28	2.14
Dense Grade, Wet Process, AC5+15%WRF30	1.00	2.82	1.59	1.014	2.78	1.47
Dense Grade, Continuous Blending, AC5+10%GY	1.06	1.057	1.38	1.12	1.057	1.42
Gap Grade, Wet Process, AC5+15%WRF30	0.882	0.50	NA	1.19	0.57	NA
AC20, Dry process, Dense-graded	NA	1.78	2.06	NA	1.82	1.39
AC20, Dry process, Gap-graded	NA	1.90	1.01	NA	1.01	1.09

5.3 Low Temperature Thermal Cracking Resistance Based on TSRST

The ability of the mix to resist low temperature thermal cracking can be measured by the Thermal Stress Restrained Specimen Test (TSRST), as recommended in the SHRP specifications. The low temperature thermal cracking resistance is reflected by a combination of the tensile fracture strength and fracture temperature. If the mix possesses low thermal expansion/shrinkage coefficient, combined with high fracture strength, then the mix should have a better low temperature thermal cracking resistance. The TSRST provides both sets of information about the mix. The fracture temperature indicates the thermal expansion/shrinkage coefficient and the fracture stress relates to the strength of the mix under tension at low temperature. The lower the fracture temperature and the higher the fracture stress correlate with better low temperature thermal cracking resistance.

5.3.1 Introduction to TSRST

Thermal Stress Restrained Specimen Test (TSRST) is a particular test developed by SHRP to evaluate the low-temperature cracking characteristics of the asphalt mixes. The tensile strength and temperature at fracture of the laboratory compacted mixtures are measured by the tensile load in a specimen, which is being cooled at a constant rate and being restrained from contraction. Tensile stress builds up in the specimen as the temperature decreases. When the tensile stress equals the tensile strength of the specimen, the specimens fractures. The objective of using this test in this study is to evaluate the low temperature thermal cracking characteristics of rubber modified asphalt mixtures.

5.3.2 Summary of Test Method

The TSRST was developed by the University of California as part of SHRP research project (project A-003A). A compacted mixture specimen, cored from the three layer rolling wheel compaction mold, is attached at the ends to the platens of a test system and enclosed in an environmental chamber. An initial tensile load is applied to the specimen and the specimen is cooled at a given rate. Thermal contraction in the long axis of the specimen is monitored electronically and the initial length of the specimen is reestablished by automatic adjustment of the platens on an incremental basis to the original position. This process continues until tensile fracture of the specimen occurs. The TSRST system and analysis program was provided by OEM, Inc. in Oregon.

5.3.3 Specimen Preparation

The nine inch high (three layer) rolling wheel compaction mold, originally developed at the University of California at Berkeley, was adopted for the preparation of the asphalt specimens for TSRST. Fig. 5-15 shows a schematic diagram of the nine inch deep rolling wheel compaction mold used in this study.

About 105 kg of mixture (target air void of 6%) was compacted with 1300 lb tandem steel drum roller in three layers. The compacted slab was then allowed to cool for 2 days, after which the cylindrical specimens were cored with a 2 inch coring bit. Actual dimension of the specimen was 1.75 inch in diameter and 8 inch long.

The TSRST was performed on six mixtures as determined previously in the Marshall mix design: (1) dense-graded aggregate with AC-20 as a control mix, (2) wet process, dense-graded aggregate with asphalt-rubber (AC10/10%WRF30), (3) wet process, dense-graded aggregate with asphalt-rubber (AC5/15%WRF30), (4) continuous blending, dense-graded aggregate with asphalt-rubber (AC5/10%GY), (5) wet process, gap-graded aggregate with asphalt-rubber (AC5/15%WRF30), (6) dry process, dense-graded aggregates with 2%CRM (AC20).

5.3.4 Test Results of TSRST

As shown in Fig. 5-16 and summarized in numerical values in Table 5-6, the CRM modified mixes in general exhibit better low temperature thermal cracking resistance, compared to the control mix. Except for the mixes prepared with the continuous blending technology and the dry process, all the other CRM modified mixes showed greater tensile

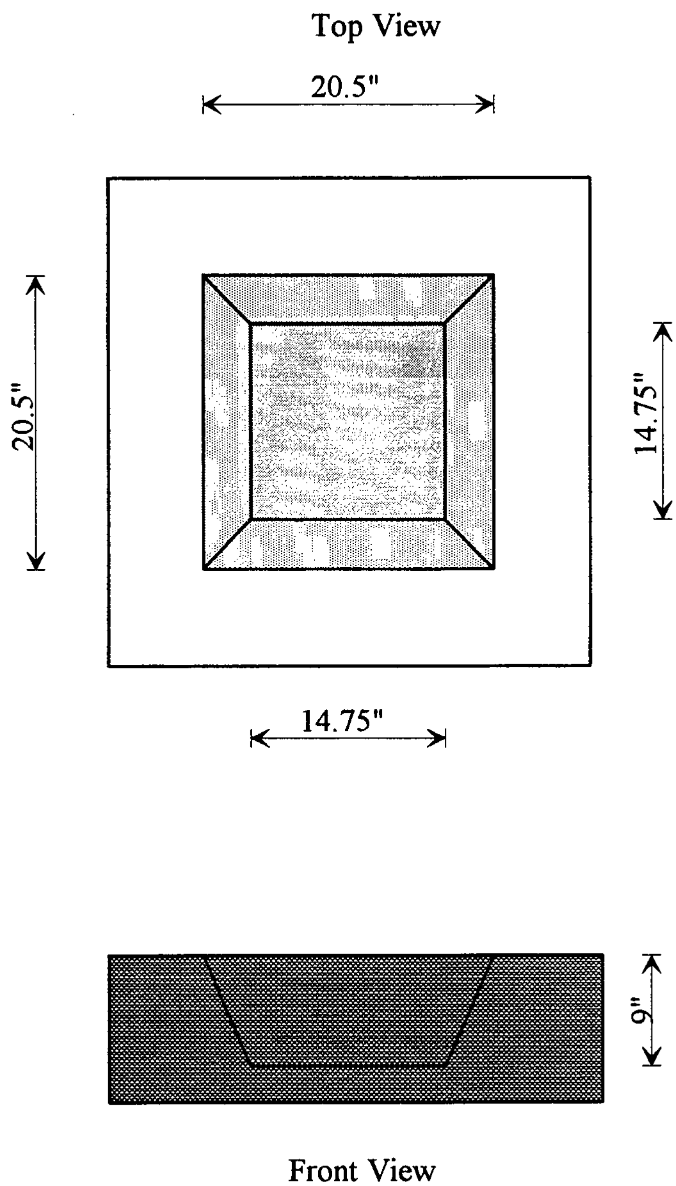


Fig. 5-15 9 inch (3 layers) rolling wheel compaction mold used in TSRST

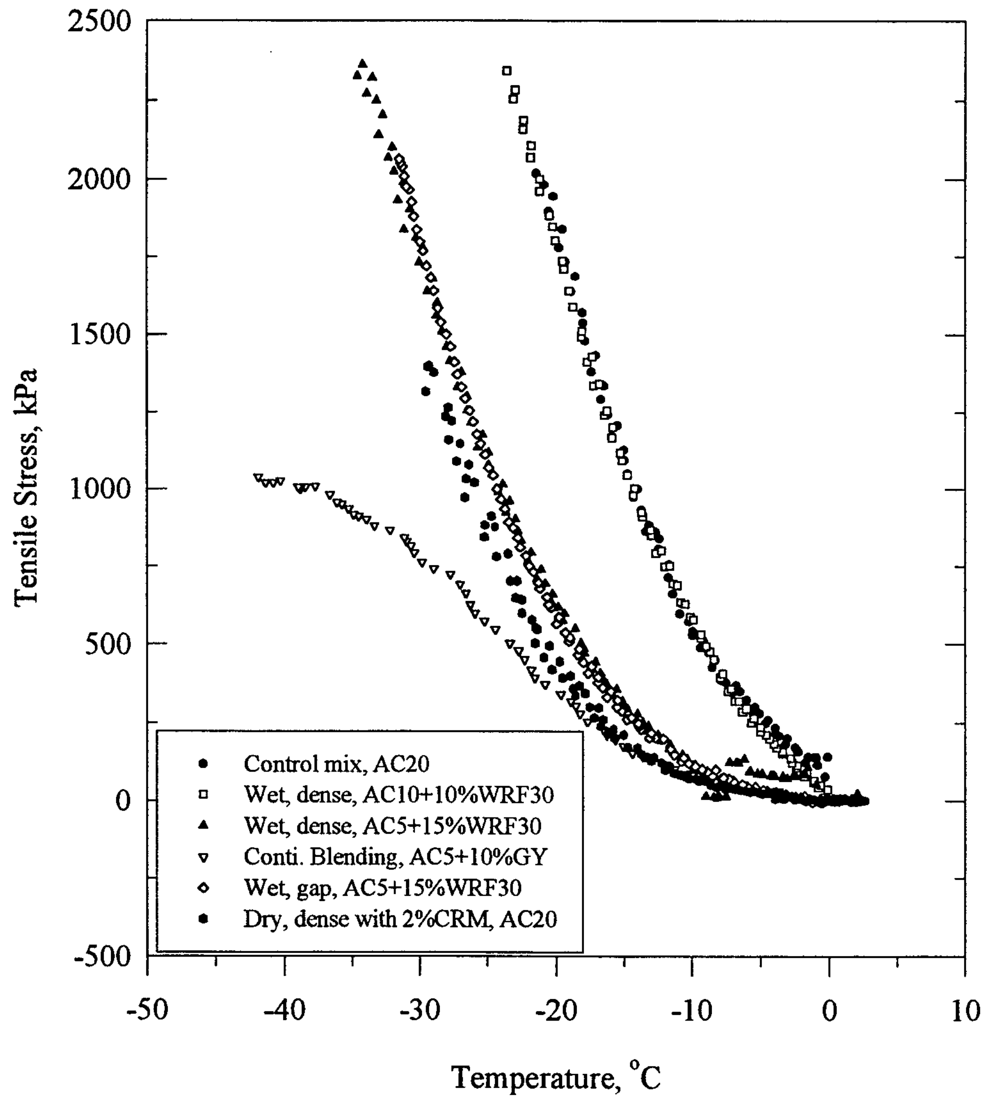


Fig. 5-16 TSRST results for various mixtures

stress at fracture and significant lower fracture temperature.

Table 5-6 Results of TSRST for Various Mixtures

Mixture	Fracture Temperature, °C	Tensile Stress, kPa
Control Mix	-20.8	1814
Dense Graded, Wet Process, AC10+10%WRF30	-23.8	2369
Dense Graded, Wet Process, AC5+15%WRF30	-34.2	2372
Dense Graded, Continuous Blending AC5+10%Ultra Fine GY	-41.9	1041
Gap Graded, Wet Process, AC5+15%WRF30	-32.1	1979
Dry process, Dense Graded with 2%CRM, AC20	-29.3	1400

5.4. Incremental Creep Test:

The incremental creep test is a recommended test by the VESTS program for the determination of the rutting characteristics of the asphalt mix. The test involves an application of axial load of 20 psi (138kPa) to an unconfined cylindrical specimen according to the load spectrum shown in Fig. 5-17. The specimens used in this test were prepared using the rolling wheel compaction which is described in the section of TSRST Test. The test protocol recommends testing the specimens at three different temperatures: 70, 86, and 104°F. For each temperature, two identical specimens should be tested. Figs. 5-18 (a) to (c) show the average result for each test temperature. It can be seen that both the mix design and the test temperature play a role in influencing the rutting characteristics of the mix. To gain a better quantitative understanding of the rutting characteristics, the

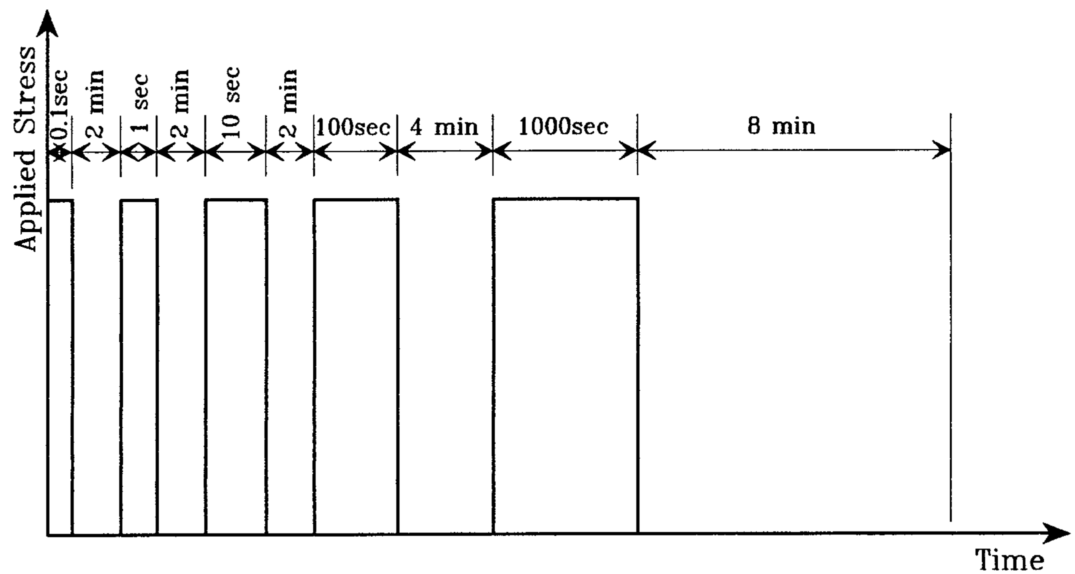


Fig. 5-17 Load spectrum used in incremental creep test

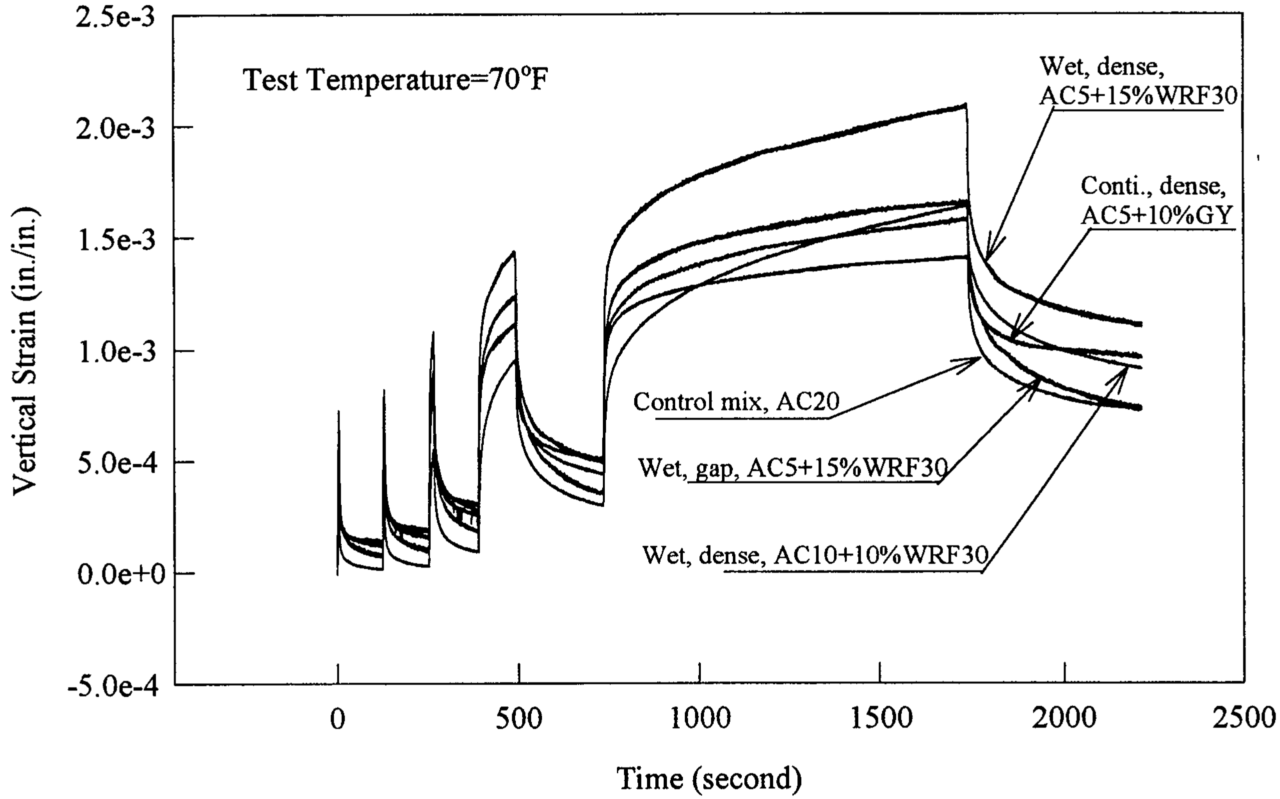


Fig. 5-18(a) Incremental creep test results (70°F)

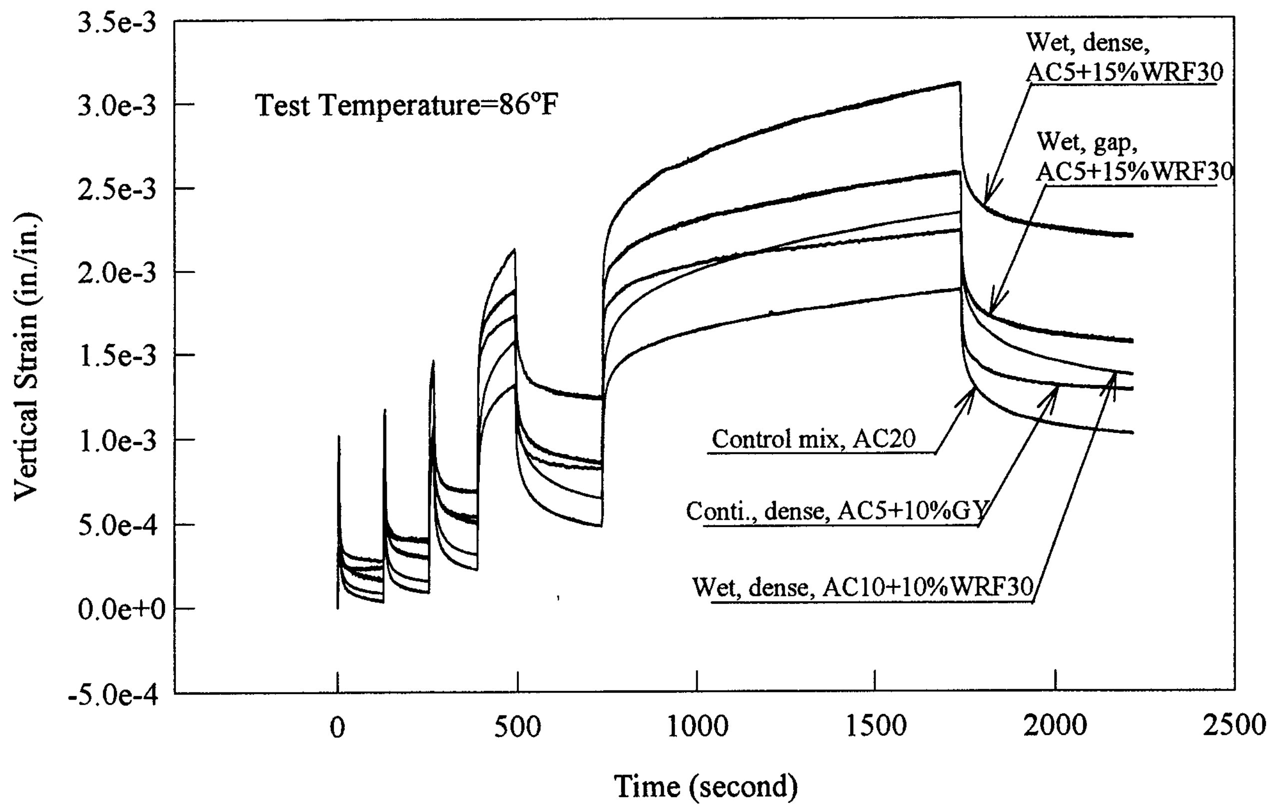


Fig. 5-18(b) Incremental creep test results (86°F)

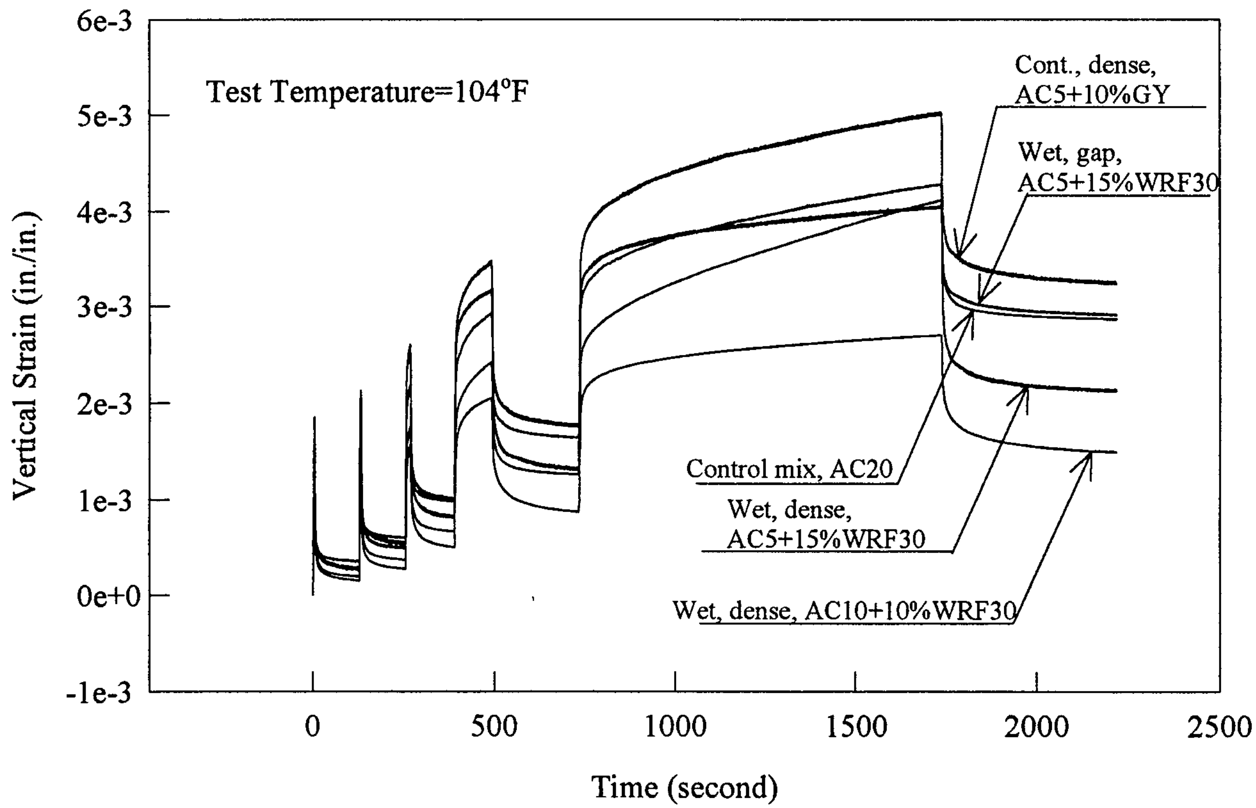


Fig. 5-18(c) Incremental creep test results (104°F) procedure outlined by the VESYS program was followed to further reduce the raw data. A logarithm vs. logarithm plot of the permanent strain vs. the incremental load duration was plotted for each temperature and shown in Figs. 5-19 (a) to (c) for 70, 86, and 104°F, respectively. From these plots, the rutting parameters I and S can be determined to fit into the following empirical power equation to predict the accumulation of permanent strain with the number of load applications.

$$\epsilon_a = IN^S \tag{5-5}$$

Apparently, the larger the numerical values of the parameter I and S, the larger the permanent deformation will accumulate with the number of load repetition N. The reduced rutting parameters I and S are summarized in Table 5-7 for each mix. Note that Eq. (5-5) is not a prediction formula for the permanent strain of the asphalt pavements in situ since the laboratory test condition is different from the loading condition of the pavements. The relationship between the permanent strain ϵ_a and the load repetition N at the temperature 104°F is plotted for the various mixtures in Fig. 5-20 to help identify a better mixture. It can be seen that the AC10+10%WRF30 mix exhibits the best resistance to permanent deformation in the load repetition range.

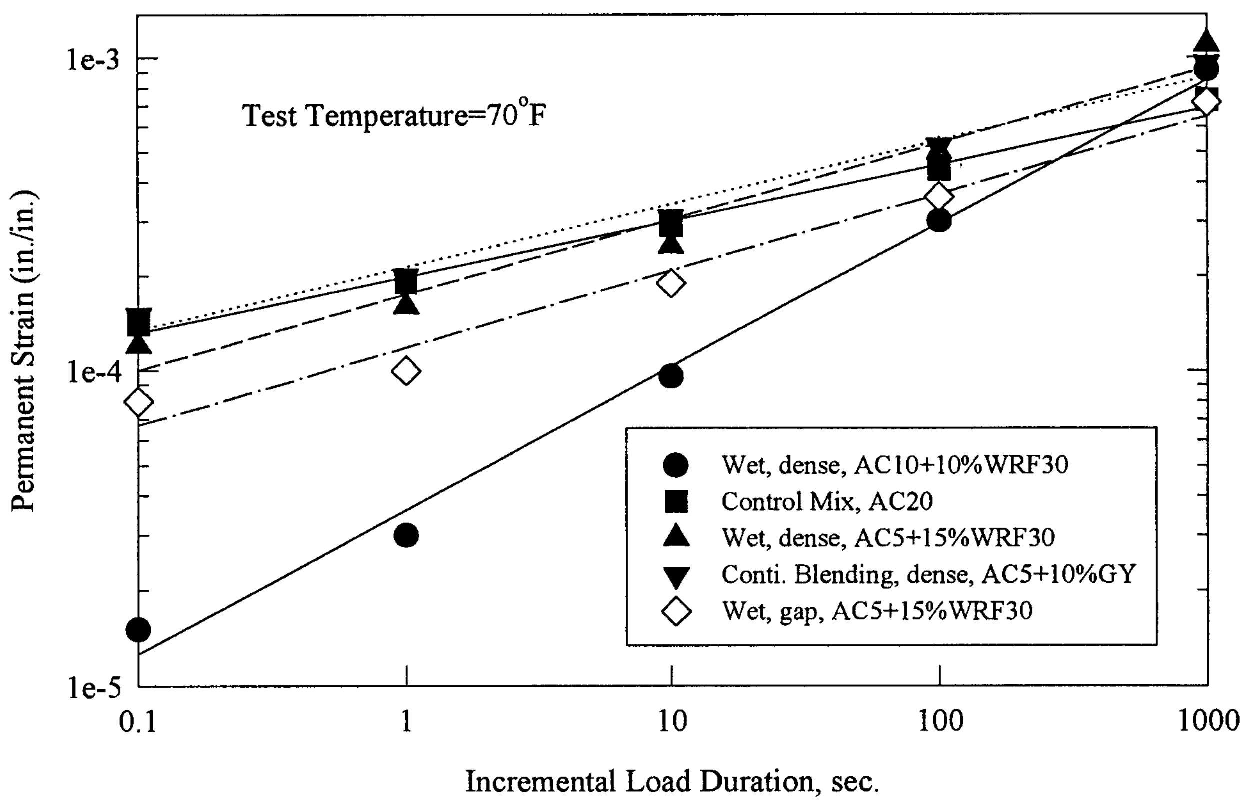


Fig. 5-19 (a) Permanent deformation vs. incremental load duration (70°F)

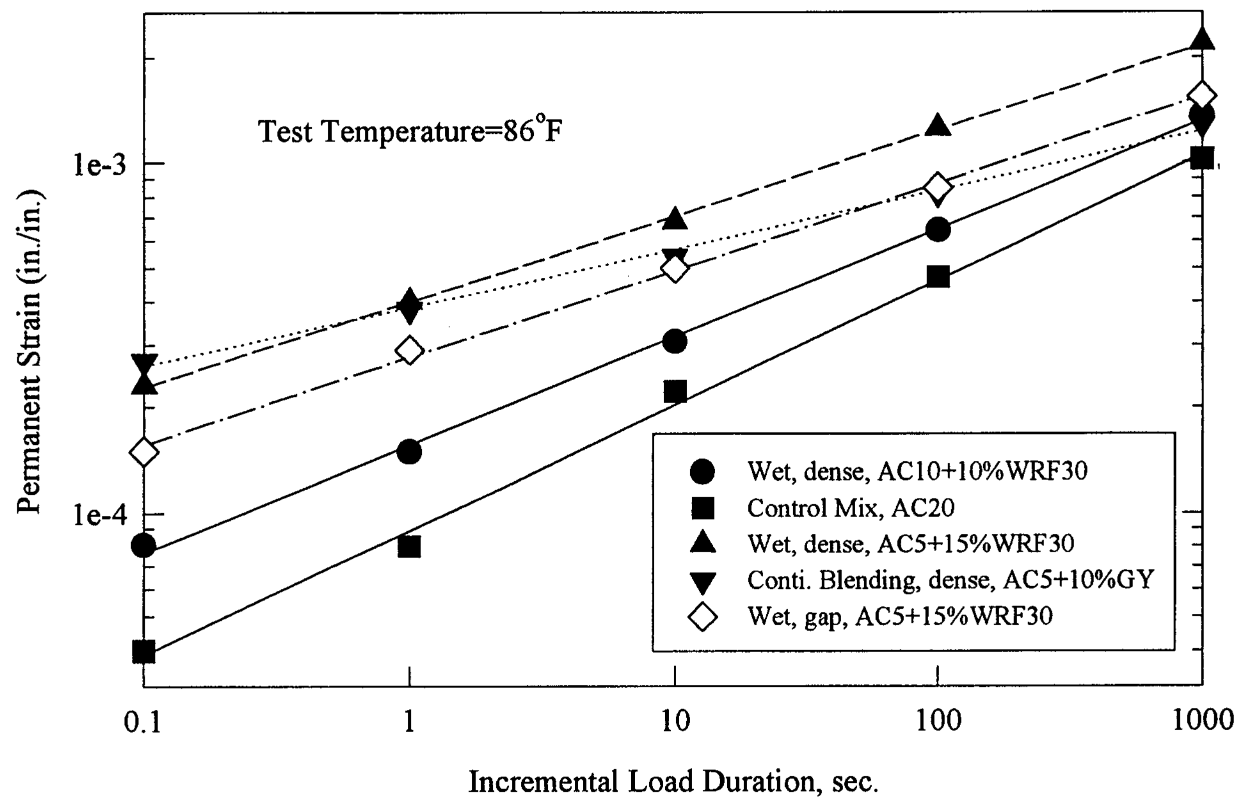


Fig. 5-19 (b) Permanent deformation vs incremental load duration (86°F)

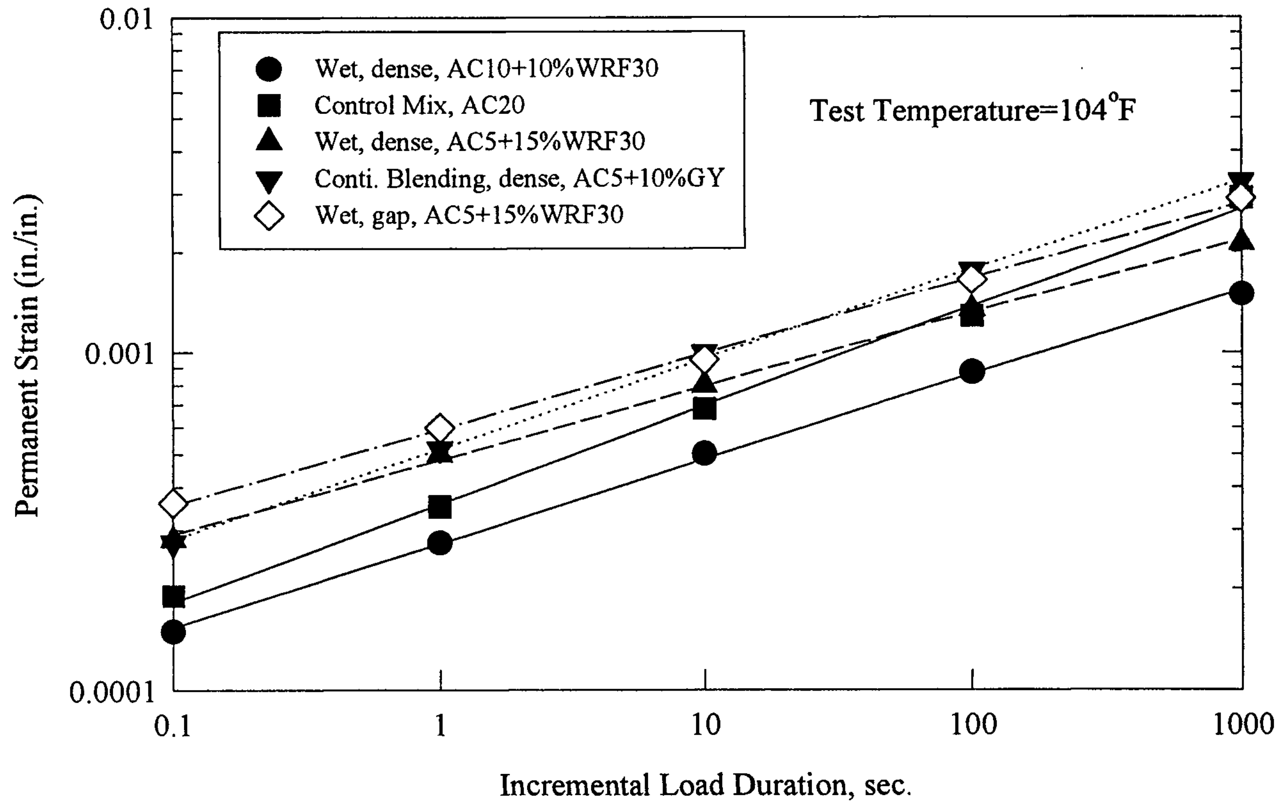


Fig. 5-19 (c) Permanent deformation vs. incremental load duration (104°F)

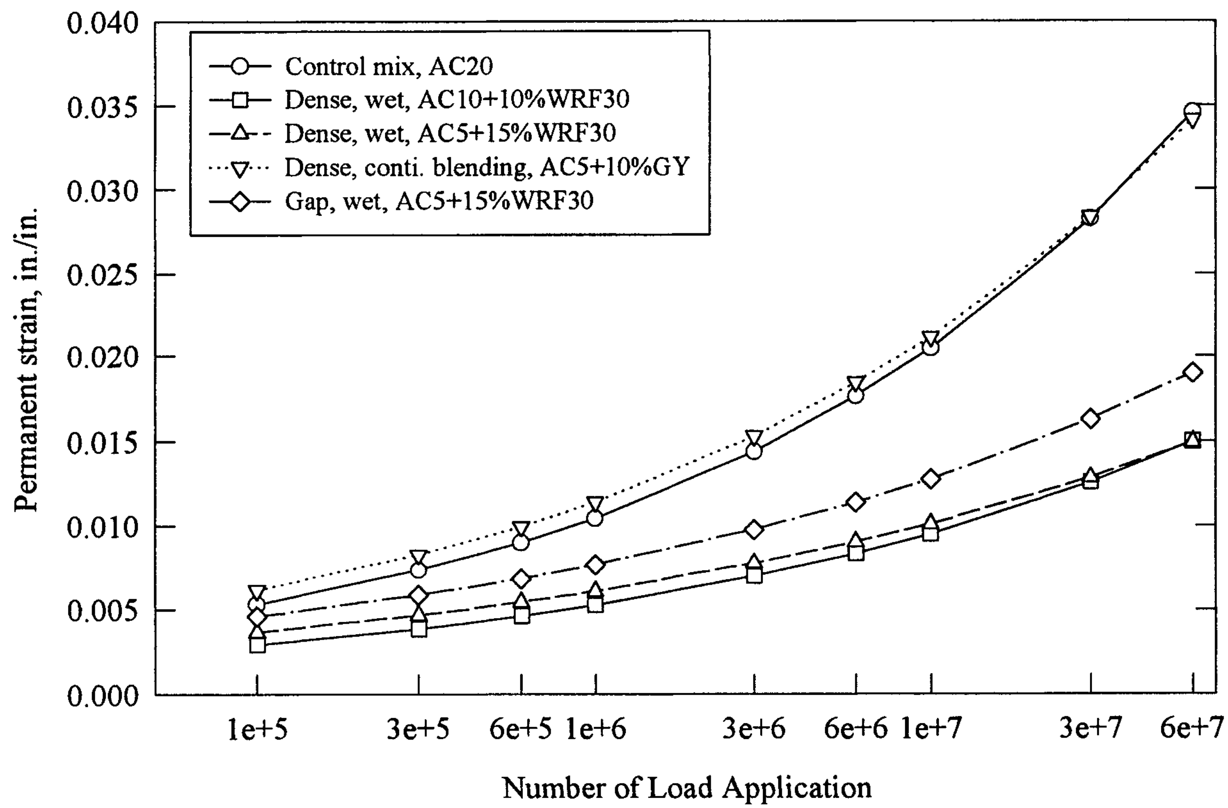


Fig. 5-20 Relationship of permanent strain with load repetition

Table 5-7 The Rutting parameters I and S used in the VESYS Program

Mixture	At 70°F		At 86°F		At 104°F	
	I	S	I	S	I	S
Control Mix, AC20 unmodified	1.31e-4	0.181	3.9e-5	0.256	1.82e-4	0.293
Dense Grade, Wet Process, (AC10+10%WRF30)	1.27e-5	0.467	7.70e-5	0.309	1.53e-4	0.256
Dense Grade, Wet Process, (AC5+15%WRF30)	1.e-4	0.241	2.27e-4	0.246	2.91e-4	0.220
Dense Grade, Continuous Blending, (AC5+10%Ultra fine GY)	1.34e-4	0.202	2.61e-4	0.169	2.81e-4	0.268
Gap Grade, Wet Process, (AC5+15%WRF30)	6.66e-4	0.246	1.57e-4	0.248	3.56e-4	0.222

5.5 Loaded Wheel Track Test

The loaded wheel track test is designed to evaluate the rutting resistance of asphalt concrete as a supplement to the creep test. Since the Laboratoire des Ponts et Chaussees (LPC) introduced the wheel-tracking rutting tester in France in 1970's, the wheel track tester has gained popularity in evaluating the rutting resistance of the asphalt concrete specimens. Different types of wheel track tester have been manufactured. In this study, a wheel track tester manufactured by Wessex Engineering&Metalcraft Ltd in Britain was used. It consists mainly of a 8" (203mm) diameter, 2" (50mm) wide solid rubber wheel bearing on the specimen underneath via a cantilevered arm with 520 N load. A reciprocating table holds the specimen and moves back and forth on a pair of tracks. On the top of the wheel is a cap which is connected to a position sensor so that the rutting depth can be directly measured and then transferred to the computer. The temperature is controlled through two thermometers: one is to monitor the cabinet air temperature and the other one is to monitor the temperature of the asphalt mixture sample.

Specimens were compacted using a tandem steel-wheeled roller. The mold for holding the mixture for rolling compaction is 15 by 15 inches on the bottom, 16 by 16 inches on the top, and 2 inches in depth. The specimens were cored using a core bit resulting a specimen with a diameter of 200mm and a thickness of 50mm.

Before running a test, the specimen is preheated to 104°F (40°C). The running time is 45 minutes and the total number of pass is 1860. The results for the various mixtures are shown in Table 5-8. From the table, it can be seen that the AC10+10%WRF30 and AC5+10%GY have higher rutting resistance than the control mix AC20, while the gap-graded AC5+15%WRF30 has lower rutting resistance. The dense-graded AC5+15%WRF30 showed similar rutting resistance as the control mix.

Table 5-8 Loaded Wheel Track Test results for various mixtures

Mixes	Rutting depth, mm	Rut in percent
AC20, control mix dense-graded	3.75	7.5
AC10+10%WRF30 wet process, dense-graded	3.15	6.3
AC5+15%WRF30 wet process, dense-graded	3.95	7.9
AC5+10%GY, wet process dense-graded	3.15	6.3
AC5+15%WRF30 wet process, gap-graded	4.25	8.5

5.6 Water Sensitivity Test

The ability of the asphalt mix to retain strength after cycles of freeze and thaw when the asphalt specimen is almost fully saturated is an important consideration, particularly when the durability of the paving mixture is concerned. The water sensitivity test is also designed to investigate the binding performance of the binder. If there is a lack of binding strength between the binder and the aggregate, then the asphalt-aggregate mix can exhibit raveling under severe weather condition. In the present study, the water sensitivity test was conducted according to the AASHTO T 283-89. The freshly prepared mixtures were placed in an aluminum pan at the room temperature for 2 hours. Then the mixtures were placed in a 140°F(60°C) oven for 16 hours for curing. After curing, the mixtures were placed in the oven at 275°F(135°C) for 2 hours prior to compaction. The target air void is 7% in this test. The compacted specimens were left at room temperature for about 24 hours before extraction. After extraction the specimens were stored at room temperature for 72 to 96 hours. Then the bulk specific gravity and theoretical maximum specific gravity were determined. Then air void was determined for each specimen. The specimens were divided into two groups, one of which was subjected to water conditioning. The specimens for water conditioning were subjected to a vacuum of 10 to 26 inches Hg for 5 to 10 minutes. After the vacuum, the specimens remained in the water for additional 5 to 10 minutes. The vacuum and the time could vary in order to achieve a degree of saturation of 55% to 80%. Each saturated specimen was wrapped with a plastic film and placed in a plastic bag containing 10 mm of water. The plastic bags that contained the conditioned specimens were placed in a freezer at 0°F (-18°C) for 16 hours.

After removal from the freezer, the specimens were placed in a water bath at 140°F(60°C) for 24 hours. Then the conditioned specimens as well as the unconditioned specimens were placed in a water bath at 77°F(25°C) for 2 hours prior to subjecting them to the indirect tensile test. For each mix, a total of three identical Marshall specimens were tested and the average results are reported in Table 5-9. None of the mix shows a satisfactory Tensile Strength Ratio (TSR) of over 0.80. The CRM modified asphalt mixes in this study are particularly susceptible.to the moisture induced damage by the AASHTO test method. This problem is worthy of further investigation.

Table 5-9 Water Stripping Test Results

Mixture	Unconditioned (kPa)	Conditioned (kPa)	Ratio (Average)
Control Mix, AC20	1052	762	0.72
WDR, AC10+10%WRF30	847	385	0.46
WDR, AC5+15%WRF30	591	182	0.30
Continuous Blending, AC5+10%GY	-	-	NA
WGR, AC5+15%WRF30	-	-	NA

5.7 Fatigue Test

5.7.1 Introduction

Distress of asphalt concrete pavement due to repeated bending from traffic loads has been a well-recognized problem in the United States since 1948 (Hveem and Carmany, 1948). In order to study and understand the fatigue-induced distress in the asphalt concrete pavement, it is necessary to conduct laboratory fatigue tests that affords the type of stress states encountered in situ.

According to Matthews and Monismith (1992), existing fatigue test methods can be conveniently categorized into the following types: simple flexural testing, supported flexural testing, direct axial testing, diametral testing, triaxial testing, fracture mechanics based testing, and wheel-track testing. The three most promising methods identified by Matthews and Monismith (1992) were simple flexure testing, diametral fatigue testing, and tests based on fracture mechanics principles. However, both the diametral and the fracture mechanics based fatigue tests were eliminated from the final SHRP selection. A brief discussion of the flexural beam fatigue test method is given below.

Flexural Beam Fatigue Test

In the flexural beam fatigue test, a simply supported asphalt concrete beam specimen is subjected to either third-point or center point loading. The test can be either load or deflection controlled.

The University of California at Berkeley (Deacon, 1965) recommended the specimen size of 1.5 in. x 1.5 in. x 15 in. (38.1 mm x 38.1 mm x 381 mm). The Asphalt Institute, on the other hand, recommended a larger specimen size of 3 in. x 3 in. x 15 in. (76.2 mm x 76.2 mm x 383 mm). Both institutes preferred the third-point loading, with a load period of 0.1 second and a frequency of 100 repetitions per minute. The European approach, as illustrated by the Shell Laboratory at Amsterdam (Heukelom and Klomp, 1964; van Dijk, 1977), preferred the center point loading. The specimen size was usually 1.2 in. x 1.6 in. x 9.2 in. (30 mm x 40 mm x 230 mm).

SHRP Equipment and Procedure

The flexural beam (third-point) fatigue test method conducted in the controlled-strain mode of loading was selected by the SHRP. This mode of loading was considered to be more compatible with the crack propagation concept and pavement fatigue cracking models that were being developed as part of SHRP Project A-005. Beams of the dimensions of 2.5 in. x 2.0 in. x 15 in. were utilized. Sinusoidal loads up to 25 Hz frequency can be applied with or without rest periods.

Significant improvements in fatigue data have resulted from the use of the new test equipment and procedure, as documented in the SHRP study. The coefficient of variation (CV) for fatigue life has been reduced from almost 90 percent to about 40 percent. This was most likely due to improvement in the control of the repeated strain, as well as the use of larger beam specimens compacted by using rolling wheel compaction. The use of rolling wheel compaction virtually eliminated fracturing of the aggregate which was observed in the pilot test program where the specimens were prepared using kneading compaction.

5.7.2 Previous Studies

Al-Abdul-Wahhab and Al-Armi (1991) conducted the diametral fatigue test on CRM-modified asphalt concrete mixture at 25^0 C and 45^0 C. The fatigue data were analyzed to determine the fatigue parameters in the following equation:

$$\varepsilon_t = a\, N_f^{\ b} \tag{5-6}$$

where ε_t = initial tensile strain; N_f = number of load repetitions to failure; a = antilog of the intercept of the logarithmic relationship; and b = slope of the logarithmic relationship. Results of their test are summarized in Tables 5-10 and 5-11 for 25°C and 45°C temperature range, respectively.

The exponent b was found to increase with increasing temperature to 45^0 C. Specimens of 10% CRM-modified binder were found to have the highest fatigue life for both mixes at 25^0 C and 45^0 C, indicating a stiffer mix. The slope of the logarithmic fatigue life was lowest for mixes of 10%CRM, except mix G2 at 45°C where its slope was found to be slightly higher than the other mixes.

Table 5-10 Fatigue life parameters at 25^0 C (Al-Abdul-Wahhab and Al-Armi, 1991)

Mix type	CRM content %	a	b	R^2
G1	0	6.913E4	-0.605	0.868
	10	2.551E4	-0.485	0.951
	20	1.01E6	-0.860	0.874
	30	5.493E4	-0.602	0.951
G2	0	9.65E4	-0.603	1.0
	10	1.47E4	-0.429	1.0
	20	6.37E4	-0.593	0.919
	30	1.44E5	-0.686	0.993
3% RUMAC	0	2.17E3	-0.220	1.0

Table 5-11 Fatigue life parameters at 45^0 C (Al-Abdul-Wahhab and Al-Armi, 1991)

Mix type	CRM content %	a	b	R^2
G1	0	3.145E6	-1.024	0.962
	10	3.413E6	-0.766	0.969
	20	5.40E5	-0.849	0.995
	30	3.87E5	-0.831	0.999
G2	0	1.392E6	-0.895	0.968
	10	3.861E6	-0.97	0.984
	20	1.828E5	-0.725	0.979
	30	2.941E5	-0.782	0.940
3% RUMAC	0	2.57E3	-0.198	0.872

As the test temperature increased, the slope of the fatigue curves became smaller for mixes with 20% and 30% CRM-modified binder when compared to the control mixes G1 and G2. Mixes with 3% RUMAC showed no improvement in fatigue life; however, compared to the different mixes, the slope of the regression line for these mixes was the least.

Researchers at Texas A&M University (Hoyt et al., 1989) conducted the third-point bending tests to study the fatigue characteristics of asphalt-rubber concrete used for airport pavements. This study followed the procedures of fatigue testing described in the VESTS IIM Manual. VESTS procedure suggests the use of a repeated load flexure device with the beam specimens. The third-point loading configuration theoretically applied a constant bending moment over the central 4 inch portion of a 15 inch long specimen. This study used a device which applied a repeated tension-compression load in the form of a haversine wave for 0.1 second duration with 0.4 second rest periods. Tests were performed in the temperature chamber at 34, 68, and 104° F, respectively.

The regression lines were fitted on the fatigue test results and the fatigue parameters were calculated according to the following fatigue equation:

$$N_f = k_1 \left(\frac{1}{\varepsilon_t} \right)^{k_2} \tag{5-7}$$

where

N_f = number of load repetitions to failure,

ε_t = tensile strain induced, and k_1, k_2 = regression constants.

The asphalt concrete control mix was prepared with an AC-10, which has an optimum binder content of 4.8 percent. The asphalt-rubber used in their study was produced by the Arizona Refining Company and was a mixture of 77 percent AC-10 asphalt cement, 20 percent rubber, and 3 percent extender oil. The rubber gradation included particle sizes between the No. 16 and No. 200 sieves. Optimum binder content for asphalt rubber mix was 4.8 percent.

Two sets of regression equations shown in Table 5-12 were generated to describe the variation of the fatigue parameters with temperature for each material.

Table 5-12 Regression equations to predict fatigue parameters for any temperature (^{0}F) for materials at optimum binder contents (Hoyt et al., 1989)

	$\log k_1$ vs. $\log T$ (^{0}F)
Material	Regression Equations
AC	$\log k_1 = 14.630 - 4.558 \log T$
ARC	$\log k_1 = 20.484 - 7.879 \log T$
	k_2 vs. $\log k_1$
Material	Regression Equations
AC	$k_2 = 1.512 - 0.280 (\log k_1)$
ARC	$k_2 = 1.900 - 0.243 (\log k_1)$

Some researchers from Oregon State University (Lundy et al., 1987; Takallou et al., 1986) conducted the fatigue test on RUMAC specimens. After determining the resilient modulus, the same specimens were tested in the diametral configuration using a constant stress mode. Results of their fatigue tests are shown in Table 5-13.

Table 5-13 Results of fatigue test on RUMAC specimens from Oregon State University

Specimen Description	Regression Equation
Lundy, Hicks, and Richardson (1987)	
3% RUMAC, cored in 1983	$N_f = 5.68 \times 10^{-19} (1/\varepsilon_t)^{6.07}$, $R^2 = 0.51$
3% RUMAC, cored in 1984	$N_f = 1.24 \times 10^{-15} (1/\varepsilon_t)^{4.89}$, $R^2 = 0.65$
3% RUMAC, cored in 1986	$N_f = 6.23 \times 10^{-14} (1/\varepsilon_t)^{4.48}$, $R^2 = 0.97$
Control, cored in 1983	$N_f = 6.56 \times 10^{-7} (1/\varepsilon_t)^{2.55}$, $R^2 = 0.96$
Control, cored in 1984	$N_f = 2.92 \times 10^{-8} (1/\varepsilon_t)^{2.86}$, $R^2 = 0.88$
Control, cored in 1986	$N_f = 2.21 \times 10^{-7} (1/\varepsilon_t)^{2.67}$, $R^2 = 0.87$
Takallou, Hicks, and Esch (1986)	
3% RUMAC, Gap	$N_f = 1.07 \times 10^{-10} (1/\varepsilon_t)^{3.6}$, $R^2 = 0.99$
3% RUMAC, Gap, Surcharge	$N_f = 4.08 \times 10^{-9} (1/\varepsilon_t)^{3.3}$, $R^2 = 0.98$
2% RUMAC, Gap	$N_f = 1.18 \times 10^{-8} (1/\varepsilon_t)^{3.0}$, $R^2 = 0.97$
3% RUMAC, Dense	$N_f = 1.14 \times 10^{-7} (1/\varepsilon_t)^{2.8}$, $R^2 = 0.98$
Control, Dense	$N_f = 9.94 \times 10^{-8} (1/\varepsilon_t)^{2.7}$, $R^2 = 0.9$

5.7.3 Test Procedures Used in this Study

A schematic diagram of the strain-controlled flexural beam (third-point) fatigue test is shown in Fig. 5-21. A sinusoidal load of a frequency of 10 Hz was used for performing the fatigue test in the present study. Fig. 5-22 shows a schematic representation of the strain-controlled fatigue load application. Fig. 5-23 shows the recorded trace of load versus time in one of the fatigue test. Notice that there is no rest period between each load cycle.

Strain-controlled fatigue beam test was performed on four different mixes: (1) dense-graded aggregate with AC20 as a control mix, (2) dense-graded aggregate with

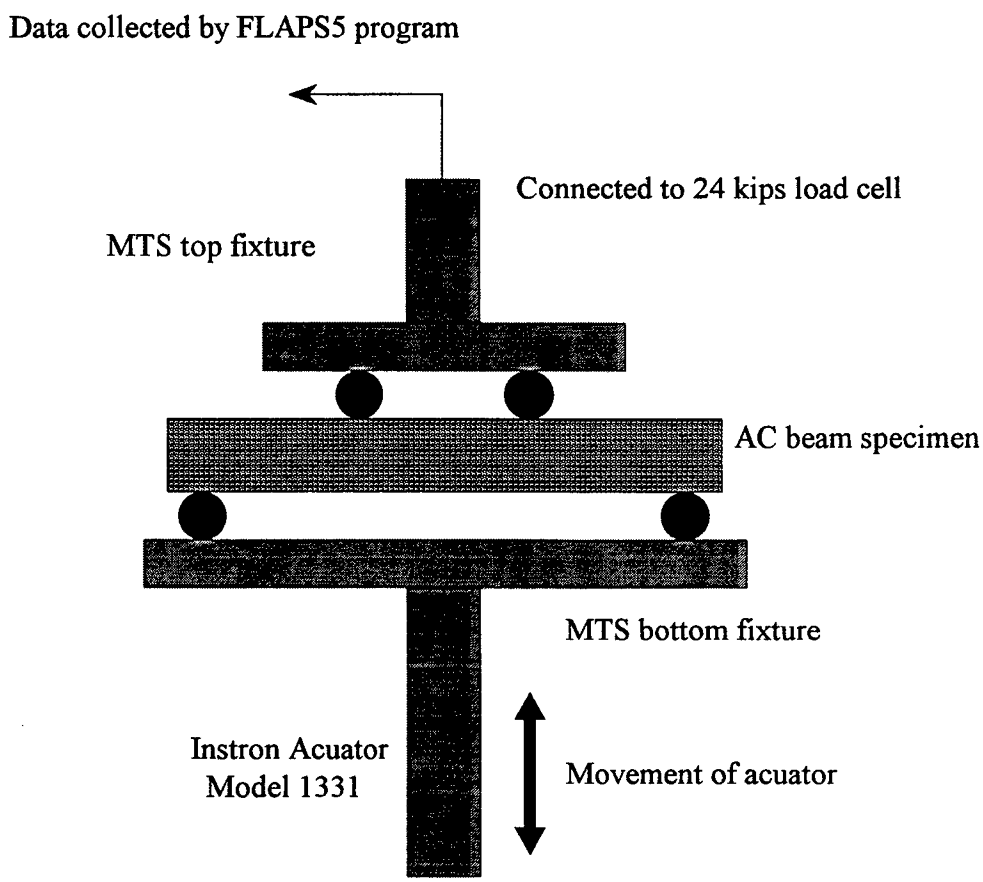


Fig. 5-21 Schematic diagram of specimen set-up for fatigue test

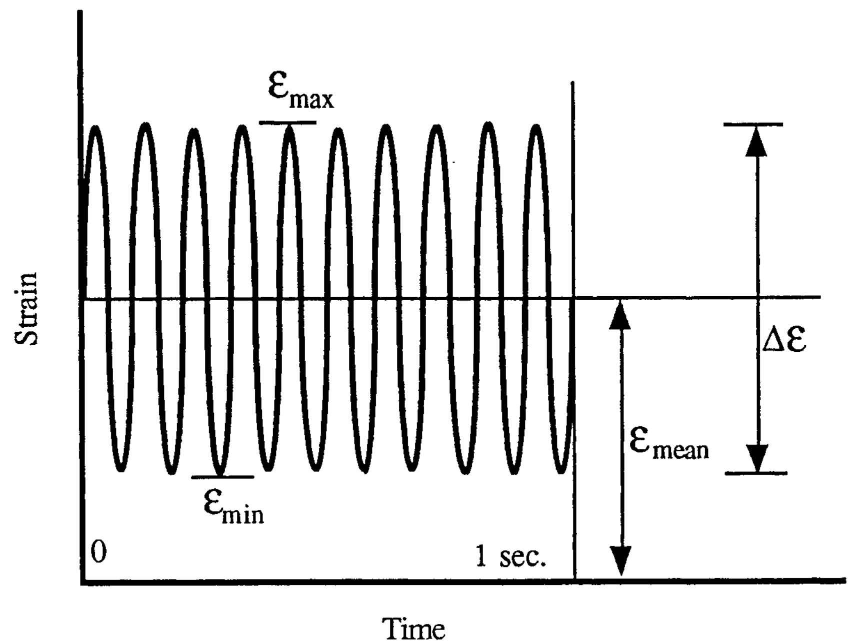


Fig. 5-22 Terminology used in strain-controlled fatigue test
(frequency = 10 Hz)

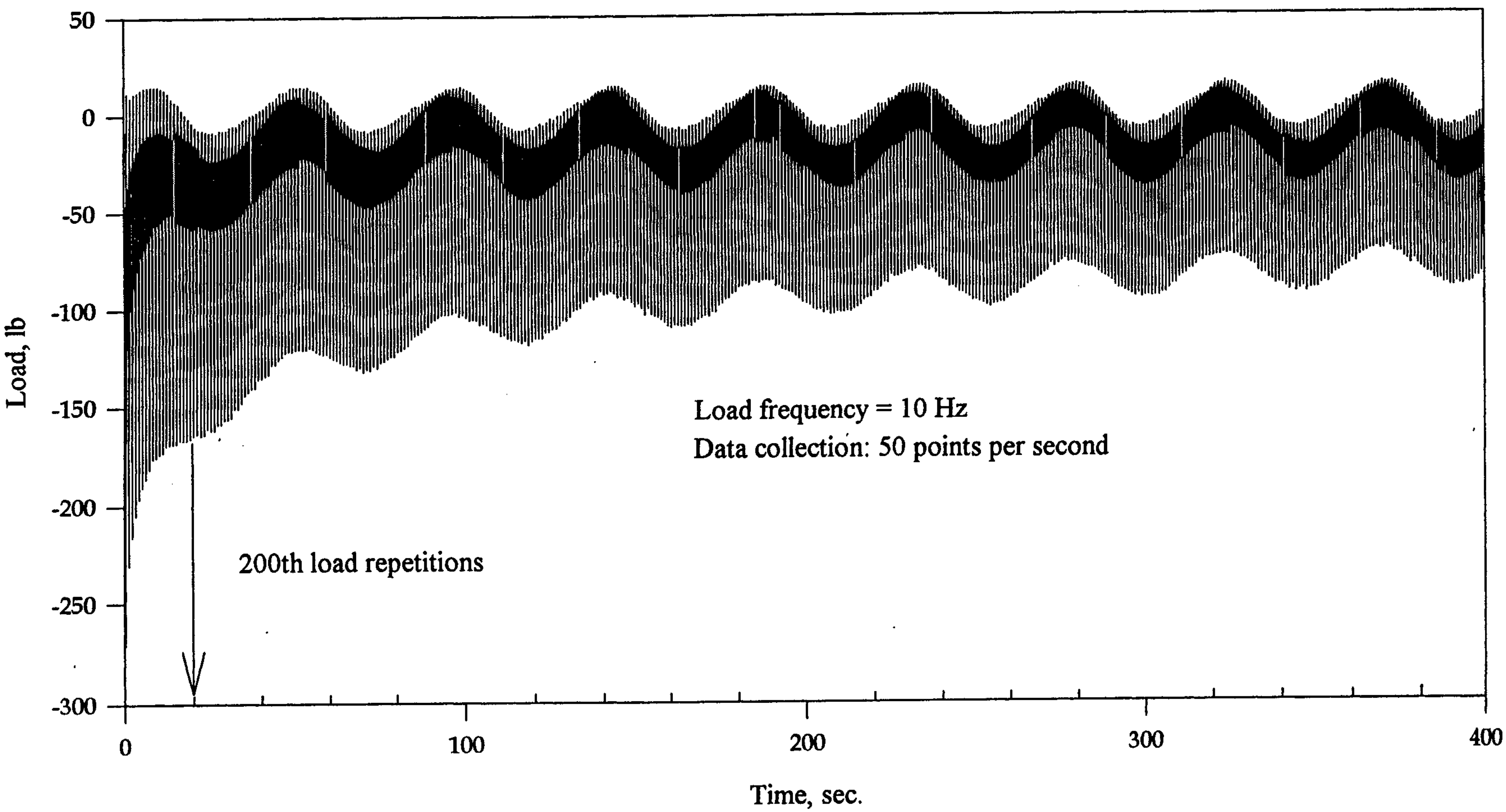


Fig. 5-23 Typical strain-controlled fatigue test result

Ecoflex, (3) dense-graded aggregate with asphalt-rubber (AC5 with 15% WRF 30), and (4) dense-graded aggregate with 2% RUMAC. Tests were conducted at 20^0 C with each mix subjected to six different initial tensile strain levels. Typical initial tensile strain ranges were between 200 to 2000 micro in./in.. These initial tensile strains were calculated from the center-point displacements using Eqs 5-8 to 5-10.

$$\sigma = \frac{3aP}{bh^2} \tag{5-8}$$

$$E_s = \frac{Pa(3L^2 - 4a^2)}{4bh^3\Delta} \tag{5-9}$$

$$\varepsilon_t = \frac{\sigma}{E_s} = \frac{12h\Delta}{3L^2 - 4a^2} \tag{5-10}$$

The notations used in the above equations are illustrated in Fig. 5-24, in which σ is the extreme fiber stress, a is the distance between the load and the nearest support, P is the total dynamic load with $P/2$ applied at each third point, b is the specimen width, h is the specimen depth, E_s is the stiffness modulus based on the center deflection calculation, L is the span length between the supports, Δ is the deflection at the center of the beam, and ε_t is the extreme fiber tensile strain.

5.7.4 Specimen Preparation

The three inch high rolling wheel compaction mold, originally developed at the University of California at Berkeley, was adopted for the preparation of the rubber

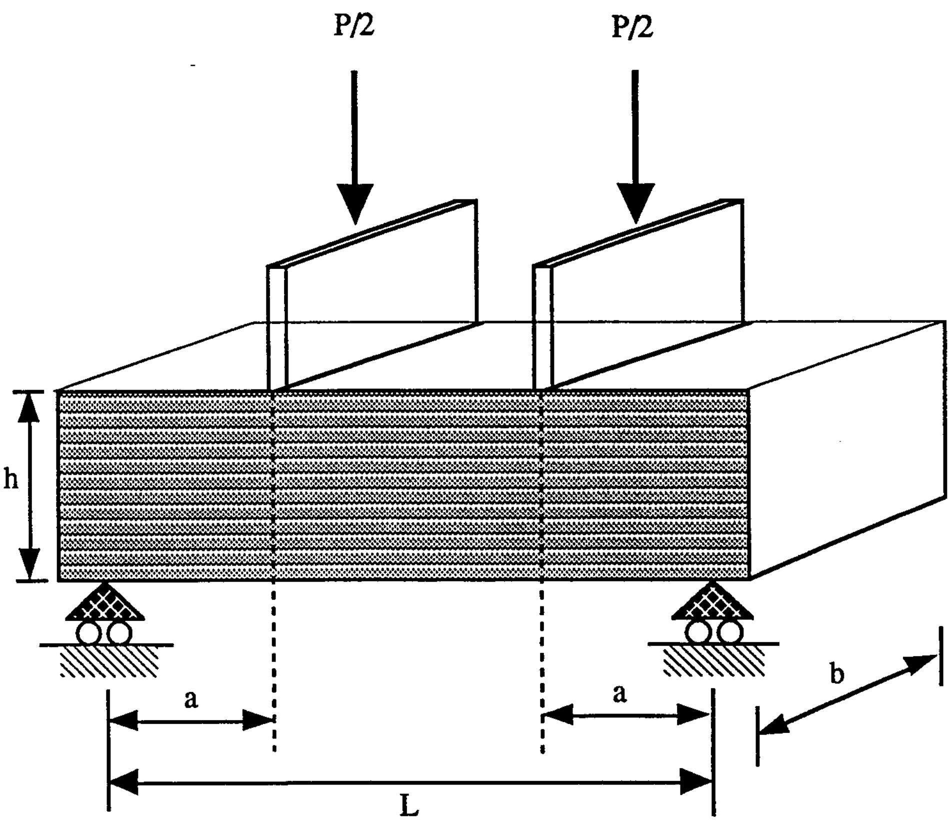


Fig.5-24 Notations of third-point bending beam specimen for fatigue test

modified asphalt concrete beam specimen. Fig 5-25 shows a schematic diagram of the rolling wheel compaction mold used in this study. Usually, 4 layers of heavy aluminum foil were laid down at the bottom of the mold before filling in the loose mixture and roller compaction. The aluminum foil was extended long enough over the platform of the mold to allow for the lifting of the asphalt concrete slab out of the mold after the specimen has undergone overnight cooling.

Preheated (350^0 F) and preweighted aggregate were added in 1 cubic foot mixing bowl followed by adding the preheated asphalt cement (300^0 F) or asphalt-rubber binder (350^0 F). The mixture, typically of 25 kg, was mixed in a single batch. Infrared heater was placed underneath the mixing bowl while mixing. After mixing, the mixture was placed in a forced-draft oven at 135^0 C (275^0 F) for an hour.

About 65 kg of mixture (target air void of 6%) was compacted with 1300 lb tandem steel drum roller (Duomat). The compacted slab was then allowed to cool overnight, after which the beam specimens were cut with a saw to a rough dimension of 2 in. x 2 in. x 11 in.. A more precise cut was done with a table top concrete saw.

5.7.5 Results of Fatigue Test

Fatigue life was defined as the number of cycles of load application required for a 50% reduction from the initial stress, which was measured at 200th load repetition. Figs. 5-26 to 5-29 show the fatigue test results plotted in terms of applied constant tensile strain versus load repetitions to failure. Table 5-14 summarizes the fatigue parameters regression analysis of fatigue data.

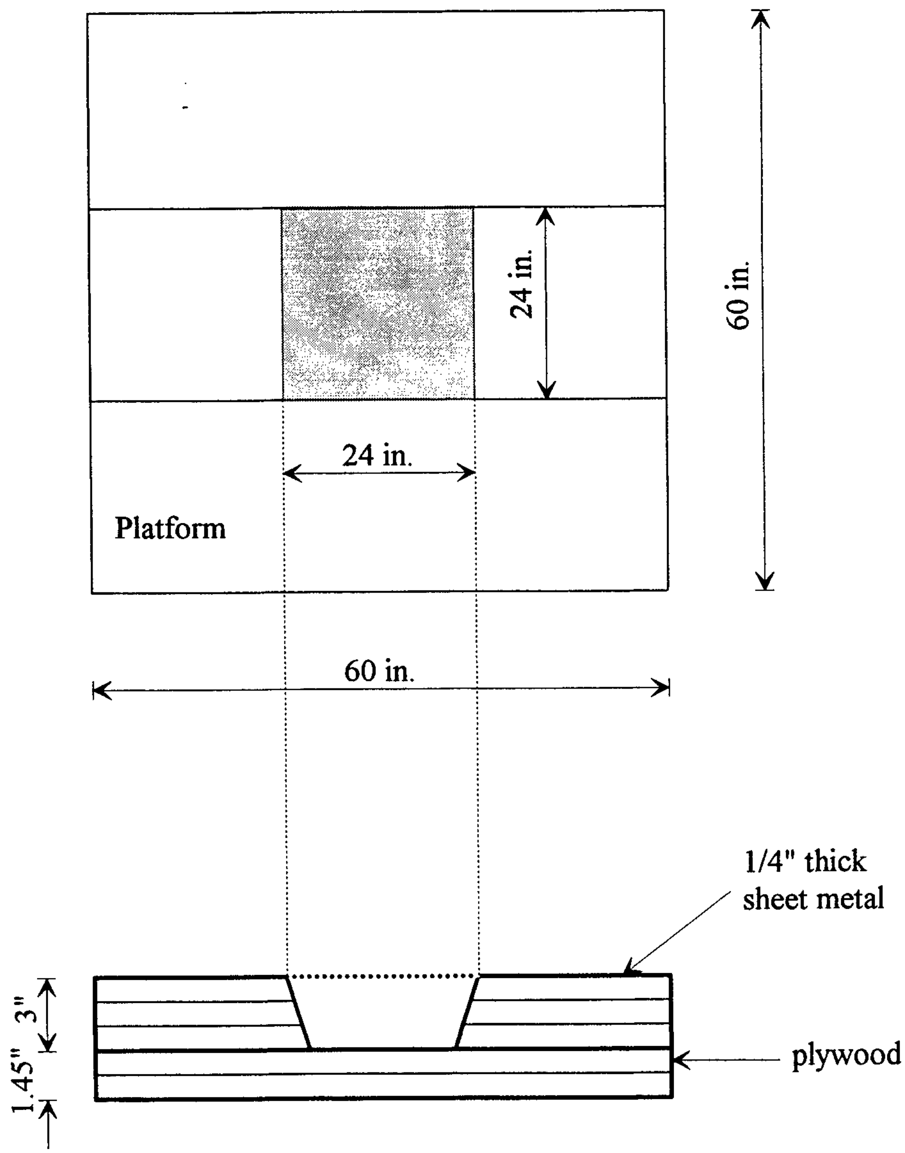


Fig. 5-25 Schematical diagram of the rolling wheel compaction mold used in fatigue test

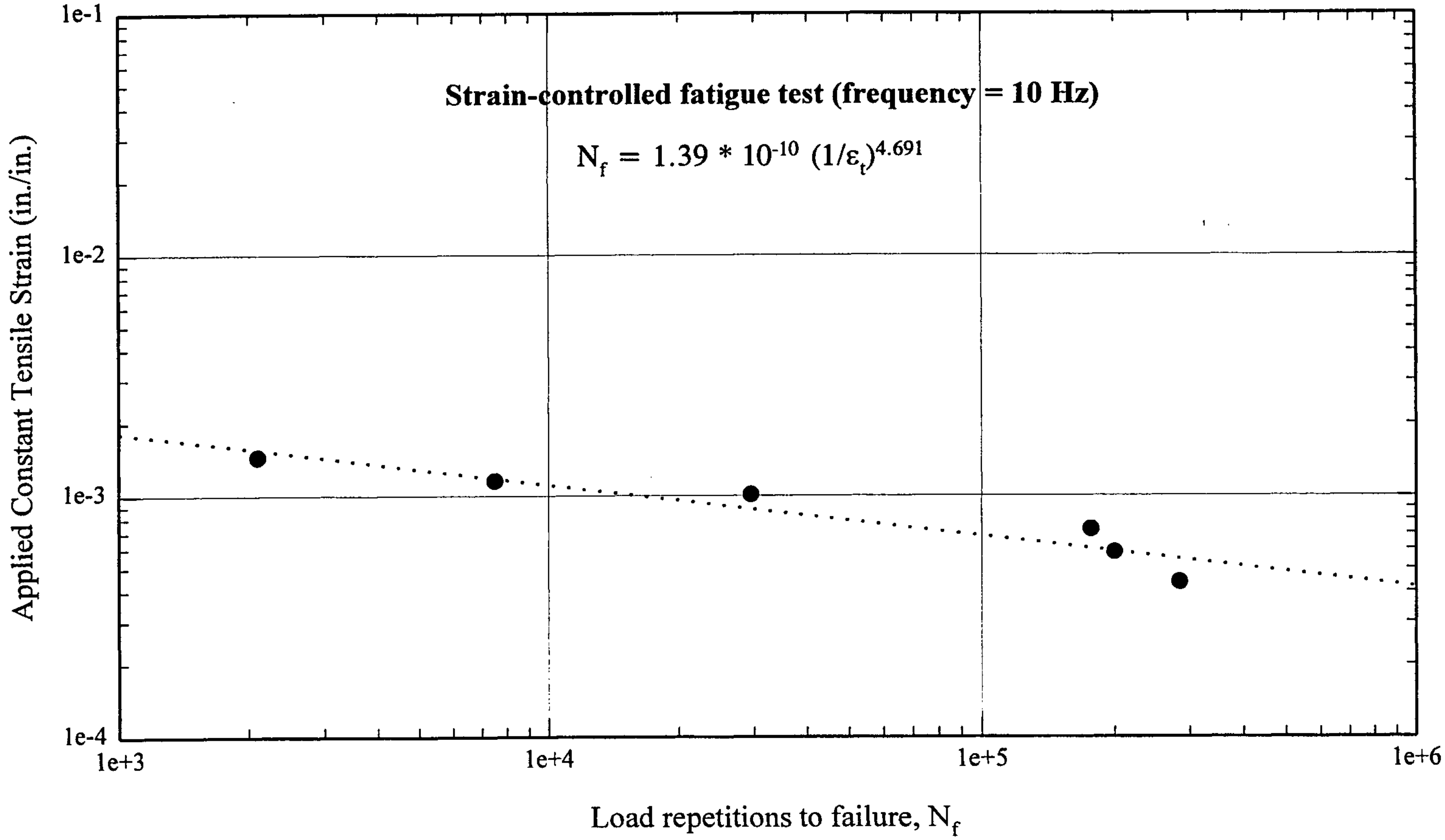


Fig. 5-26 Result of fatigue test for control mix (dense-graded aggregate with AC-20)

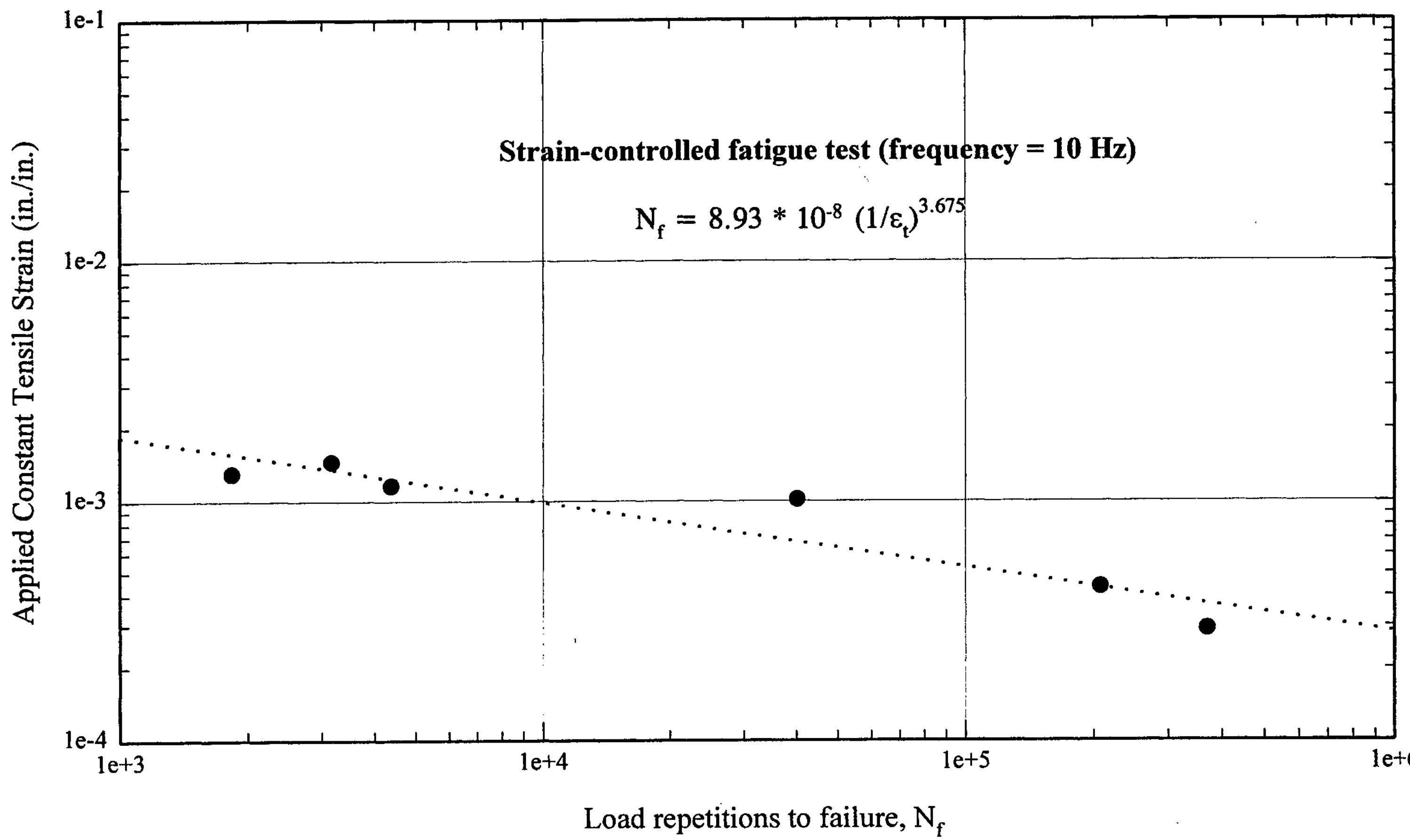


Fig. 5-27 Result of fatigue test for dense-graded aggregate with Ecoflex

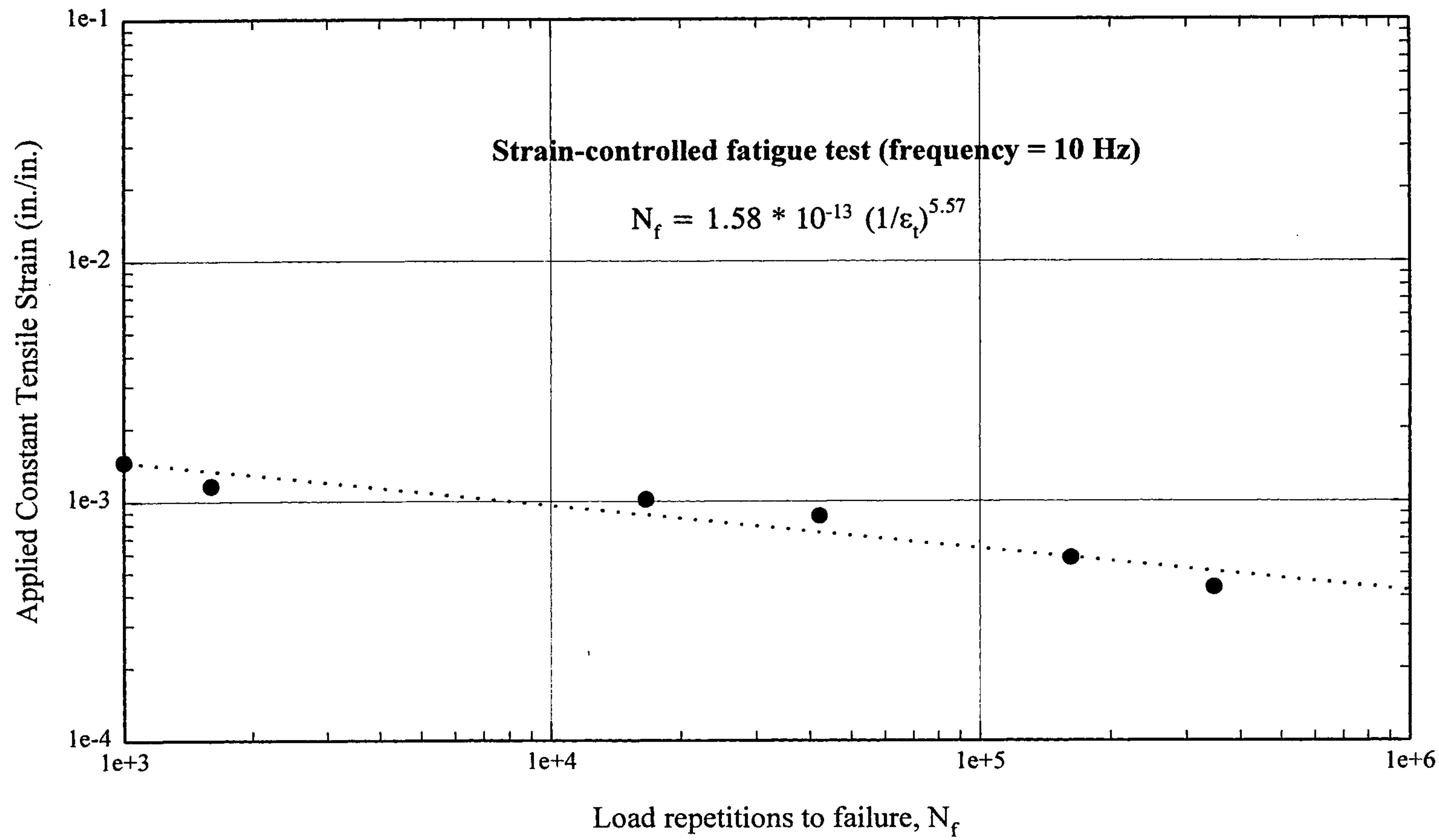


Fig. 5-28 Result of fatigue test for dense-graded aggregate with asphalt-rubber AC5+15%WRF30

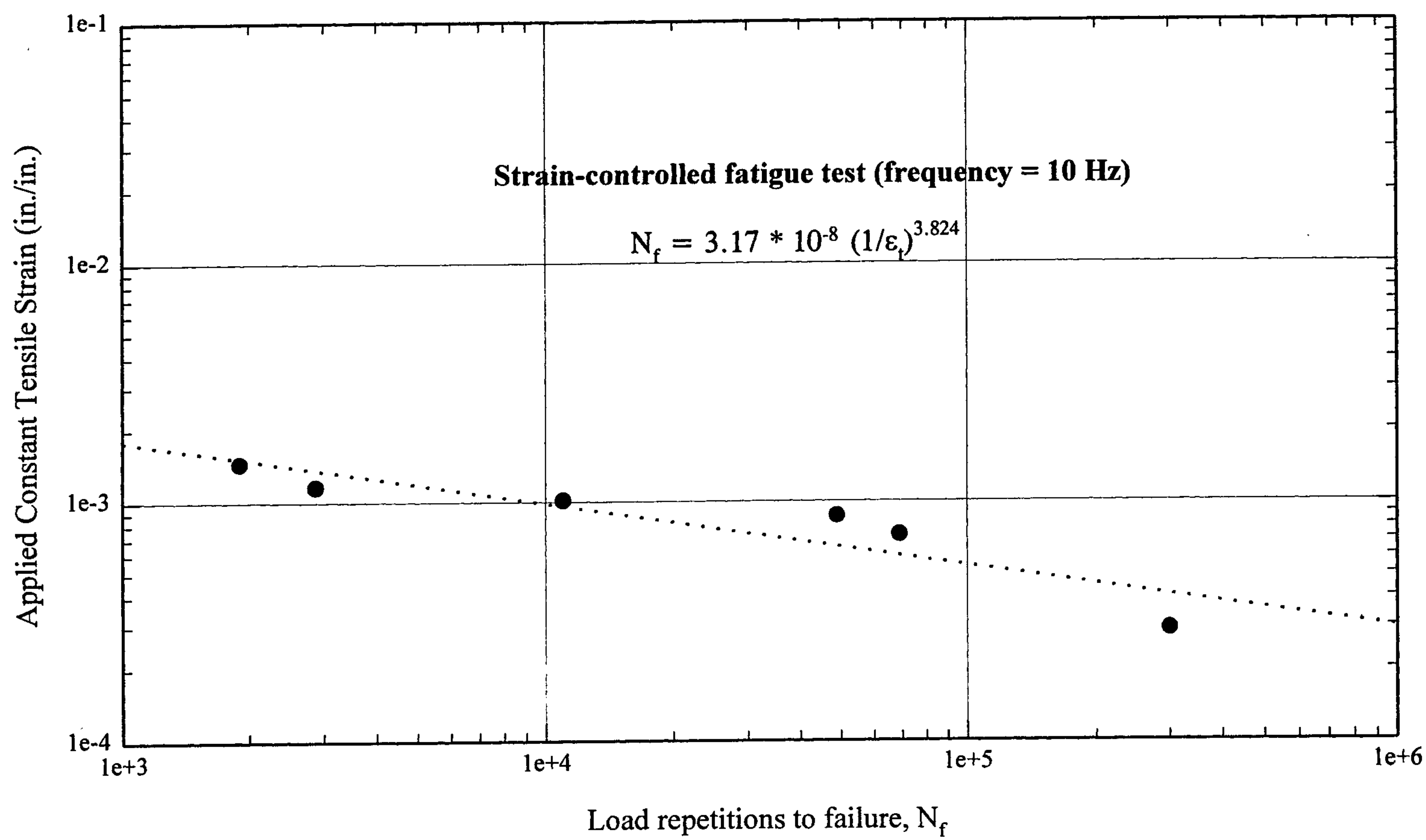


Fig. 5-29 Result of fatigue test for dense-graded aggregate with 2%RUMAC

Table 5-14 Regression equations of fatigue tests for different mixes

Specimen Description	Regression Equation
Dense-graded with AC-20	$N_f = 1.39 \times 10^{-10} (1/\varepsilon_t)^{4.691}$, $R^2 = 0.894$
Dense-graded with Ecoflex	$N_f = 8.93 \times 10^{-8} (1/\varepsilon_t)^{3.675}$, $R^2 = 0.877$
Dense-graded with asphalt-rubber	$N_f = 1.58 \times 10^{-13} (1/\varepsilon_t)^{5.57}$, $R^2 = 0.910$
Dense-graded with 2% RUMAC	$N_f = 3.17 \times 10^{-8} (1/\varepsilon_t)^{3.824}$, $R^2 = 0.832$

The mixture with the AC-5 based asphalt-rubber (wet process) showed the longest fatigue life at overall tensile strain ranges. Also, the wet precess mixture showed the highest number in inverse slope of straight line (k_2 parameter) for the tensile strain versus N_f plot. It means that the slope of the regression line in tensile strain versus load repetition to failure (N_f) plot for the wet process, dense-graded asphalt-rubber mixture was flatter than those of others.

CHAPTER VI

PERFORMANCE PREDICTION AND LIFE CYCLE COST ANALYSIS

6.1 Introduction

This chapter provides the results of performance analysis of the rubber modified asphalt concrete pavements using the computer program KENLAYER. The material properties presented in the previous chapter, including the resilient modulus and parameters for fatigue cracking, are used as input in the computer analysis. A comparison of service life between the rubber modified asphalt concrete section and the conventional HMA section is given, followed by the life cycle cost analysis. It should be noted that although the rutting distress mode is addressed in the computer program KENLAYER, only the permanent deformation due to the subgrade is considered in prediction of the pavement life. Therefore the life prediction by the rutting distress mode is not accurate. In fact, the pavement life predicted by the rutting distress mode in the computer program KENLAYER is much longer than that predicted by the fatigue distress mode. The Superpave computer program for low temperature thermal cracking is not available at the present time.

6.2 Review of Analysis Method of Flexible Pavements

Historically, pavement design procedures have relied on empirical test methods that correlate with the past performance of pavement structures. This suggests that uncertainty exists with the use of alternative materials until a data base that represents the performance of new materials is established (Lee, 1991).

The two existing approaches to the structural modeling of flexible pavement systems are based on either multi-layered elastic theory or finite element method. Historically, the CHEVRON-based multi-layered program had some advantages in having the ability to accommodate horizontal surface loading and variable degrees of bonding between layers. The major advantage of the finite element method (e.g. ILLI-PAVE) is that it can accommodate the stress dependent behavior of the materials comprising the flexible pavement layers. The recent development of computer program KENLAYER could accommodate nonlinear behavior of subgrade and subbase materials in predicting life expectancy according to the fatigue distress mode and the rutting distress mode. But as was mentioned above the rutting distress mode only addresses the permanent deformation due to the subgrade. The permanent deformation of the asphalt concrete is not considered.

The first mechanical analysis/design of flexible pavements was attempted by Burmister (1943), who considered the pavement as a layered elastic system. The use of layered elastic theory has made it possible to identify critical points in a pavement structure where the strain should not exceed certain limiting values as determined by the characteristic of the materials considered. Two different limiting strain criteria are considered in the design of flexible pavement structures. One is the limiting tensile strain criterion at the bottom of asphalt bound layers. The criterion is intended to control cracking due to repetition of tensile strain. The other criterion limits the subgrade compressive strain, which is used to control excessive rutting.

6.3 KENLAYER

The KENLAYER computer program is a multi-layered pavement analysis program developed by Professor Huang and his associates at the University of Kentucky. The program can be used to estimate the number of wheel load repetitions to failure, where the failure condition is defined in two types of distress: fatigue cracking and permanent deformation (rutting). The backbone of KENLAYER program is the solution for an elastic multi-layer system with a circular loaded area representing the tire load. Damage analysis can be made by dividing each year into a maximum of 24 periods, each with a different set of material properties to reflect the seasonal temperature change resulting in resilient modulus change. Each period can have a maximum of 24 load groups, either single or multiple. The damage caused by fatigue cracking and permanent deformation in each period over all load groups is summed up to evaluate the design life.

In the methodology adopted for the KENLAYER, loads on the surface of the pavement produce two strains which, as shown in Fig. 6-1, are critical for design purpose. They are (1) horizontal strain, ε_t, on the underside of the lowest asphalt-bound layer, and (2) the vertical compressive strain, ε_c, at the surface of the subgrade layer. If the horizontal strain is excessive, fatigue cracking of the asphalt layer will result. If the vertical compressive strain is excessive, permanent deformation will result at the surface of

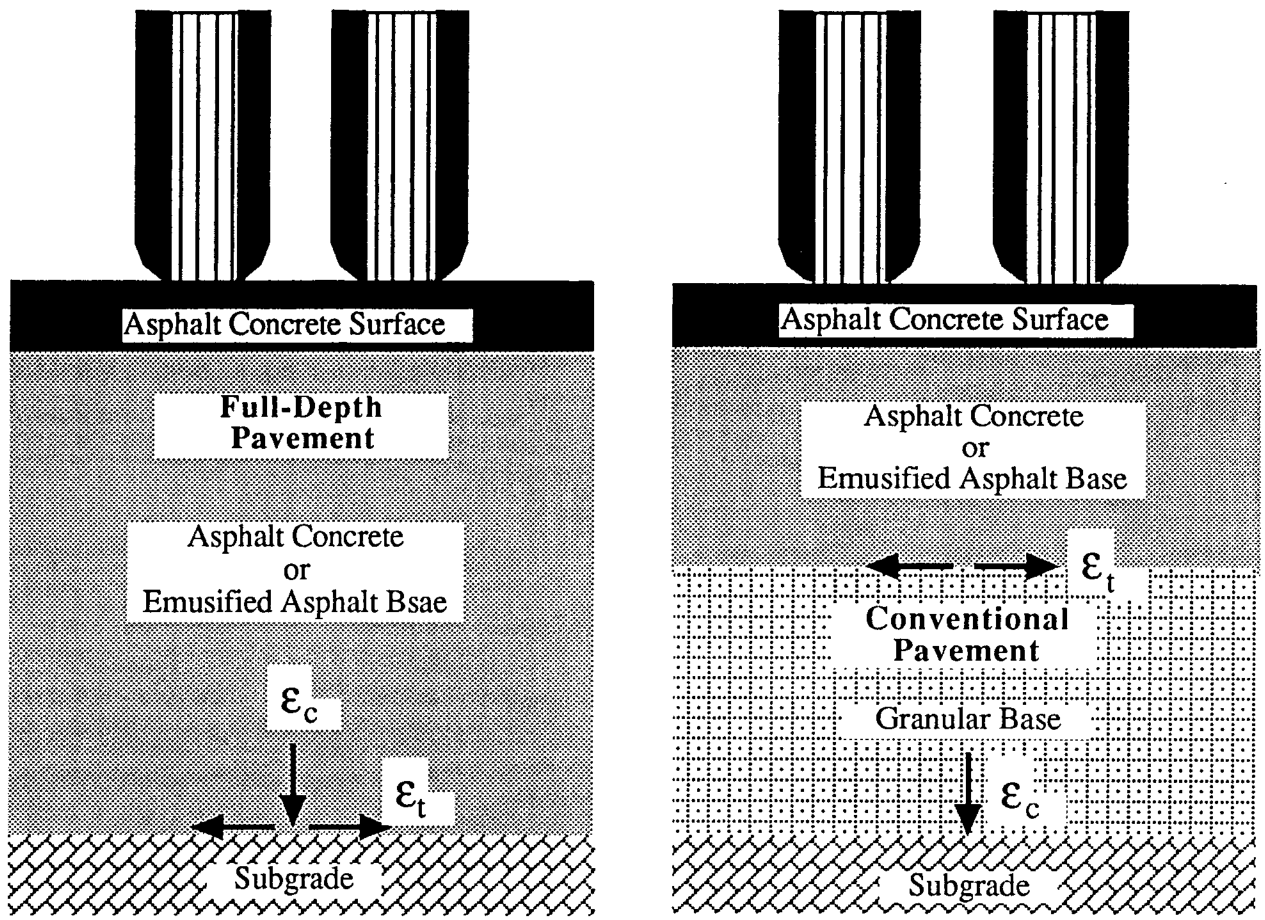


Fig. 6-1 Locations of strains considered in pavement design procedure

the pavement structure.

6.3.1 Failure Criteria

The failure criterion for fatigue cracking is expressed as

$$N_f = k_1 \left(\frac{1}{\varepsilon_t}\right)^{k_2} \left(\frac{1}{E}\right)^{k_3} \tag{6-1}$$

in which N_f is the allowable number of load repetitions to prevent fatigue cracking; ε_t is the tensile strain at the bottom of asphalt layer; E is the elastic modulus of asphalt layer; and k_1, k_2, and k_3 are constant determined from laboratory fatigue tests with k_1 modified to correlate with field performance observation. The Asphalt Institute used 0.0796, 3.291, and 0.854 for k_1, k_2, and k_3, respectively, in their mechanistic-empirical design procedures; the corresponding values used by Shell are 0.0685, 5.657, and 2.363.

The failure criterion for permanent deformation is expressed as

$$N_d = k_4 \left(\frac{1}{\varepsilon_c}\right)^{k_5} \tag{6-2}$$

in which N_d is the allowable number of load repetitions to limit the permanent deformation, ε_c is the compressive strain at the top of the subgrade layer; and k_4 and k_5 are constants determined from road tests or field performance. Value of k_4 and k_5 were suggested as 1.365 $*10^{-9}$ and 4.477 by the Asphalt institute, 6.15 $* 10^{-7}$ and 4.0 by Shell and 1.138 10^{-6} and 3.571 by the University of Nottingham.

6.4 Pavement Analysis

For the purpose of analysis, traffic was expressed in terms of repetitions of an Equivalent 80 KN (18-kips) Single Axle Load (ESAL) applied to the pavement on two sets of dual tires. The dual tire was approximated by two circular plates with radius of 3.785 in. spaced 13.5 in. center to center, corresponding to the 80 KN (18-kips) axle load and a 100 psi contact pressure. Since actual traffic data was not available, traffic class V, corresponding to ESAL of $3*10^6$, as suggested by Asphalt Institute (1981), was used for analysis. The traffic class V represents urban freeways, expressways, and other principal arterial highways as well as rural interstate and other principal arterial highways.

Conventional asphalt pavement with granular base layer was used for pavement analysis as shown in Fig. 6-1. Schematic representation of pavement system used in this study is shown in Fig. 6-2. As can be seen in Fig. 6-2, the thickness of asphalt layer varied from 2 to 10 inches. The granular base was assigned a thickness of 15 inch.

The resilient modulus for granular base and subgrade was assigned as 50 ksi and 20 ksi, respectively. The resilient moduli of both granular and subgrade layers were kept constant through the analysis.

Since the values of resilient modulus for rubber modified asphalt concrete (either wet process or dry process) have been determined at three different temperature. Therefore, the load periods in the KENLAYER analysis were divided into three temperature ranges, representing spring and fall, summer, and winter. Due to the

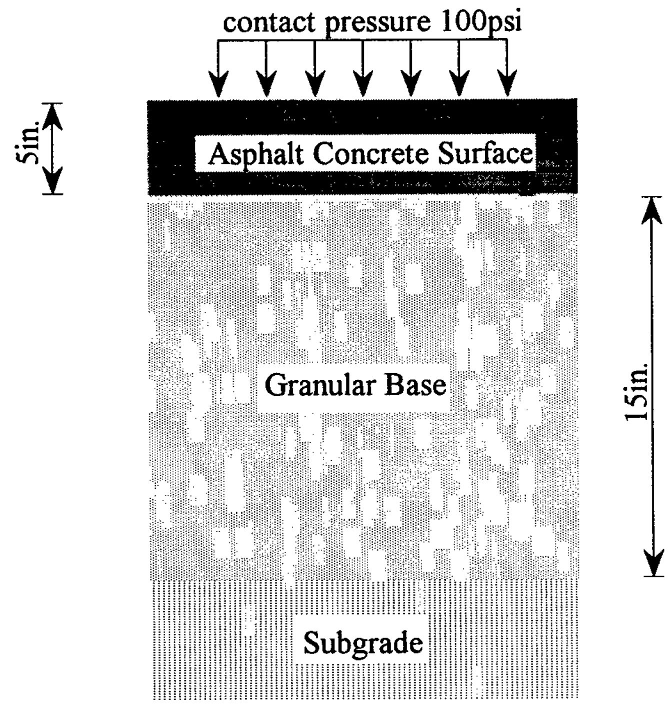


Fig. 6-2 Schematic representation of pavement system considered by KENLAYER

limitation of laboratory test data, only two types of asphalt mixtures were investigated in the KENLAYER analysis, namely, the control mix AC20, the wet process mixture AC5+15%WRF30. The values of resilient modulus used in the analysis are summarized in Table 6-1.

Table 6-1 Total resilient modulus for two mixes at three temperatures, ksi

Mix Designation	5^0 C	25^0 C	40^0 C
AC20	1392	725	223
AC5+15%WRF30	1029	207	166

Parameters k_1, k_2 and k_3 in fatigue cracking criterion equation (6-1) for the control mix are the values suggested by Shell, while those for AC5+15%WRF30 mix are modified by laboratory fatigue cracking test results. Recall the Table 5-14 in Chapter V. The regression equation for the fatigue test result for the AC5 based asphalt-rubber-mixture is

$$N_f = 1.58 \times 10^{-13} (1/\varepsilon_t)^{5.57} \tag{6-3}$$

comparing Eq. (6-1) with Eq. (6-3), it can be found that

$$k_1\left(\frac{1}{E}\right)^{k_3} = 1.58 \times 10^{-13} \tag{6-4}$$

$$k_2 = 5.57 \tag{6-5}$$

Letting k_1 be the same as that suggested by Shell, i.e., k_1=0.0685, then k_3 can be found by substituting the resilient modulus E=207000 psi into Eq. (6-4). k_3= 2.189. k_2 =5.57 as

Eq. (6-5). These parameters are summarized in Table 6-2. Parameters in rutting criterion equation (6-2) are the values suggested by Shell, with k_4 and k_5 being $6.15*10^{-7}$ and 4.0, respectively for subgrade.

Table 6-2 Fatigue cracking parameter for two mixtures used in KENLAYER analysis

Mix Designation	k_1	k_2	k_3
Dense-graded, Control Mix AC20	0.0685	5.657	2.363
Wet Process, Dense-graded, AC5+15%WRF30	0.0685	5.57	2.189

6.5 Results of Pavement Analysis

Assume a multiple-layer pavement shown as in Fig 6-2. The asphalt concrete layer has a thickness of 5 in., the granular base has a thickness of 15 in., and the subgrade has an infinite thickness. Predicted pavement lives of the two types of mixes are summarized in Table 6-3. As can be seen, the rubber modified asphalt concrete shows a longer fatigue life than the conventional HMA.

Table 6-3 Predicted pavement lives using KENLAYER program

Mix	Damage ratio	cycle life (yrs.)
Control Mix AC20	0.121	8.25
AC5+15%WRF30	0.094	10.59

6.6 Cost Analysis

A cost-effectiveness analysis is performed on the two types of pavements discussed above. Since the only difference between the two pavement types is the materials used in the surface layer, only the materials of the surface layer and their associated manufacture cost is analyzed here. Assume that a square yard ($0.836m^2$) of pavement is placed with an asphalt concrete thickness of 5in.

Table 6-4 Cost per square yard of asphalt concrete of Conventional HMA AC20 (Optimum binder content 5.7%, Unit Wt.=147.3pcf)

Component	Wt. (US Ton)	$/Ton	Cost ($/yd^2)
Asphalt Cement	0.01575	20	0.315
Aggregates	0.2605	14	3.647
Energy Cost	--	1.6	0.442
Mixing	--	4.5	1.243
Haul, Laydown and Compaction	--	8.0	2.21
Miscellaneous	--	1.0	0.276
Mark-up (15%)	--	4.3	1.220
Total			9.353

Table 6-5 Cost per square yard of asphalt-rubber concrete of AC5+15%WRF30 (Optimum binder content 7.2%, Unit Wt.=145.1pcf)

Component	Wt. (US Ton)	$/Ton	Cost ($/yd^2)
Asphalt Cement	0.01655	20	0.331
Rubber	0.00292	140	0.409
Aggregates	0.25091	14	3.513
Energy Cost	--	1.7	0.460
Blending and Reaction	--	6.0	1.622
Mixing	--	5.0	1.352
Haul, Laydown and Compaction	--	8.0	2.163
Miscellaneous	--	1.0	0.270
Mark-up (15%)	--	5.6	1.518
Total			11.638

Using the approximate costs given and the performance predictions presented previously, a measure of the cost-effectiveness of each material was obtained by calculating the equivalent uniform annual cost construction. This is defined as the cost which, if paid annually over the life of a given pavement, will be equivalent to the initial construction cost. The formula for equivalent uniform annual cost is:

$$A = \frac{P\left[i * (1+i)^n\right]}{\left[(1+i)^n - 1\right]} \tag{6-6}$$

where, A=equivalent uniform annual cost

P= initial construction cost

i = interest rate

n = pavement life in years

It is realized that the cost-effectiveness is also influenced by maintenance and user costs. However, very little data exist for estimating these costs for asphalt-rubber pavements. Consequently, only the initial construction cost was considered in evaluating cost effectiveness. Comparisons of the equivalent annual cost of the two mixtures evaluated are presented in Table 6-6. As we can see, the asphalt-rubber concrete shows a lower equivalent uniform annual cost than the conventional asphalt concrete.

Table 6-6 Equivalent Uniform Annual Costs for Conventional HMA AC20 and Asphalt-rubber Concrete (interest rate = 4%)

Mixes	Approximate In-place Cost ($/yd^2)	Predicted Service Life (years)	Equivalent Uniform Annual Cost ($/yd^2)
Control mix AC20	9.353	8.25	1.353
Asphalt-rubber concrete AC5+15%WRF30	11.638	10.59	1.370

CHAPTER VII

CONCLUSIONS

Based on the laboratory tests conducted on asphalt-rubber binder and asphalt-rubber-aggregate mixture, it can be concluded that rubber modified asphalt concrete mixture using wet process can be a viable asphalt paving material in the light of its superior fatigue performance, low-temperature thermal cracking resistance, and rutting resistance. However, the laboratory water sensitivity tests showed that CRM modified asphalt concrete may have water stripping problems. Specific conclusions are summarized as follows.

- The swelling test results indicated that the reaction temperature played a significant role in affecting the rate of weight increase of crumb rubber in asphalt cement. However, the projected maximum percent weight increase was around 50 percent in wet process and about 2 percent in dry process.
- Viscosity of asphalt-rubbers increased with an increase in CRM content, reaction period (especially, with coarse CRM), and a decrease in temperature.
- The slopes of viscosity versus temperature plot (temperature susceptibility) showed that all asphalt-rubbers with different CRM content were flatter than those

of unmodified asphalt cements, thus indicating that their viscosity was not as sensitive to temperature change as the unmodified asphalt cement, such as AC5, AC10, and AC20.

- All binders (either unmodified or CRM modified) used in this study showed non-Newtonian behavior (shear-thinning) at low range of shear rate for all testing temperatures. The unmodified asphalt showed mild shear-thinning behavior, while the asphalt-rubber with 20 percent CRM content showed pronounced shear-thinning behavior at low temperature range (225^0 F to 250^0 F).
- From the results of TFOT, weight losses of asphalt-rubbers with very fine CRMs (WRF 30 and Goodyear) were much lower than those of unmodified asphalts, while asphalt-rubbers showed higher ratio of viscosity increase after thin film oven aging.
- The unmodified asphalts showed reduced non-Newtonian behavior after thin film oven aging, while all CRM-modified asphalts showed increased non-Newtonian behavior at 275^0 F.
- The results from the dynamic shear rheometer test indicated that dynamic viscosities of short-term aged binders (either unmodified or CRM-modified) were higher than those of unaged binders. Also, an increase in both test oscillatory frequency and temperature resulted in a decrease in the dynamic viscosity. Both complex shear modulus and phase angle increased with short-term aging for both unmodified asphalt and CRM-modified asphalt. Increase in complex shear modulus indicates the stiffening effects; however, increase in phase angle

represents more viscous behavior. Both effects are more pronounced for asphalt-rubber than unmodified asphalt.

- $G^*/\sin\delta$ of both unaged and short-term aged asphalt-rubber binder is greater than the minimum values of 1.00 kPa and 2.2kPa, respectively, required by the Performance Graded Asphalt Binder Specification to possess the potential of resistance to rutting at a high temperature of 60°C.
- The results from Marshall mix design indicated that rubber modified mix (either wet or generic dry process) showed smaller stability, higher flow, and higher optimum binder content. Especially, stability for generic dry process (either dense or gap gradation) was about half of control mix and flow was about twice.
- The general observation of the indirect tensile test results is that the mixture prepared with CRM modified binder tend to exhibit lower tensile strength compared to the control mix. Either the control mix or the CRM modified mixes tend to increase the indirect tensile strength due to the aging effect except for the continuously blended mix which shows a decreased indirect tensile strength due to the short-term aging effect. There is no significant discernable difference in the aging behavior between the control mix and the CRM modified mixes.
- The mixes prepared with wet process and the continuous blending process show higher resilient modulus than the dry mixes which have too low resilient modulus values. The AC10 based asphalt mixture seems to possess less temperature susceptibility than the others.
- The aging behavior for the CRM modified mix does not seem to differ drastically

from the conventional control mix. Both short-term and long-term aging increase the resilient modulus of control mix and the CRM modified mixes due to the hardening process.

- The results from TSRST indicated that the fracture temperature of rubber modified specimens was lower than that of control mixed specimen. Except for the mixes prepared with the continuous blending technology and the dry process, all the other CRM modified mixes showed greater tensile stress at fracture and significant lower fracture temperature.
- The results of creep test and loaded wheel track test indicated that the rubber modified asphalt mixtures have a higher rutting resistance than the control asphalt mixture.
- The results of water stripping tests showed that none of the mixes in this study exhibits a satisfactory Tensile Strength Ratio (TSR) of over 0.80. The CRM modified asphalt mixes are particularly susceptible to the moisture induced damage by the AASHTO test method. This problem is worthy of further investigation.
- The results from strain-controlled fatigue test for four mixes indicated that the fatigue life of the wet process rubber modified mixture (dense-graded AC5+15%WRF30) was longer than that of the control mix. The fatigue life of the dry process rubber modified mixture was the shortest.
- Based on the resilient modulus test, TSRST test, and indirect tensile test, we found that the AC5 based, wet process, dense-graded asphalt-rubber mixture

(AC5+15%WRF30, dense graded) exhibited the best low temperature thermal cracking resistance, followed by dense-graded AC10+10%WRF30, continuous blending dense-graded AC5+10%GY, control mix AC20, gap-graded AC5+15%WRF30, and dry process with 2%CRM.

- Based on the incremental creep test, loaded wheel track test, and indirect tensile test, we found that the AC10 based, wet process, dense-graded asphalt-rubber mixture (AC10+10%WRF30) exhibited the best rutting resistance, followed by continuous blending dense-graded AC5+10%GY, dense-graded AC5+15%WRF30, control mix AC20, and gap-graded AC5+15%WRF30.
- From the performance prediction of pavements using the computer program KENLAYER, pavement surface layer with rubber modified asphalt mixture by wet process showed a longer predicted life expectancy compared to the conventional asphalt mix.
- From the life cycle cost analysis, the conventional asphalt concrete is a little more economical in terms of the equivalent uniform annual costs compared to the rubber modified asphalt concrete.
- In summary, the rubber modified asphalt mixture may exhibit a better performance than the conventional asphalt mixture. The AC10 based asphalt-rubber mixture may be recommended to be paved as a field performance test.
- The continuous blending technology seems to be an interesting topic because of its eliminated asphalt-rubber reaction period. The lower optimum binder content obtained in this study also justifies its economic advantage. However performance

test results have not yet shown that the continuous blending produced mixture is better than the other asphalt-rubber mixtures. More tsets can be conducted on continuous blending technology to investigate its practicableness.

- Due to the limitation of laboratory test and field performance data, the prediction of service life and the life cycle cost analysis need more investigation.

REFERENCES

1 Al-Abdul-Wahhab, H. and Al-Amri, G. (1991) "Laboratory Evaluation of Reclaimed Rubber Asphalt Concrete Mixes," ASCE Journal of Materials in Civil Engineering, Vol. 3, No. 3, pp. 189-203

2 Ahmed, Imtiaz and Lovell, C. W. (1992) "Use of Rubber Tires in Highway Construction," Utilization of Waste Material in Civil Engineering Construction, Proceedings of Material Engineering, ASCE, pp. 166-181

3 Anderson, David A. and Kennedy, Thomas W. (1993) "Development of SHRP Binder Specification," Association of Asphalt Paving Technologist (AAPT), pp. 481-507

4 Anderton, Gary L. "Evaluation of Asphalt Rubber Binders in Porous Friction Courses" US Army Corps of Engineers, Interim Report CPAR-GL-92-1, May, 1992

5 Anderson, David A., Christensen, D. W., Dongre, R., Sharma, M. G., Runt, J., Jordhal, P. "Asphalt Behavior at Low Service Temperatures," U.S. Department of Transportation, Federal Highway Administration, FHWA-RD-88-078, Mar., 1990

6 Bahia, Hussain U., Anderson, David A. and Christensen, Donald W. (1992) "The Bending Beam Rheometer; A Simple Device for Measuring Low-temperature Rheology of Asphalt Binders," AAPT, pp. 117-153

7 Blumenthal, Michael and Zelibor, Joseph L. (1992) "Scrap Tires in Rubber-Modified Asphalt Pavement and Civil Engineering Applications," Utilization of Waste Material in Civil Engineering Construction, Proceedings of Material Engineering, ASCE, pp. 182-192

8 Charania, Equbalai, Cano, Joe O and Schnormeier, Russel H. (1991) "Twenty-year study of Asphalt-rubber Pavements in Phoenix, Arizona," TRR 1307, pp. 29-38

9 Doty, Robert N. (1989) "Flexible Pavement Rehabilitation Using Asphalt-rubber Combinations: Progress Report," TRR 1196, pp. 212-223

10 Eaton, Robert A., Roberts, Richard J. and Blackburn Robert R. (1991) "Use of Scrap Rubber in Asphalt Pavement Surfaces," Special Report 91-27, US Army Corps of Engineers, Cold Regions & Engineering Laboratory

11 Estakhri, Cindy K., Button, Joe W. and Fernando, Emmanuel G. (1992) "Use, Availability, and Cost-effectiveness of Asphalt-rubber in Texas," TRR 1339, pp. 30-37

12 Green, E. L. and Tolonen, William J. (1977) "The Chemical and Physical Properties of Asphalt-rubber Mixtures, Part I - Basic Material Behavior," FHWA-AZ-HPR14-162, Arizona Department of Transportation

13 Heitzman, Michael (1992) "State of the Practice-Design and Construction of Asphalt Paving Materials with Crumb Rubber Modifier," FHWA-SA-92-022, Federal HighWay Administration

14 Hoyt, Denise M., Lytton, Robert L. and Robert, Freddy L. (1989) "Performance Prediction and Cost-effectiveness of Asphalt-rubber Concrete in Airport Pavements," TRR 1207, pp. 88-99

15 Huang, Yang H. (1993) "Pavement Analysis and Design," Prentice Hall

16 Roberts, F. L., et al. (1996) "Hot Mix Asphalt Materials, Mixture Design and Construction", Second Edition, NAPA Rearch and Education Foundation

17 Huff, B. J. and Vallerga, B. A. (1981) "Characteristics and Performance of Asphalt-rubber Material Containing a Blend of Reclaim and Crumb Rubber," TRR 821, pp.29-36

18 Krutz, Neil C. and Stroup-Gardiner, Mary (1992) "Permanent Deformation Characteristics of Recycled Tire Rubber-modified and Unmodified Asphalt Concrete Mixtures," TRR 1339, pp. 38-44

19 Lalwani, Steve, Abushihada, Adnan and Halasa, Adel (1982) "Reclaimed rubber-asphalt Blends Measurement of Rheological Properties to Assess Toughness, Resiliency, Consistency, and Temperature Sensitivity," AAPT, pp. 562-579

20 Lee, Seung W. and Fishman, K. L. (1993) "Waste Products as Highway Materials in Flexible Pavement System," ASCE Journal of Transportation Engineering, Vol. 119, No. 3, pp. 433-449

21 Lundy, James R., Hicks, R. G. and Richardson, Emory (1988) "Evaluation of Rubber-modified Asphalt Performance - Mt. St. Helens Project," AAPT, pp. 573-594

22 Maupin Jr., G. W. (1992) "Virginia's Experimentation with Asphalt Rubber Concrete," TRR 1339, pp. 9-15

23 McDonald, Charles H. (1966) "A New Patching Material for Pavement Failures," HRR, Vol. 146, pp. 1-16

24 Monismith, C. L. and Tayebali, A. A. (1988) "Permanent Deformation (rutting) Considerations in Asphalt Concrete Pavement Sections," AAPT, pp. 414-463

25 Brosseaud, Yves et al. (1993) "Use of LPC Wheel-track Rutting Testerto Select Asphalt Pavements Resistant to Rutting," TRR 1384, pp. 59-68

26 Corte, J. F. Et al. (1994) "Investigation of Rutting of Asphalt Surface Layers:Influence of Binder and Axle Loading Configuration", TRR 1436, pp. 28-37

27 Morris, Gene R. (1989) "Asphalt-rubber Concrete - A New Horizon," Proceedings of Creative Application of Material Engr, ASCE Material Engineering Division, pp. 7-15

28 Fager, Glenn A. (1994) "Use of Rubber in asphalt pavements: Kansas Experience," TRR 1436, pp. 88-97

29 Morris, Gene R. and McDonald, Charles H. (1976) "Asphalt-rubber Stress-absorbing Membranes: Field Performance and State of the Art," TRR 595, pp. 52-58

30 Oliver, John W. H. (1981) "Modification of Paving Asphalt by Digestion with Scrap Tire," TRR 821, pp. 37-44

31 Page, Gale C., Ruth, Byron E. and West, Randy C. (1992) "Florida's Approach Using Ground Tire Rubber in Asphalt Concrete Mixtures," TRR 1339, pp. 16-22

32 Page, Gale C. (1992) "Florida's Initial Experience Utilizing Ground Tire Rubber in Asphalt Concrete Mixes," AAPT, pp. 446-472

33 Robert, Freddy L., Lytton, Robert L. and Hoyt, Denise (1986) "Criteria for Asphalt-rubber Concrete in Civil Airport Pavements: Mixture Design (I)", DOT/FAA/PM-86/39, Vol. I, Texas Transportation Institute, Texas A & M University, College station, Texas

34 Ruth, Baron E. (1990) "Documentation of Open-graded, Asphalt-rubber Friction Course Demonstration Project on Interstate 95, St. Johns Country," Department of Civil Engineering, University of Florida, Gainesville, Florida

35 Salter, R. J. and Mat, J. (1990) "Some Effects of Rubber Additives on Asphalt Mixes," TRR 1269, pp. 79-86

36 Schnormeimer, Rusell Howard (1986) "Fifteen-year Pavement Condition History of Asphalt-rubber Membranes in Phoenix, Arizona," TRR 1096, pp. 62-67

37 Shuler, T. S., Pavlovich, R. D. and Epps, J. A. (1985) "Field Performance of Rubber-modified Asphalt Paving Materials," TRR 1034, pp. 96-102

38 Smith, Harry A. (1992) "Performance Characteristics of Open-graded Friction Courses," NCHRP Synthesis 180

39 Takallou, H. Barry and Sainton, Alain (1992) "Advances in Technology of Asphalt Paving Material Containing Used Tire Rubber," TRR 1339, pp. 23-29

40 Takallou, H. Barry and M. B. (1991) "Benefits of Recycling Waste Tires in Rubber Asphalt Paving," TRR 1310, pp. 87-92

41 Takallou, H. B., Hicks, R. G., Esch, D. C. and Vinson, T. S. (1989) "Performance of Rubber-modified Asphalt Pavements in Cold Regions," Cold Engineering, Proceedings of 5th International Conference of ASCE, pp. 81-91

42 Takallou, H. B. and Hicks, R. G. (1988) "Development of Improved Mix and Construction Guidelines for Rubber-modified Asphalt Pavements," TRR 1171, pp. 113-120

43 Takallou, H. B., Hicks, R. G. and Esch, D. C. (1986) "Effect of Mix Ingredients on the Behavior of Rubber-modified Asphalt Mixes," TRR 1096, pp. 68-80

44 Valkering, C. P., Lancon, D. J. L., deHilster, E. and Stoker, D. A. (1990) "Rutting Resistance of Asphalt Mixes Containing Non-conventional and Polymer-modified Binders," AAPT, pp. 590-609

45 Yoder, E. J. and Witczak, M. W. (1975) "Principles of Pavement Design," John Wiley & Sons, Inc.

46 "Calibrated Mechanistic Structural Analysis Procedures for Pavements," Vol. I & II, NCHRP 1-26, University of Illinois at Urbana-Champaign Construction Technology Laboratories, 1990

47 "Computer Program DAMA User's Manual": Pavement Structural Analysis Using Multi-Layered Elastic Theory, The Asphalt Institute, 1990

48 "Construction and Material Specifications," State of Ohio Department of Transportation, Columbus, Ohio, Jan., 1991

49 "Crumb Rubber Modifier: Design Procedures and Construction Practice," FHWA-SA-93-007, U. S. Department of Transportation, Federal Highway Administration, 1993

50 "Mix Design Methods for Asphalt Concrete and Other Hot-mix types," Asphalt Institute MS-2, 1993

51 "National Seminar on Asphalt Rubber," Federal Highway Administration Demonstration Projects Division, Kansas City, Missouri, 1988

52 "National Seminar on Asphalt Rubber," Federal Highway Administration Demonstration Projects Division, San Antonio, Texas, 1981

53 "Research and Development of The Asphalt Institute's Thickness Design Manual (MS-1)," Ninth Edition, The Asphalt Institute Research Report No. 82-2, 1982

54 "Summary Report on Fatigue Response of Asphalt Mixtures," SHRP-A/IR-90-011, Strategic Highway Research Program, National Research Council, Feb., 1990

55 "Summary Report on Permanent Deformation in Asphalt Concrete," SHRP-A/IR-91-104, Strategic Highway Research Program, National Research Council, 1991

56 "The Asphalt Handbook," The Asphalt Institute, MS-4, 1989

57 "Thickness design Asphalt Pavements for Highways & Streets," The Asphalt Institute, MS-1, Feb., 1991

58 "Use, Availability and Cost-effectiveness of Asphalt Rubber in Texas", Research Report 1902-1F, Texas Transportation Institute, Sep. 1990

59 "Criteria for Asphalt-rubber Concrete in Civil Airport Pavements, Vol. II--Evaluation of Asphalt-rubber Concrete", Final Report, Texas Transportation Institute, Mar. 1987

60 Kenneth Troy, Peter E. Sebaaly, and Jon A. Epps (1996) "Evaluation System for Crumb Rubber Modified Binders and Mixtures," TRR, 1530, pp. 3-10.

61 G. W. Maupin, Jr. (1996) "Hot Mix Asphalt Rubber Application in Virginia,"

TRR, 1530, pp. 18-24.

62 Gary V. Gowda, Kevin D. Hall, and Robert P. Elliott (1996) "Arkansas Experience with Crumb Rubber Modified Mixes Using Marshall and Strategic Highway Research Program Level I Design Methods," TRR, 1530, pp25-33.

63 Mary Stroup-Gardiner, Bruce Chadbourn, and David E. Newcomb (1996) "Babbitt, Minnesota: Case Study of Pretreated Crumb Rubber Modified Asphalt Concrete," TRR, 1530, pp. 34-42.

64 Ludo Zanzotto and Gerhard J. Kennepohl (1996) " Development of Rubber and Asphalt Binders by Depolymerization and devulcanization of Scrap Tires in Asphalt," TRR, 1530, pp. 51-58.

65 Raghu Ram Madpati, K. Wayne Lee, Francis J. Manning, and Colin A. Franco (1996), "Feasibility of Crumb Rubber Use for Asphalt Pavement Construction," TRR, 1530, pp.64-71.

66 Ihab H. Hafez and Matthew W. Witczak (1995), " Comparison of Marshall and Superpave Level I Mix Design for Asphalt Mixes," TRR, 1492, pp. 161-175.