Polytechnic University

Department of Civil & Environmental Engineering

Project
Laboratory Evaluation of Asphalt Concrete Mixtures Using Waste Tires

Final Report 423-087-2F

Evaluation and Performance Based Mix Design of Rubber Modified Mixtures

Prof. Dimitrios G. Goulias
(Principal Investigator)

Al- Hosain M Ali
(Research Fellow)

February 1997

LIST OF CONTENTS

Page

CHAPTER 5. MIXTURE BEHAVIOR AND ENHANCEMENT OF MARSHALL MIX DESIGN METHOD

CHAPTER 6. ASPHALT RUBBER MIXTURE EVALUATION WITH INDIRECT TENSILE TEST

CHAPTER 7. FATIGUE AND CREEP CHARACTERIZATION OF ASPHALT-RUBBER MIXTURES

CHAPTER 8. MIXTURE DESIGN METHODOLOGY

CHAPTER 9. CONCLUSIONS AND RECOMMENDATIONS

LIST OF FIGURES

Page

Page

Page

Page

LIST OF TABLES

Page

Page

Page

CHAPTER 1. INTRODUCTION

INTRODUCTION

The use of tire rubber in asphalt pavement materials is being investigated since the 1960's. Main objective of highway engineers was to enhance the properties and performance of conventional bituminous materials and mixtures with the use of rubber from scrap tires. Charles McDonald started his work in the early 1960's developing a highly elastic maintenance surface patch material by using crumb rubber. Asphalt-rubber modified mixtures were used in Sweden since the early 1970's. In 1968, Arizona Department of Transportation (ADOT) placed its first stress absorbing membrane, (SAM), using an asphalt rubber binder, followed by the placement of a stress absorbing membrane interlayer in 1972, (SAMI), and a hot mix asphalt open graded friction course with an asphalt rubber binder in 1975. The environmental concerns related to the disposal of an estimated 240 million passenger vehicle tires and 40 millions truck tires discarded each year in the U.S. and the 1991 ISTEA mandate on the tire rubber use in federally founded projects generated a significant momentum and interest in the investigation of rubber modified materials.

Today, several State D.O.T.s are being investigating the use of tire rubber with local conventional materials, including Alaska, California, Kansas, Massachusetts, New Jersey, New Mexico, New York, Oklahoma, Rhode Island, South Dakota, Tennessee, Texas, Utah and Washington. The two basic methods of adding crumb rubber to asphalt paving materials are the wet and the dry processes. In the wet process the finely ground rubber is added into the hot asphalt cement at high temperature to create a rubberized asphalt binder which may then be mixed with the aggregate. Depending on the mixture and material characteristics the rubber content may range between 18 to 25% by the weight of the binder. The dry method, developed in the late 1960's in Sweden and patented under the trade name of "Rubit" and "PlusRide" in the U.S., involves the use of relatively large rubber particles (1/16 in to 1/4 in) to replace a portion of the conventional aggregate, typically 3 to 4 percent by the weight of the mixture.

A variety of applications are being identified for the use of rubber modified materials. Examples include: asphalt rubber concrete, where asphalt cement and finely ground tire rubber particles are mixed to create a rubberized asphalt binder which is then mixes with the aggregate; rubber-filled systems, in which rubber from granulated tires is blended with aggregate and then with the asphalt cement, (such as the Plus Ride and Chunk Rubber Asphalt Concrete, CRAC, mixtures); rubber-aggregate blended with conventional aggregate for sub-base layer material; use of asphalt rubber binder for seal coats, chip seals, surface treatment, interlayers.

Several of the ongoing investigations identified potential benefits from the use of these materials, including improvements in material properties and performance. For example, when the rubber particles substitute portion of the conventional aggregate, they act as an elastic aggregate that flex on the pavement surface under traffic loads. Results of several pilot projects shown that the inclusion of rubber into asphalt mixtures improved physical characteristics such as elasticity, flexibility, rebound, and aging properties, increased fatigue resistance, improved retardation of reflective cracking, reduced rutting potential, improved skid resistance, and increased durability. As a consequence, several DOTs are using routinely rubber modified and rubber filled materials since they find them advantageous for specific applications. For example, Arizona is being using asphalt rubber as a crack sealant since the 1970s. California uses routinely asphalt rubber on approximately 10% of the pavement projects. Connecticut is using asphalt rubber for crack sealant and overlays. Texas is using routinely this material for crack sealant.

However in some instances, and due to several shortcomings related to the high initial material preparation cost, mixing and production barriers, questionable results regarding the long term material performance, lack of appropriate thickness equivalency ratio between conventional and asphalt rubber materials, recyclability of asphalt containing rubber, and other, the use of tire rubber materials is not always being recommended. Today, several projects are initiated by highway agencies for addressing these issues and identifying possible applications. One of the major problems is being associated with the transferability of asphalt rubber technology without appropriately considering the effects of the variety of conventional materials on mixture behavior and performance. Typically, the design of these mixtures is being adapted to the physical properties of the conventional materials by using the empirical Marshall mixture design and without considering fundamental mixture behavior and performance.

As expected, type and origin of asphalt cement, type and characteristics of aggregate affect rubber modified mixture properties. Thus, asphalt-rubber compatibility, material properties and behavior, mix design methods, and field performance should be considered in determining the best and most economical rubber modified mixture by using local conventional materials. The rubber modified mixture should be able to withstand specific range of loading and environmental conditions depending on the region that is used, and without excessive permanent deformation, fatigue, and cracking leading to premature deterioration. Current mix design methods should be revised so that mixture properties and/or behavior are evaluated through laboratory tests, and directly linked to mixture performance. Use of design selection criteria related to the most common modes of failure for asphalt mixtures, such as rutting, fatigue cracking, and low temperature thermal cracking have to developed and used for identifying the "best mixture," in terms of performance, for the specific local materials and loading conditions.

RESEARCH OBJECTIVES

New Jersey Department of Transportation is being investigating the use of rubber modified materials over the last years with the design and use of dense and gap graded mixtures, and in some cases the incorporation of RAP materials, in selected projects. While the short term field performance of these materials is being satisfactory, their long term performance is unknown. These mixtures were designed with the traditional Marshall mixture design method, and thus it was not considered design criteria related to mixture behavior and performance into mixture selection. The main objective of this study is the development of a mixture design methodology for rubber modified materials that considers mixture behavior and performance. In order to achieve this objective a laboratory investigation able to evaluate mixture properties that can be related to mixture performance, (in terms of rutting, low temperature cracking, and fatigue), and simulating the actual field loading conditions that the material is being exposed to, was conducted. The possibility of coupling the traditional Marshall mix design method with parameters related to mixture behavior and performance was investigated since this technique is being used over the years by the agency, and the necessary testing apparatus is available to both the agency and material laboratories. The SHRP SUPERPAVE mix design methodology was reviewed and considered in this study for the development of an integrated performance based design procedure. However, its applicability and use on routine bases was not considered at this time since it requires specific equipment with ongoing evaluation for its repeatability and precision. Finally, for the conduct of this investigation materials and mixtures used by NJDOT in rubber modified paving projects were used.

The steps undertaken during the first phase of the study for achieving these objectives follow.

- conduct an extensive literature review on past and current experience of States with rubber modified paving materials. Both laboratory and short and long term field performance results were focused;

- contact NJDOT material suppliers and collect conventional materials and crumb rubber used in NJDOT projects;

- evaluate material properties including conventional and rubber modified binder characteristics, crumb rubber and aggregate properties. The evaluation of the rubber modified binder included an evaluation of the reaction curve and the effects of aging on binder properties through the use of factorial experiments;

- identify measurable mixture behavior parameters able to be coupled with the Marshall mix design methodology. In this step rubber modified mixtures were designed and examined with the Marshall mix design method and parameters such as stiffness, toughness, and absorbed energy were evaluated. Comparison with conventional mixtures were conducted as well for relevant conclusions;

- identify and select a reliable test to be used in the mixture design methodology able to evaluate mixture properties that can be related to mixture performance, and simulating the actual field loading conditions that the material is exposed to. Both static and dynamic-repeated load tests were considered in this step;

- design and perform laboratory factorial experiments for determining the best testing conditions, in terms of testing temperature and loading characteristics;

- identify methods for evaluating the long term performance of asphalt rubber mixtures. Prediction models in terms of rutting, low temperature cracking, and fatigue, to be used in the development of the integrated mixture design methodology were reviewed. The distress models used in the SUPERPAVE mix design were considered as well;

- integrate a data acquisition system to the conventional Marshall apparatus and the repeated-load testing machine for continuous monitoring of testing parameters and automation of testing;

For meeting the objectives of this research study the following steps are undertaken during the second phase of this research:

- extent factorial experiments with additional NJDOT asphalt rubber mixtures;

- complete and finalize the integrated mixture design methodology and identify the criteria and steps of the methodology;

- present the results from the coupling of Marshall mix design with mixture behavior parameters;

- develop guidelines with easy-to-use flow charts for the integrated mix design methodology;

ORGANIZATION OF THE REPORT

This report presents the results and analysis undertaken during the first and the second phases of the study. The first chapter presents a brief historical review on the status and experience of rubber modified materials in the U.S., along with the research objectives and a description of the organization of this report. Chapter 2 provides the results from the extensive literature review on these materials. The review includes information and results from studies examining and addressing: crumb rubber processing and characteristics; asphalt-rubber binder preparation and evaluation techniques; applications of rubber modified mixtures; factors affecting asphalt-rubber mixture characteristics, mixture and sample preparation characteristics; and design and field performance of rubber modified mixtures. In addition, a review of the available laboratory testing methods for asphalt mixtures was conducted. The review included both static and dynamic tests, along with past experience regarding their repeatability, ability to evaluate mixture properties and simulate actual field conditions.

The third chapter, Chapter 3, presents the laboratory study undertaken for achieving the study objectives. Details on material characteristics, and the factorial experiments for binder and mixture evaluation according to Marshall, static and repeated load indirect tensile, diametrical fatigue, and repeated-load creep testing for conventional and rubber modified mixtures are provided along with a description of the testing conditions. Chapter 4, presents the results of the asphalt rubber binder evaluation and the Marshall results along with a comparison of rubber modified and conventional mixtures. Chapter 5, presents the analysis undertaken for coupling Marshall with measures of mixture behavior related to mixture performance. Results on absorbed energy, stiffness, and toughness of the modified and conventional mixtures are being reported. Chapter 6, presents the results from the static and repeated load indirect tensile tests. Instantaneous and total resilient modulus were evaluated for both asphalt-rubber (the wet process) and rubber-filled (the PlusrRide II) mixtures at different temperature and stress levels. These results were used in mixture performance evaluation and subsequently in the design methodology development. Chapter 7, presents the fatigue and the creep testing results. These results are used with performance models to develop the integrated mix design methodology. Chapter 8, presents the integrated mix design methodology. The results from the analytical analysis for pavement structure design, stress-strain analysis, and pavement performance are presented. Pavement performance models (for excessive subgrade deformation, fatigue, rutting, thermal cracking and moisture damage) are identified and used. The analytical and laboratory results are used in the integrated mix design development. Finally, Chapter 9 presents the summary, conclusions and recommendations.

CHAPTER 2. BACKGROUND

INTRODUCTION

Over the years the use of tire rubber in asphalt binder and mixtures is being examined for specific applications since the incorporation of rubber could produce some degree of binder and mixture enhancement. In recent years, and with the increasing problem of disposing tires and the mandatory use of tire rubber in federally funded projects by the 1991 ISTEA, a significant effort was undertaken nationwide for investigating and evaluating the cost effectiveness, properties and performance of rubber modified mixtures. In this chapter, a comprehensive literature review on past and current applications and experience with rubber modified paving materials is being presented. Crumb rubber characteristics, process techniques, and interaction with asphalt binder and mixtures are presented. In addition, this chapter presents available asphalt mixture evaluation techniques and specimen characteristics, and potential modifications for asphalt rubber mixtures. Such review will provide the input for selecting the most appropriate and suitable testing technique for asphalt-rubber modified mixtures. Finally the effort undertaken by several DOTs for designing asphalt-rubber mixtures are discussed along with a description of the factors affecting mixture characteristics.

CRUMB RUBBER CHARACTERISTICS

Tire rubber is primarily a composite of a number of blends of natural rubber, synthetic rubber, and carbon black. Natural rubber provides the elastic properties while the synthetic rubber improves the thermal stability of the compound. Depending on the composition different types of tires rubber may be obtained. Crumb rubber is produced from scrap tires processed by different methods providing alternative sizes, surface area characteristics, particle shapes, and gradation. These parameters affect the rubber asphalt interaction when a modified asphalt binder is to be designed.

Typically, there are four processing technique for producing crumb rubber (Heitzman 1992). The Crackermill process is the most common method. It tears apart scrap tire rubber by passing the material between rotating corrugated steel drums to reduce its particle size. The Granulator process sheers apart the scrap tire rubber, cutting the rubber with revolving steel plates that pass at close tolerance. The Micro-mill process reduces the crumb rubber to a very ground particles by first mixing the rubber with water, and then forcing the slurry rubber between rotating abrasive discs to produce finer crumb rubber. These three production methods take place at ambient temperature. At the opposite, the cryogenic process involves "freezing" the scrap tire so as to crush in small particles the brittle rubber.

Each one of these method produce crumb rubber particles with specific characteristics. The Crackermill process produces irregularity shaped particles with large surface area. The Granulator process produces cubical, uniformly shaped particle with low surface area and the Micro-miller process produces very fine ground crumb rubber. The Cryogenic process is too costly and not frequently used. The desired surface area characteristics, size and shape of the crumb rubber for a particular project will determine the best method for processing of scrap tire.

Crumb Rubber: Testing, Suggested Specifications and Quality Control Guidelines

Suggested material specifications for crumb rubber to be used as a modifier in asphalt binder and mixtures were presented in a recent FHWA report (Heitzman 1992). Some of the applicable standards for this material include: the AASHTO *Standards* M17 for mineral filler for bituminous paving mixtures, T2 sampling aggregate techniques, T27 sieve analysis of fine and coarse aggregate, and T225 total moisture content of aggregate by drying; and the *ASTM Standards* D242 mineral filler for bituminous paving mixture, C136 sieve analysis, D297 chemical analysis of CRM (natural rubber content). The suggested FHWA specifications identify that crumb rubber from any combination of passenger and truck tires may be used and it should be free of steel, fabric or other deleterious substances. The fiber content shall be less than 0.5 % (by weight), and the moisture content less than 0.75 % by sample weight. The crumb rubber gradation shall meet one of the gradations presented in Table 2.1. The specific gravity of the crumb rubber shall be 1.15 (± 0.05), and the crumb rubber composition shall meet the following limits:

- Natural rubber	15-30 %
- Carbon black	25-38 %
- Ash	8 max
- Acetone extract	10-18 %
- Rubber hydrocarbon	40-50 %

The crumb rubber gradation is to be tested according to ASTM C136 using a 50 gram sample, and the fiber content is to be determined by weighing the fiber particles that are formed during the gradation evaluation. The metal content is to be determined by passing a magnet through a 50 gram sample, while the moisture content, to be determined according to AASHTO T255, should take place in a controlled temperature oven at 60°C (140°F) and with a 50 gram sample. The mineral contaminant content is to be determined by a saline float separation. A 50 gram sample is stirred in one liter glass beaker filled with saline solution, one part of table salt into three parts of distilled water, and then the sample is allowed to pose for 30 minutes. The mineral contaminant is then the material that does not float on the top of the beaker.

CRUMB RUBBER IN ASPHALT MIXTURES

The two methods of using crumb rubber with asphalt paving materials are the "wet" and the "dry" process. The first one considers mixing of the rubber with the conventional binder prior of adding the aggregate into the mixture. The second method, dry process, involves adding the crumb rubber directly with the heated aggregate prior to the inclusion of the asphalt cement into the mixture.

Table 2.1 Suggested FHWA Crumb Rubber Gradations (Heitzman 1992)

Sieve Size	CRM-I	CRM-II	CRM-III	CRM-IV	CRM-V	CRM-VI
	percent of weight passing					
6.3 mm (1/4")	100					
4.75 mm (No. 4)	75-100	100				
2.36 mm (No. 8)	35-50	80-100	100			
161.18 mm(No. 16)	20-30	40-70	80-100	100		
600 μm (No. 30)		0-20	40-60	70-100	100	
300 μm (No. 50)			0-20	20-40	40-60	100
150 μm (No. 100)						50-80

The dry process was developed in late 1960's in Sweden and marketed under the patented name "Rubit," (Takallou 1980). The technology was introduced in the United States in the 1970's as Plus Ride. This process typically uses 3 % by weight granulated coarse and fine rubber to replace some of the conventional aggregate in the mixture. Some of the advantages/effects that are present in the literature include: reflective and thermal pavement cracking may greatly be reduced; skid resistance may increase; structural characteristics of the asphalt mixture are improved; and suppression of pavement tire noise is expected. No special equipment is needed for this process and both batch and drum plants, with little modification, may be used to produce crumb rubber modified asphalt mixtures with this process. In The dry process some reaction between the crumb rubber and asphalt cement during mixing and compaction time are taken place.

For the wet process the asphalt cement and the crumb rubber are blended together, providing a new modified binder through the interaction of the materials. This reaction between the conventional asphalt binder and the crumb rubber is influenced by several factors including: the bending temperature and time, the type and amount of mechanical mixing energy, the size and texture of crumb rubber, and the aromatic content of the asphalt cement. When the rubber is mixed with the asphalt cement, the rubber particles swell (react) causing the viscosity to increase. However, when the heat is maintained for a prolonged period of time the rubber may melt and break down, with a drop in viscosity. Maupin (1992) investigated the asphalt rubber interaction by examining the reaction curve, (i.e., viscosity versus time relationship), for rubber contents of 5, 10, and 15 percent with both coarse and fine crumb rubber. In this study a portable Haake viscometer was used. The rubber was mixed with the asphalt cement and maintained at 177°C (350 °F), while viscosity measurements were taken at regular intervals for approximately 24 hours. The reaction curves for coarse and fine rubber are shown in Figures 2.1., and 2.2., respectively. The viscosity reached a maximum after approximately 1 hour in all cases expect the one containing 15 percent coarse rubber that appeared to still gain viscosity after 20 hours. For the 10 percent, or less, rubber content the reaction appears to be almost instantaneous. The results of this study indicated that the rubber did not appear to break down even after 20 hours, indicating that could be safely stored at 177°C (350°F). For both coarse and fine rubber the viscosity increased significantly

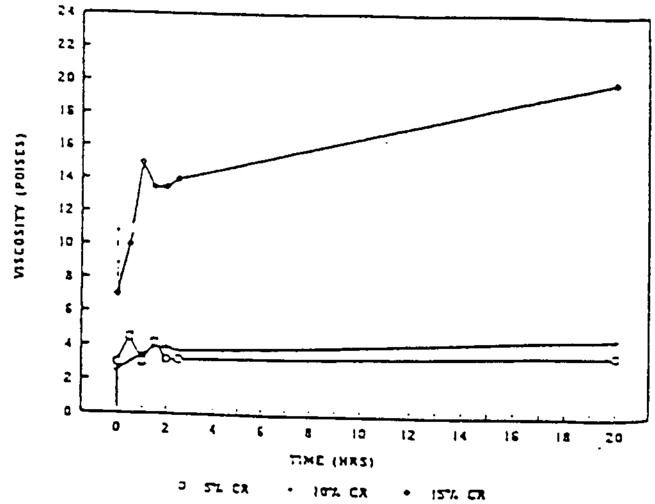


Figure 2.1 Reaction Curve for Asphalt and Coarse Crumb Rubber (Maupin 1992)

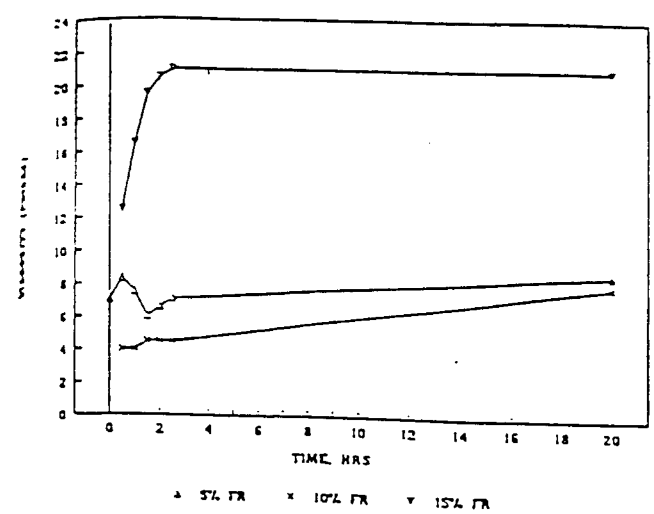


Figure 2.2 Reaction Curve for Asphalt and Fine Crumb Rubber (Maupin 1992)

between the 10 percent and the 15 percent rubber content. Some difference in the viscosity of the binder with the coarse and the fine rubber was observed. However the effects at the various rubber content were the same in both cases indicating that similar behavior is expected in blending operations in the field.

Asphalt-Rubber Binder Preparation and Testing

A method for preparing asphalt-rubber in the laboratory was suggested by Roberts (1987). According to this method the following steps are identified:

1. Heat gradually about 1000 ml of asphalt cement. Once the asphalt cement is fluid add another 100 ml of asphalt cement insert into a mixer. Continue heating the asphalt and increase the mixer speed to 500 rpm.

2. At 190°C (375°F) add the rubber to the flask (within 10 seconds).

3. Continue until the output from the stirrer reaches a uniform level and/or not less than an hour. Blending time will depend on the rubber type and concentration, and the rubber gradation.

Once the modified binder is being prepared is ready to be used with the aggregate. FHWA suggested the use of current standards for evaluating the asphalt rubber binder characteristics. Some of the standards include: AASHTO Standards M226 Viscosity graded asphalt binders, T49 Penetration of bituminous materials, T51 Ductility of bituminous materials, T179 Effect of heat and air on asphalt materials; and ASTM Standards D2994 Standard test method for rubberized tar, with Brookfield Viscometer, D36 Standard test method for softening point of bitumen, D3407

Standard test method for joint sealants, D88 Standard test method for saybolt viscosity, D92 Standard test method for flash point, D2007 Test method for characteristic groups in rubber extender and processing oils by clay gel absorption chromatographic method.

Some of the tests and their modifications reported in the literature for evaluating the asphalt rubber binder characteristics are briefly described herein (TTI 1986). In one of the studies a rotational viscometer was used for preparing the asphalt rubber binder. This system consists of a constant speed motor with stirrer assembly which is capable of recording torque changes as load varies on the stirrer. The resulting rotational viscometer is able to measure relative changes in fluid viscosity during mixing. Through the use of this viscometer the relative viscosity is being measured over time. The test, Torque Fork Mixer Test, considers slowly heating the asphalt cement while is being stirred. The mixer speed is increased gradually till 500 rpm. Upon reaching the required blending temperature the rubber is being added rapidly, within 10 seconds, and the reaction time is being recorded after all the rubber is being added to the asphalt and the mixer speed is maintained at 500 rpm throughout the blending process.

Another test that is being used for asphalt rubber binder evaluation is the Haake Viscometer Test. The Haake is a simple device which measures viscosity by the same principal as the Torque Fork Mixer, with the exception that changes in torque are monitored by the deflection of a calibrated spring rather than by increases in electrical current as with the Torque Fork. The Haake consists of a constant speed motor to which a cylindrical viscometer cup is attached. The cup is submerged in the asphalt cement. When the motor is started drag forces on the cub are generated as it rotates in the asphalt cement, and transmitted to the calibrated spring. The viscosity is measured then in poises.

A third binder evaluation method for asphalt rubber mixtures is the Force Ductility Test, a modification of the standard ductility test. In this case the tensile load-deformation characteristics of the binders are measured. The apparatus consists of the standard ductility apparatus, described in ASTM D113, using briquette specimens that are pulled apart by a tensile force while immerse in a water bath at 4°C. The force ductility results are reported as load required to cause elongation of the briquette sample till failure. As the sample is pulled apart, the force increases until it reaches a peak. Then the force decreases as the elongation of the specimen increases and finally becomes zero, i.e., when the specimen ruptures. Figure 2.3 shows an example of the force versus elongation for 4 blends containing 7 percent rubber.

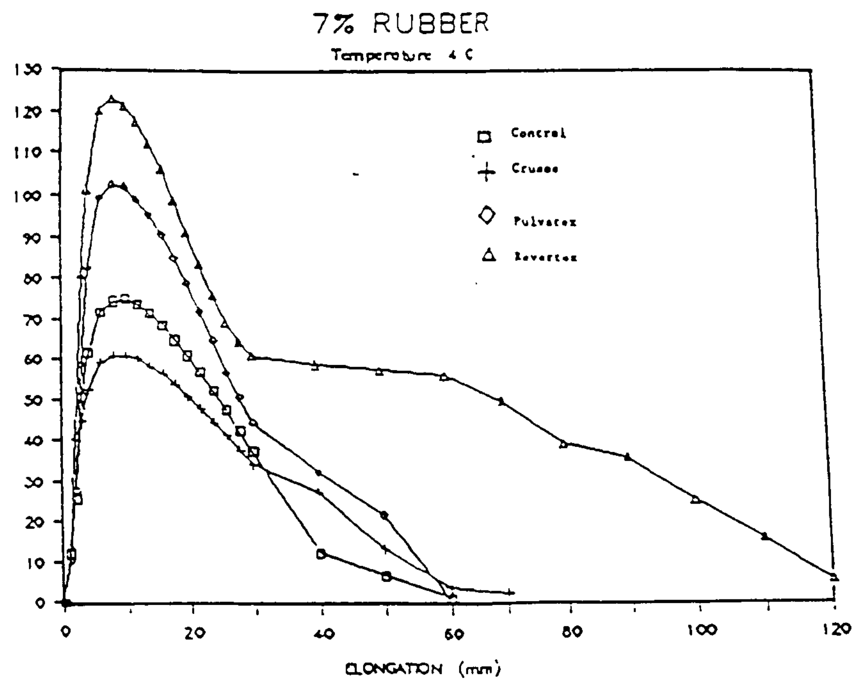


Figure 2.3 Force Ductility Test Results (Salter 1990)

Another binder evaluation test used in previous studies for rubber modified mixtures is the Double Ball Softening Point Test. The test is a modified version of the ASTM ring and ball softening point test. The apparatus consists of two 3/8 inch diameter stainless steel ball bearings cemented together with the test material. One of the ball bearings is fixed to the ring holder of the standard ring and ball assembly, and the other ball is suspended from the first by the test material, see Figure 2.4. As temperature rises in the immersed assembly the weight of the lower ball begins to stretch the asphalt rubber specimen. The double ball softening point is recorded as the temperature in the bath in which the suspended ball reaches the bottom plate of the assembly. FHWA provided suggested specifications for asphalt rubber binder. these specifications are reported in Table 2.2.

EVALUATION AND TESTING OF ASPHALT-RUBBER MIXTURES

Several tests have being proposed for evaluating asphalt mixtures. Objective of every test is to simulate in the best manner the field loading conditions and consider one or more of the three typical failures of asphalt materials, (i.e., fatigue, rutting, and low temperature thermal cracking). The examination of these tests lead to the selection of the testing technique to be used with improved mixture design methodology for rubber modified mixtures. The characteristics of these test are described briefly herein.

Repeated-Load Indirect Tensile Test

The indirect tensile test simulates the state of stress in the lower position of the asphalt layer, (i.e., tension zone). The test may be conducted with a single load to failure or with repeated loads (ASTM D 1423), and both laboratory and field-recovered cores may be evaluated. The test may be used to evaluate mixture properties that are directly related to distresses, and including the tensile strength, the modulus of elasticity and Poisson's ratio, fatigue and permanent deformation characteristics.

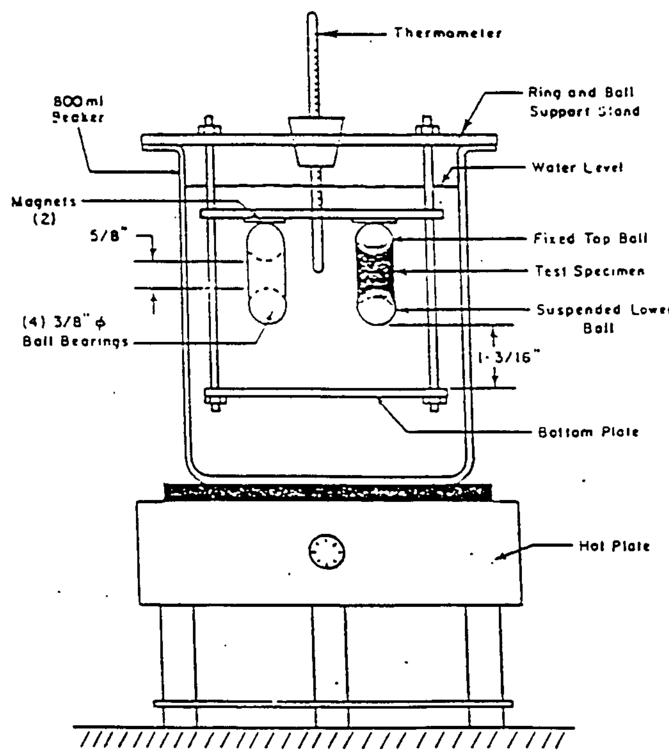


Figure 2.4 Double Ball Softening Point Apparatus (TTI 1986)

Table 2.2. Suggested Requirements for Asphalt-Rubber Binder (FHWA 1992).

GRADE	ARB-1	ARB-2	ARB-3
	CLIMATE ZONE		
TEST METHOD	HOT	MODERATE	COLD
Highest Mean Weekly Temp. °C (°F)	> 38 (> 100)	26 to 38 (80 to 100)	< 26 (< 80)
Lowest Mean Monthly Temp. °C (°F)	> 0 (> 30)	-12 to 0 (10 to 30)	< -12 (< 10)
Apparent Viscosity (P) 175°C (347°F) ASTM D 2994 spindle 3, 12 rpm	1000 min 4000 max	1000 min 4000 max	1000 min 4000 max
Penetration (1/10 mm) 25°C (77°F) AASHTO T 49 100 gram, 5 sec	25 min 75 max	50 min 100 max	75 min 150 max
Penetration (1/10 mm) 4°C (39.2°F) AASHTO T 49 200 gram, 60 sec	15 min	25 min	40 min
Softening Point °C (°F) ASTM D 36	54 min (130 min)	49 min (120 min)	43 min (110 min)
Resilience (percent) 25°C (77°F) ASTM D 3407	20 min	10 min	0 min
Ductility (cm) 4°C (39.2°F) AASHTO T 51 1 cm/min	5 min	10 min	20 min
Tests on Thin Film Oven Residue, AASHTO T 179			
Penetration (percent of original retained) 4°C (39.2°F) AASHTO T 49 200 gram, 60 sec	75 min	75 min	75 min
Ductility (percent of original retained) 4° (39.2°F) AASHTO T 51 1 cm/min	50 min	50 min	50 min

NOTE: The binder measured for compliance shall include the extender oil and any other additive proposed in the job mix formula.

According to the recommended testing conditions of the test, (ASTM D4123), the following conditions may be considered: testing temperature at 5°C (41 °F), 25°C (77 °F), and/or 40°C (104°F); load duration of 0.1 to 0.4 seconds; load frequencies of 0.33, 0.5, and 1 Hz; and load strain level of 10 to 50 % of the indirect tensile strength.

Failure criteria

The fatigue life of the specimen, or other failure modes, may be predicted from the laboratory testing, and are significantly influenced by the definition of "failure." Kim (1991) concluded that failure in diametral fatigue testing occurs when the permanent horizontal deformation reaches a level between 7 mm (0.28 in) and 9mm (0.36 in). Kim (1991) examined fatigue life of asphalt mixtures from diametral fatigue testing parameters. The failure criteria in this study was related to the horizontal tensile strain (ε_x) under the line of loading, given by:

$$\varepsilon_x = (2\,P/\,E\pi td)\ [\ \{(1+3\mu)d^4 - 8x^2 d^2 (1+\mu) + 16x^4 (1-\mu)\ \} / \{\ (d^2+4x^2)^2\ \}\ ]$$

where: p is the load amplitude of the applied load, Ib; E is the elastic modulus, psi; t and d are the specimen thickness and diameter, in; μ represents the Poisson's ratio; and x is the distance from the center line of the specimen, in;

By integrating the above equation along the diameter of the specimen the total horizontal strain is obtained:

$$\delta_H = \{\ P\,/\,E\,t\}\ [\{\ 4\,/\pi\} - 1 + \mu\ ]$$

The largest horizontal strain, that occurs in the middle plane of the specimen, occurs in the x = 0, and is given by:

$$\varepsilon_o = \{2\,P\,(1+3\mu)\ \}\,/\,E\pi td$$

where ε_o is the horizontal strain in the middle plane. In this investigation it was concluded that the horizontal deformation increased drastically after a value of 2.5mm (0.1 in) of horizontal deformation, see Figure 2.5, and thus it was concluded that failure of the mixture is when the total horizontal deformation reaches 2.5mm (0.1 in).

Repeated-Load Indirect Tensile Testing

Repeated load indirect tensile testing involves the use of 100 mm (4") diameter by 62.5mm (2.5") height samples. Measurements of elastic, total, and plastic (permanent) deformations are evaluated along the vertical and horizontal diameters along the applied load. Figure 2.6 illustrates a typical plot of load and deformation versus time. The response curves (load, vertical deformation, and horizontal deformation) over 2 cycles or more cycles are examined for determining the instantaneous and total rccoverable horizontal and vertical deformations. Data points related to the beginning of the relaxation period are used to compute the "instantaneous" properties while the values associated with the end of the relaxation period are used to compute "total" mixture properties, see Figure 2.6. During the static indirect tensile test conditions (required step before the repeated load testing of mixtures), the failure load as well as the total vertical and horizontal deformation along the vertical and horizontal specimen diameters are recorded. These values are used in the evaluation of mixture properties described next. During this test results are not affected by surface condition and specimen failure occurs in an area of relatively uniform tensile stress, see Figure 2.7.

Testing Recommendations

The Resilient modulus of asphalt mixtures may be used in evaluating the relative quality of materials, as well as an input in pavement design or pavement evaluation analysis. Several of the studies investigated the effect of testing parameters on resilient modulus. Almudaiheem et al (1991) investigated the effect of loading magnitude on the resilient modulus of asphalt concrete mixtures using the indirect tensile test. In this study the Marshall mix design method was used to prepare the specimens at different asphalt contents. For each asphalt content, 20 specimens were prepared, half to

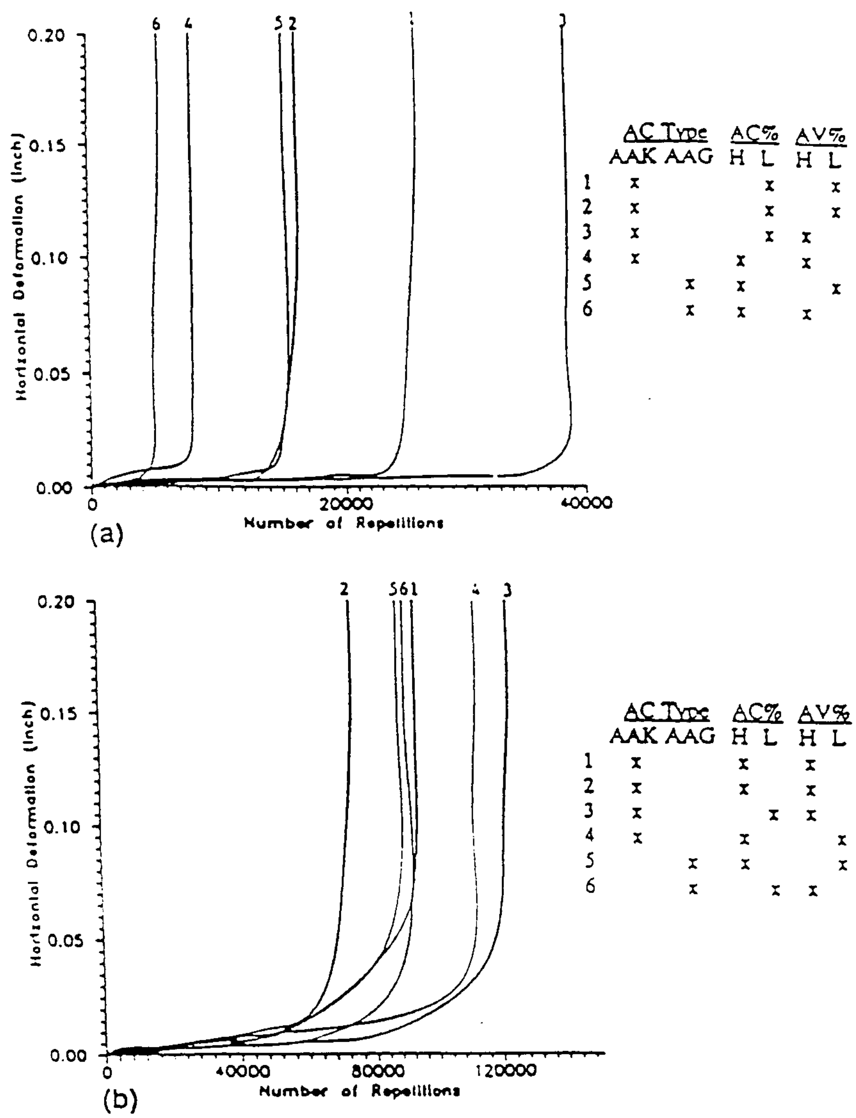


Figure 2.5. Horizontal Deformation at 0 °C (32 °F), for High (a) and Low (b) Stress Level (Kim 1991)

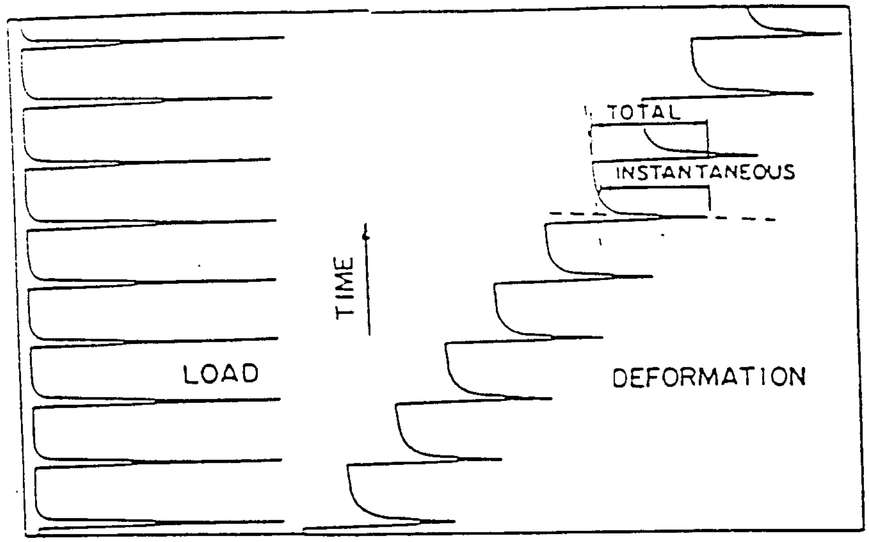


Figure 2.6 Plot of Load and Deformation versus Time for Resilient Modulus Testing (Mamlouk 1988)

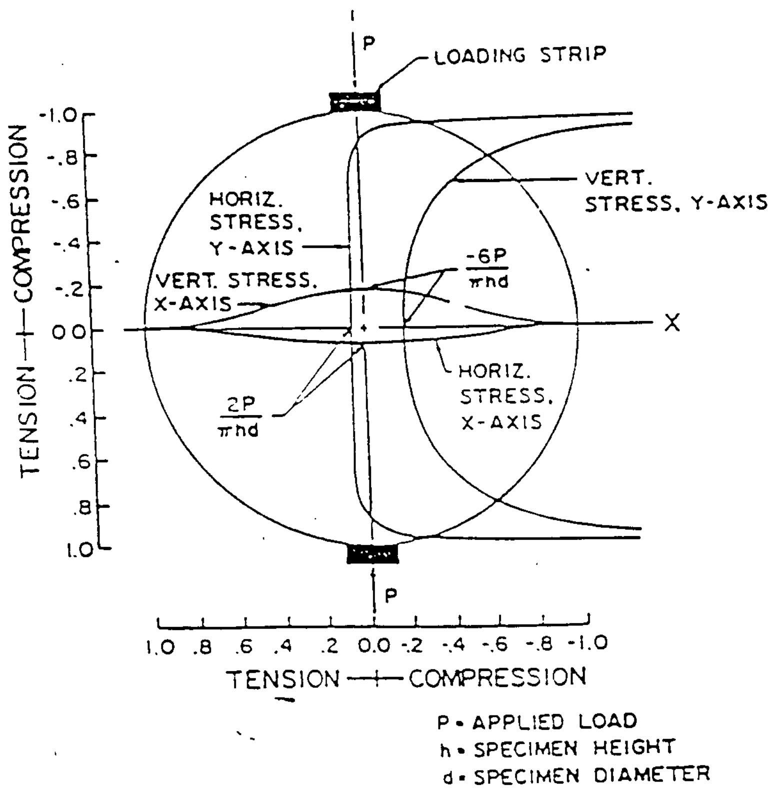
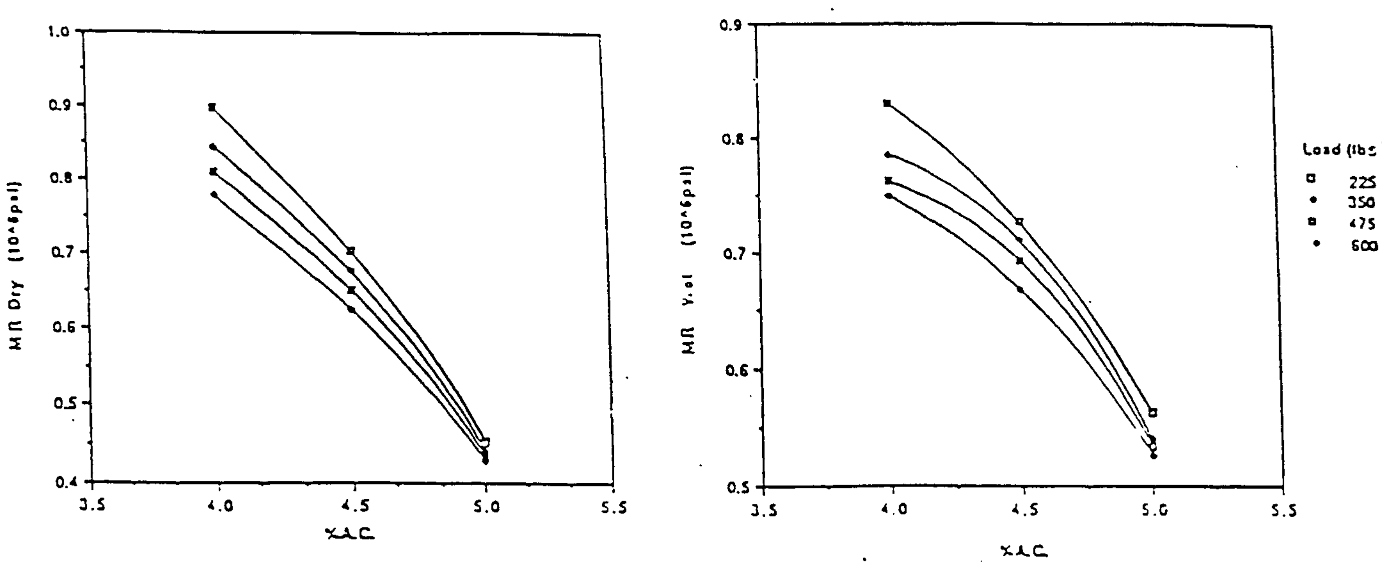


Figure 2.7 Stress Distribution Along the Principal Axes of Specimen in Diametral Resilient Modulus Testing (Mamlouk 1988)

be tested after being subjected to moisture conditioning, (wet specimens), and the other half at normal, (dry conditions). Each specimen was tested on three different axes, 60 degree apart, according to ASTM D4123. The specimens before testing were cured at 60°C(140°F) for 72 hours. The loading magnitude for each mixture was selected so that the applied stresses is within the 10-30 percent range of the indirect tensile strength of the specimens. Four levels of pulsating load were used 101.8kg (225lb), 158.3kg (350lb) , 214.9kg (475lb) , and 271.5kg (600 Ib). Table 2.3 provides the exact percentages of the indirect tensile strength used as the applied load. The results, presented in Figure 2.8, indicate that MR obtained from the diametral resilient modulus test depend on the load magnitude. Typically, a larger load yields a smaller MR-value. Thus for a conservative design a larger load may be used since it will provide a more conservative design associated with smaller MR value.

An extensive investigation was conducted by Boudreau (1992) on the effects of materials and testing parameters, recommended by ASTM D4123, on Resilient modulus results. Several replicates were prepared for each mixture, using two different type of aggregates, and prepared at two air void contents. The testing variables included in the study are shown in the factorial of Table 2.4. The specimens were compacted with the Marshall method and two different levels of compaction were used for achieving the 4 and 10 % air void content. A dense graded mix was prepared with an AR-4000 asphalt cement.

The mean instantaneous and total resilient modulus for the four type of mixtures, (A4 indicating use of aggregate A and mixture air void content of 4%, A10 for aggregate A and air void of 10%, B18 for aggregate B and 10% air void, and BL4 for aggregate B with lime addition and 4% air voids), are shown in Tables 2.5 and 2.6. The links on the right side of the columns indicate the mixtures which although have different air void content the resilient modulus results could not differentiate between them. On the other hand, the links on the left side identify mixtures of the same air void content and tested at the same loading conditions for which the resilient modulus results could not differentiate. The analysis of variance analysis conducted in this study identify the 5°C (41°F) temperature as the preferred testing temperature since is able to



a. Dry Resilient Modulus b. Wet Resilient Modulus

Figure 2.8 Resilient Modulus versus Asphalt Content for Different Load Magnitudes (Almudaiheem 1991).

Table 2.3 Exact Percentages of Indirect Tensile Strength Used as Applied Load (Almudaiheem 1991)

Load (lbs)		A.C.% 4	4.5	5
225	Dry	10.51	10.64	10.84
	Wet	10.00	10.79	11.35
350	Dry	16.36	16.45	16.86
	Wet	15.55	16.78	17.40
475	Dry	22.20	22.61	22.89
	Wet	21.31	22.78	23.54
600	Dry	28.05	28.38	28.91
	Wet	25.66	28.77	29.73

Table 2.4. Factorial Experiment (Boudreau 1992)

Temperature (°F)		41			77			104		
Microstrain		50	75	100	50	75	100	50	75	100
Duration (hr.)	Frequency (sec.)									
0.1	0.33					X				
	0.5	X	X	X	X	X	X	X	X	X
	1.0					X				
0.2	0.33									
	0.5					X				
	1.0									
0.4	0.33									
	0.5					X				
	1.0									

Table 2.5 Mean Values of Instantaneous Resilient Modulus in Ksi, with n=3, (Boudreau 1992)

	CONDITIONS*									
	1	2	3	4	5	6	7	8	9	10
Temperature(°F)	41			77						
Frequency (hz.)	0.5			0.5	0.33	0.5			1.0	0.5
Duration (sec.)	0.1			0.1	0.1	0.1	0.2	0.4	0.1	0.1
Microstrain	50	75	100	50	75					100
MATERIALS**										
A4	2085	2083	2121	1283	1109	948	785	582	866	935
BL4	1743	1758	1571	1125	867	1033	714	726	921	928
A10	1336	1223	1284	503	576	672	281	199	820	738
B10	1327	1435	1362	746	759	568	624	490	772	699
Average	1623	1625	1585	914	828	830	601	499	845	825

* Conditions are combinations of temperature, load frequency and duration, and microstrain level.
** Materials are combinations of aggregate type, air void content and additive type.

Table 2.6 Mean Values of Total Resilient Modulus in Ksi, with n=3, (Boudreau 1992)

	CONDITIONS*									
	1	2	3	4	5	6	7	8	9	10
Temperature(°F)	41			77						
Frequency (hz.)	0.5			0.5	0.33	0.5			1.0	0.5
Duration (sec.)	0.1			0.1	0.1	0.1	0.2	0.4	0.1	0.1
Microstrain	50	75	100	50	75					100
MATERIALS**										
A4	1801	1801	1840	410	398	396	256	183	392	409
BL4	1642	1610	1406	433	504	429	283	268	436	409
A10	1063	1033	1017	187	175	132	108	74	231	211
B10	1224	1259	1211	300	353	306	214	173	344	310
Average	1433	1426	1369	333	358	331	215	175	351	335

* Conditions are combinations of temperature, load frequency and duration, and microstrain level.
** Materials are combinations of aggregate type, air void content and additive type.

discriminate between materials and provide higher reparability. Further examination conducted by Bourdeau indicated that 10°C (50°F) and 15°C (60°F) testing temperatures provides similar testing accuracy and results associated with 0.1 second load duration, 0.33 Hz load frequency, and 50 to 75E-4 percent induced strain (50 to 75 micostrain).

For the repeated load test, a minimum of 50 to 200 load repetitions are typically applied to the specimen before any readings are taken, so as to properly seat the loading strips on the specimen and to allow the sample deformation to stabilize. In addition, a small static load, 4.5 kg (10lb) to 22.6 kg (50 Ib), is applied to hold the sample in place.

Material Properties Evaluation

The analytical tools for evaluating material properties consider the following assumptions: homogenous and isotropic material; and load is applied in the normal direction to the contact area between the specimen and the loading strip (i.e., there is no friction between the specimen and the loading strip). Considering the static indirect tensile test, the asphalt mixture static characteristics such as: Poisson's ratio, resilient modulus or modulus of elasticity, and indirect tensile and compression strengths at the center of specimen are then calculated according to the follow equations:

$$\mu = [A1 - (A2)(DR)] / \{A3 + (DR)\}$$

$$MR = P\,[A1 - (A3)(\mu)\,] / L\,(DV)$$

$$MR = (A4)\,P\,\mu / DL$$

$$INTS = (A6)\,P / L$$

$$INCS = (A5)\,P / L$$

where: μ is the Poisson's ratio; DR represents the deformation ratio DV/DH, with DV the vertical total deformation along the vertical diameter of the specimen (in) and DH the horizontal total deformation along the horizontal diameter of the specimen (in); DL is the radial deformation along the thickness of the specimen (in); MR is the Resilient modulus (psi); L represents the sample thickness (in); P is the magnitude of the applied load (Ib); INCS represents the indirect compression strength at the specimen center (psi); INTS is the indirect tensile strength at the specimen center (psi); and Ai are constants obtained in function of specimen diameter and shown in Table 2.7.

Table 2.7 Regression Constants for 100 mm (4") and 150mm (6") specimen diameters (Baladi 1989)

Constants	Specimen Diameter mm (in)	
	100 mm(4")	150mm(6")
A1	3.587910	4.085950
A2	0.269895	0.271760
A3	0.062745	0.041733
A4	0.319145	0.212453
A5	0.475386	0.105242
A6	0.156241	0.317695

Under the assumption of homogenous, isotropic and linear elasticity the resilient modulus provided from the previous two equations should have identical values. Since, asphalt mixes are heterogeneous and anisotropic difference between these two calculated values are expected. In order to obtain more realistic values for the modulus and Poisson's ratio the deformations in the three directions may be used. Using the least square technique and the above equations these parameters may be calculated from the following (Baladi 1989):

$$\mu = 1/D\,[0.225127\ H^2 - 0.269895\ V^2 - 0.0447676\ A^2 + 3.570975\ (H)(V) + 0.086136\ (A)(H) + 1.145064\ (A)(V)]$$

$$MR = \{\ 0.253680\ H + 3.9702876\ V - 0.0142874\ A\}/\ D$$

$$D = 1.105791\ (H^2 + V^2 + A^2) - (H - 0.0627461\ V + 0.319145\ A)^2$$

where: H= DH (L/P); V = DV (L/P); and A = DL/P.

Considering the repeated-load indirect tensile test, the instantaneous and the total values of Poisson's ratio and resilient modulus are calculated using the following equations:

$$E_{RI} = P\ (\mu_{RI} + 0.27)\ /\ tDH_I$$
$$E_{RT} = P\ (\mu_{RT} + 0.27)\ /\ tDH_T$$
$$\mu_{RI} = 3.59\ DH_I\ /DV_I\ - 0.27$$
$$\mu_{RT} = 3.59\ DH_T\ /DV_T - 0.27$$

where: E_{RI} is the instantaneous resilient modulus, Mpa; E_{RT} is the total resilient modulus, Mpa; μ_{RI} is the instantaneous resilient Poisson's ratio; μ_{RT} is the total resilient Poisson's ratio; t represents the specimen thickness, mm; P is the repeated load, N; DH_I is the instantaneous recoverable horizontal deformation, mm; DH_T is the total recoverable horizontal deformation, mm; DV_I is the instantaneous recoverable vertical deformation, mm; and DV_T is the total recoverable vertical deformation, mm.

Static and Repeated-Load Creep Test

As it was mentioned previously, creep evaluation of asphalt mixtures may be used in the mixture design process. The creep test may be used for: evaluating mixture susceptibility to deformation, and b) the determination of stiffness at long loading duration to be used in thermal cracking analysis of the mixture.

The specimens used for creep evaluation of asphalt mixtures have a height to diameter ratio of 2:1. Typically 100 mm (4") diameter by 200 mm (8") height specimens are used with sample ends well greased with graphite-based lubricant to reduce friction between sample and testing plate.

The static creep test is a uniaxial, unconfined creep test involving a 2 minutes preconditioning load followed by 5 minutes of rest period (Krutz and Neil 1991 and 1992). Immediately after the rest period a static load is applied for 60 minutes followed by 15 minutes rebound period. The test is conducted at 25°C (77°F) using a static stress of 345kPa (50 psi), and at 60°C (140°F) using a static stress of 138kPa (20 psi). The resulted axial deformations are recorded every 60 seconds.

The repeated -load creep test, is the triaxial repeated-loading confined test covered by SHRP A-003A (Krutz and Neil 1991 and 1992). This test uses 1 minute precondition period followed by a 60 minutes testing period. The repeated loading sequence consists of 0.1 sec duration haversine pulse load followed by 0.6 rest period, with frequency of 1.43 Hz. In all steps a confining pressure of 103.5kPa (15 psi) is used. As for the static creep test, the repeated load creep test is conducted at 25°C (77°F) using a peak deviator stress of 345kPa (50 psi), while for the 60°C(140° F) a deviator stress of 138 kpa (20 psi) is being used.

Mixture Properties Evaluation

Mixture deformations may be continuously monitored with the use of two linear differential transducers (LVDTs). The LVDTs are positioned 180° apart and measure deformations over the total sample height. The deformations are electronically averaged and recorded every 60 seconds throughout the test with an accuracy to 0.0025mm (0.0001 in). The measurements can then be used for calculating the compressive strain for each test over the sample height:

$$E(t) = [d(t)/Ho]$$

where: E(t) represents the strain at time t, in/in; Ho is the original sample height; d(t) is the deformation of the sample along its height at time t, in inches. Such evaluation may be used with both unconditioned and moisture-conditioned samples. The creep modulus (mix stiffness, Smix) is then calculated by the following equation:

$$S_{mix}\,(T,t) = \sigma / \varepsilon_t$$

where: S_{mix} (T,t) is the mixture stiffness at temperature (T) and the time of loading (t); σ is the applied stress, psi; and ε_t is the axial strain at time (t). The creep compliance (J_t) is calculated by dividing the strain at time (t) with the applied stress:

$$J_t = \varepsilon_t / \sigma$$

Mohammed (1992) provided a relationship for the creep modulus, S(t), in function of the applied load, P_o, the Poisson's ration, μ, the sample thickness, t, and the horizontal deformation at time t, $\delta H(t)$. In this experiment the creep test was conducted at 25°C(77°F) using a ramp of load, 113.1kPa (250 Ib), applied using the stress controlled mode of the MTS machine. The load, vertical and horizontal deformations were monitored continuously with a data acquisition system and the test was completed either at 60 minutes of load duration or at failure. The mean horizontal and vertical deformations were computed for each set of specimens and the creep modulus was computed by:

$$S(t) = P_o(\mu + 0.27)/\, t\, \delta H(t)$$

Effect of Sample size on Creep Testing

The effect of sample size on creep evaluation of modified mixtures was investigated by Neil (1991) with the static unconfined creep test at 25°C (77°F). In this case six polymer modified mixtures were experimentally placed in test sections in Nevada from where samples were collected behind the paver. The mixtures were prepared with one type of aggregate and including two levels of modifier, low (L) and high (H), as well as control mixtures. Two sample sizes were used in this experiment, 100 mm (4") diameter by 200 mm (8") height, and 100 mm (4") diameter by 62.5mm (2.5") height, and the samples were moisture conditioned according to the Lottman's procedure. The deformations were was measured over both the full length for both samples and the center third for the 200 mm (8") samples, see Figure 2.9.

The test results are presented in Table 2.8, for both sample sizes. As it can be seen from this Table the sample size, and the method of measurement for the 200 mm (8") height samples provides different values of permanent deformation. Historically, the height-to-width ratio of axially compressed specimens has been 2 :1 due to the effect of end constraints, and thus the results from the 200 mm (8") height samples might be more accurate. Overall, the mixture with lower average permanent strain is expected to exhibit less rutting in field conditions than the mixtures with higher values.

Table 2.8 Creep Testing Results (After Neil 1991)

Binder Type	**Average Permanent Strain (in/in)**		
	Sample Size		
	4" x8"		4" x2.5"
	Full Depth	Center Third	-----
AC-20-RL	0.002636	0.001785	0. 003507
AC-20RH	0.003166	0.002263	0.003514
AR-4000-R	0.005519	0.003529	0.002574
AC-20pm	0.008942	0.004844	0.004243
AC-20-P (Control)	0.01127	0.009385	0.001924
AR-4000-pm	0.011364	0.010131	0.003272

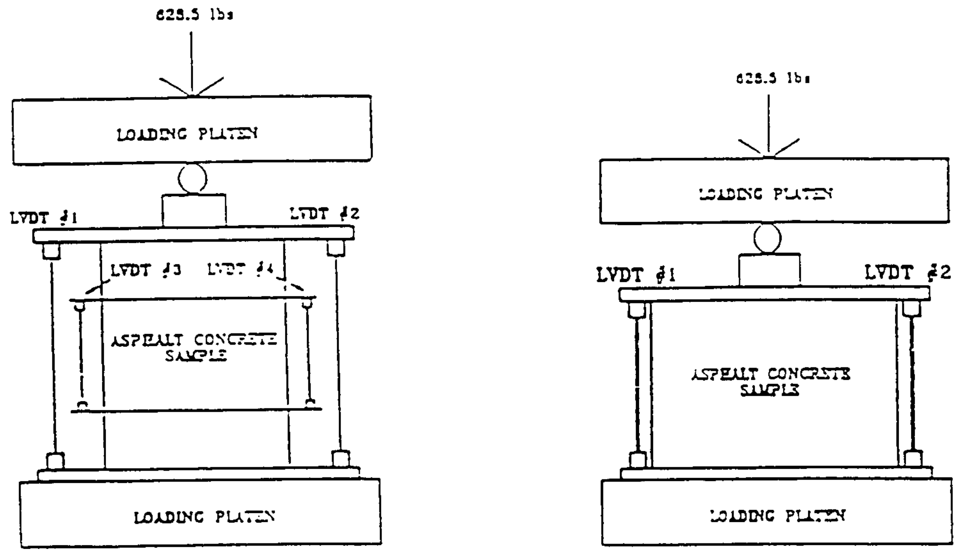


Figure 2.9 Test Setup for Different Sample Sizes (Neil 1991)

Rutting Prediction from Creep Test Results

Van de Loo (1988) provided a relationship for evaluating the permanent deformation in the asphalt layer from the creep test results. This relationship is given by:

$$\delta = C_m \; H_o \, \sigma_{avg} \, / \, S_{mix}$$

where: δ is the reduction in asphalt layer thickness; C_m is the correction factor for the so-called dynamic effect, which takes account the difference between the static (creep) and the dynamic (rutting) behavior of the mixture. This factor depends on the mixture and must be determined empirically; H_o is the design thickness of the asphalt layer; S_{mix} represents the mixture stiffness which was shown to be affected by the binder stiffness; and σ_{avg} is the vertical compressive stress. The vertical compressive stress may be determined with the use of ELSYM5 or other analytical tools.

Fatigue Evaluation

Different specimen characteristics and testing conditions have been used for fatigue evaluation of asphalt mixtures. Raad (1993) in his investigation used beam specimens, 75mm (3") x 75mm (3") x 375mm (15") that were loaded at 125mm (5") interval third points using an MTS. The specimens were evacuated with constant strain testing and with a load duration of 0.1 second and frequency of 1 Hz, at 21°C (70°F) with ±1.6°C(3°F). Another study conducted by Salter (1990) for evaluating the effects of natural rubber powder into the fatigue life of asphalt mixtures, used a sinusoidal load with 2 Hz frequency for testing the 37.5 x 37.5 x 375 mm mixture samples at -4°C(20°F) with ±0.8°C(1°F). Nick (1990) investigated the fatigue characteristics of different types of asphalt mixes using beam specimens of 50mm(2") x 50mm(2") x 375mm(15 in) with a displacement control mode and 0.1 second load duration with 0.9 seconds rest period. For the mean specimens the loads are typically applied in the third-point loading condition and ensuring a constant bending moment over the middle third of the beams. The long rest period is usually used to ensure that full material recovery is achieved before the next loading cycle.

The fatigue life of asphalt mixes is obviously dependent on the failure definition. Raad (1993) assumed that fatigue failure will occur when the flexural stiffness "E", determined from the central beam deflections and the applied load, is reduced by 50%. In another study (Salter 1990), it was assumed that the failure was achieved when a reduction in the applied load of the beam was reduced to half and after the first 200 load repetitions. Similarly, Nick (1990) selected the fatigue failure as the point where the applied load drops to 50 percent of its initial value and for producing a given strain level.

Mixture Properties Evaluation

Load, deflection, and strain are to be measured during testing every one second. Loads may be measured with a load cell, while beam deflections may be measured with LVDTs. The strains may be directly measured using an extensometer attached to the beam with adhesive. During testing, strain is measured only in the tension region of the beam but to confirm that the system is functioning adequately initial tests using two extensometers, to measure strains on both sides of the beam, may be performed. Stresses can be calculated from the monitoring of load and deflection, and the beam dimensions, as described below:

$$\Delta_{max} = Pa\,(3L^2 - 4\,a^2)\,/\,24\,EI$$

where: Δ_{max} is the maximum deflection at the center of the beam, in; P is the applied load, Ib; a is the distance between the exerted load and the nearest support, in; L is the reaction span, in; I is the specimen moment of inertia, in^4; and E is the flexural beam modulus, psi. For a = L/3 the equation takes the form:

$$\Delta_{max} = 23\,P\,L^3\,/\,72\,EI$$

The maximum bending moment, M_x in lbxin, at the center of the beam is calculated by:

$$M_x = Pa$$

and at a = L/3 the maximum banding moment is given by:

$$M_x = PL/3$$

Knowing I and M_x, the normal stresses equation may be used for calculating the tensile stress at the bottom of the beam:

$$\sigma_x = M_x \ y / I$$

where: σx is the tensile stress at the bottom of the beam, psi; and y is half of the beam depth, in; The modulus, E, is calculated from the computed stress and the measured by:

$$E = \sigma / \varepsilon$$

with σ the computed stress, psi; and ε the measured strain, in/in.

Repeated Loaded-Wheel Test

This test is being used for characterizing the rutting potential of asphalt mixtures through the testing of beam specimens. Several repeated wheel loading apparatus have been developed and used over the years. One of the first testing devices, (Lai and Minglee 1990), was using a 25mm (1") wide aluminum loading wheel of 75mm (3") diameter, fitted in a hard rubber tire, since the use of the tire would exert nonuniform contact pressure on the sample surface. A modified version of this device was proposed, with an 200 mm (8") aluminum wheel and a 25mm (1") diameter high pressure rubber-hose wrapped around the rim. The hose in this setup could be pressurized till

690kPa (100 psi). Results from the use of this device indicated that higher skidding was occurring at the end of the samples causing wear of the rubber hose and excessive rutting at the sample ends and with a tendency of pushing instead of rolling of the hose in the wheel path. Shoving at ends of the specimen became evident along these regions and slowly progressed towards the center.

To overcome these problems, additional modifications were suggested for the loaded-wheel testing machine (LWT) by Lai and Minglee (1990). A flexible linear tube, made of high pressure rubber hose was pressurized to the required pressure and placed on the top of the specimen, see Figure 2.10. The degree of rutting was reduced but it was still greater at the ends than in the middle region of the specimen, potentially due to the longer load duration at the ends, Figure 2.11. Additional questions related to this device to be addressed include: the nature of the contact pressure; and the effects of hose stiffness on mixture rutting, Figure 2.12. For this test beam specimens of 75mm (3") x 75mm (3") x 375mm (15") were used (Lai and Minglee 1990) with an 45.2kg (100 Ib) load and 690Mpa (100psi) contact pressure. The load repetitions were kept at 22 and 44 cycles/minute and a testing temperature of 35°C (95°F) was used. The longitudinal and transverse profiles were measured after 0, 200, 500, 1000, and 2000 cycles see Figure 2.11.

The rutting potential of conventional mixtures was evaluated with the loaded wheel test by Lai (1986) and Minglee (1990). The asphalt beam samples were prepared with different type of aggregates, filler contents, asphalt cements, and modifier contents. The optimum asphalt contents for each mixture were based on the Marshall method and based on a 4.5% air void content. The testing conditions included a temperature of 35°C (95°F), a load of 45.2 kg (100 Ib), testing frequency of 22 cycle/minute, and a contact pressure of 690 kPa (100 psi). An example of the rutting profiles along the wheel path measured at 0, 200, 500, 1000, 2000 cycles and so on, is shown in Table 2.9, while the relationship between the number of load repetitions and the resulted rut depths are shown in Figure 2.13.

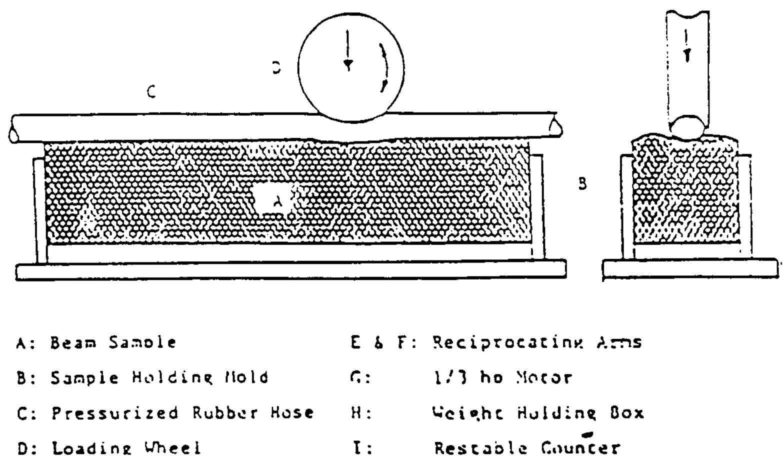


Figure 2.10 Modified Loaded Wheel Testing Apparatus (Lai and Minglee 1990)

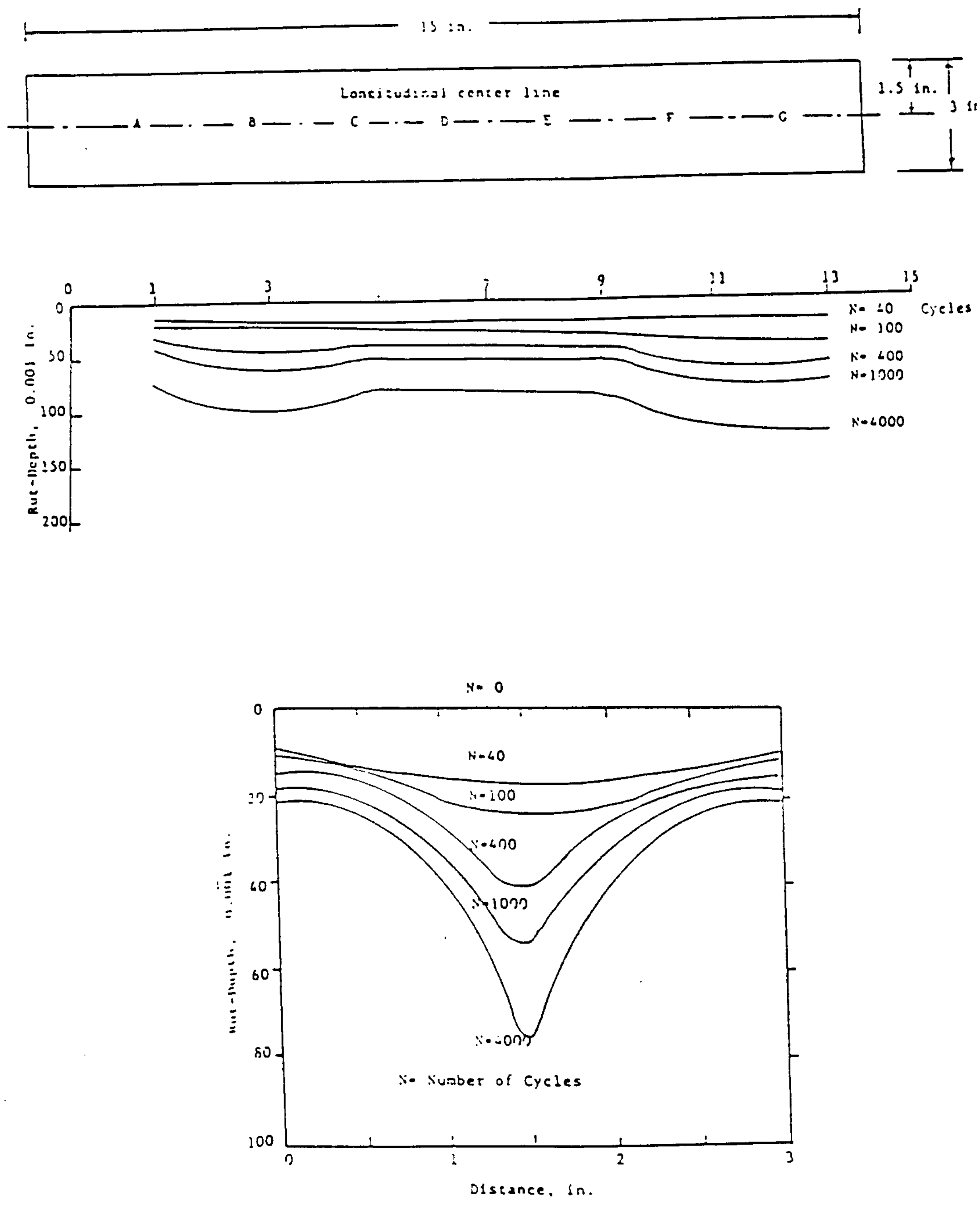


Figure 2.11 Longitudinal and Transverse Rutting (Lai and Minglee 1990)

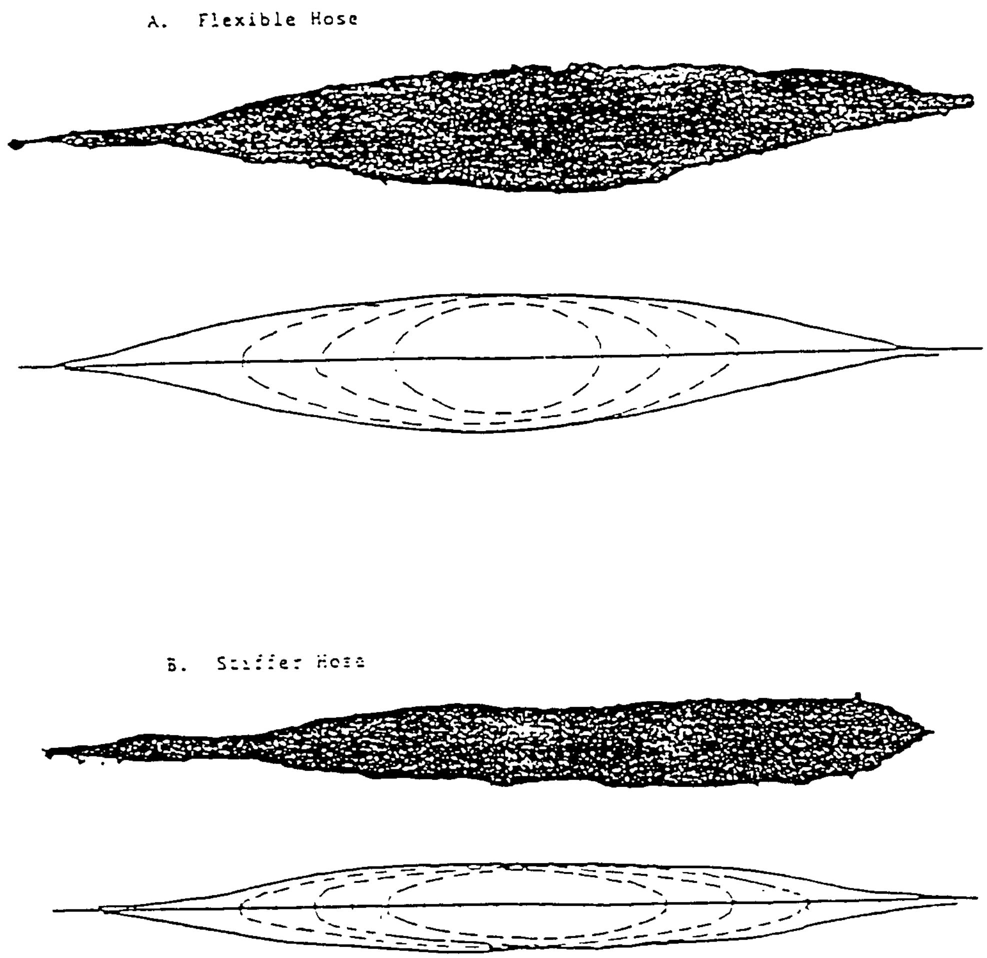


Figure 2.12 Contact Imprints of Rubber Hose at 45.2 Kg (100 lb) Load and 690 kPa (100 psi) pressure (Lai and Minglee 1990)

Table 2.9 Rutting Results with Loaded Wheel (Lai and Minglee 1990)

Mix Design	MARSHALL DESIGN Density	Stability	Flow	Tensile	FD Stiffness	air voids	Unit Weight	BEAM RUT-DEPTH ,1/1000 IN. @ /Cycles 200	500	1000	2000	3000	4000	5000	6000	7000	8000	9000	10000
CS-1						6.01%	151.0	56	72	86	102	110	124						
CS-2						7.73%	148.5	50	66	81	91	102	107	113	117				
CS-3						6.44%	150.4	45	59	74	92	102	111						
CS	152.6.	2020	13.7	42.6	14077		150.0	50	66	80	95	105	114						
C3M-1						7.20%	151.6	43	65	68	83								
C3M-2						8.46%	149.8	52	73	91	117								
C3M-3						7.62%	151.0	43	66	74	86								
C3M	155	2620	9.8	44.3	17908		150.8	46	61	78	95								
CSM-1						6.42%	152.9	4F	48	60	72	80	85	88	92				
CSM-2						7.68%	151.0	50	66	82	101	106	119	126	129				
CSM-3						5.16%	154.8	38	51	62	75	82	87						
CSM	155.1	2820	9.7	56	19667		152.9	45	55	68	83	89	97						
CST-1						8.49%	149.1	49	64	78	93								
CST-2						8.07%	149.8	37	51	68	89								
CST-3						8.91%	148.5	36	49	60	73	88	95	101	106	111	121	124	128
CST	154.4	2510	9.1	38	14988		149.5	41	55	69	85								
C6K-1						6.22%	151.0	48	62	71	83	87	84	87	86				
C6K-2						7.04%	149.8	38	45	52	59	63	65	66	68				
C6K-3						6.22%	151.0								62				
C6K	152.9	2330	11.3	42.8	13662		150.5	43	54	62	71	75	75	77	77				
CSS-1						6.56%	151.6	26	36	44	58	64	71	76	82	85	89	93	96
CSS-2						7.09%	151.0	37	45	52	66	75	81						
CSS-3						7.09%	151.0	25	34	42	51	59	61						
CSS	154.2	2760	13.1	43.5	20680		151.2	29	38	46	58	66	71						
CIM-1						6.28%	151.6	41	52	62	72	94	102						
CIM-2						5.81%	152.3	51	65	78	91	108	116						
CIM-3						6.20%	151.6	56	70	86	104	117	120						
[illegible]	[illegible]	[illegible]	11.3	[illegible]	[illegible]		151.8	49	62	75	09	106	115						

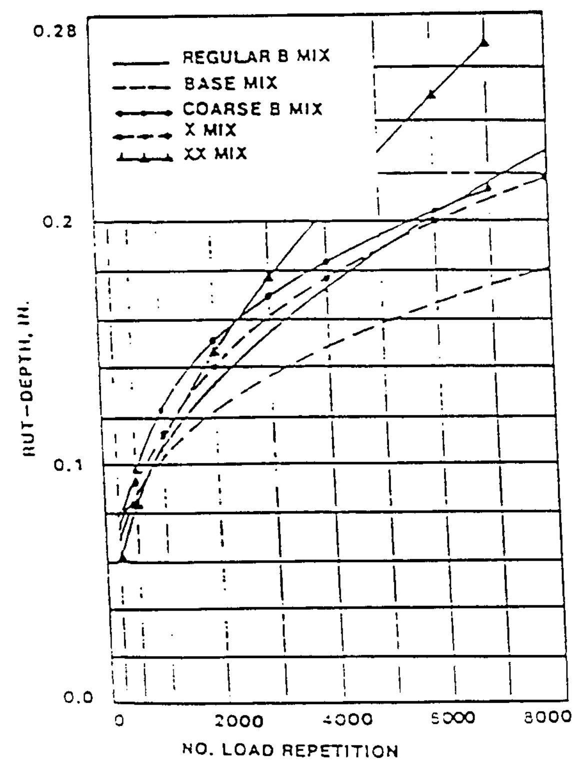


Figure 2.13 Rut Depth and Load Repetitions Relationships (Minglee 1990)

ASPHALT MIXTURE SPECIMEN CHARACTERISTICS AND PREPARATION PROCEDURES

Depending on the tests selected for evaluating mixture behavior and performance cylindrical or beam laboratory prepared or field cores may be used. Two types of cylindrical specimens may be used, 100 mm (4 in) in diameter by 62.5mm (2.5 in) in height or 100 mm (4 in) in diameter by 200 mm (8 in) in height, depending on the testing program. The first type of specimens is used for the standard Marshall mix design method and for fatigue evaluation of asphalt mixtures with the indirect tensile test in static or repeated-loading mode. The second type of specimens may be used for evaluating the dynamic properties of the asphalt mixes. However, based on the characteristics of the available apparatus for dynamic testing specimens with a 2:1 height to diameter ratio may be used.

Different compactors may be used for the preparation of cylindrical specimens and including the standard Marshall compactor, the California kneading compactor, and the gyratory testing machine. Depending on the compaction device, standard methods of compaction are available today and include: the standard Marshall method (ASTM D1559); the preparation of bituminous mixture test specimens by means of the California kneading compactor (ASTM D1561); the test method for compaction and sheer properties of bituminous mixtures by the U.S. Corps of Engineers gyratory testing machine (ASTM D3387); and the standard method for the preparation of bituminous mixture specimens for dynamic modulus testing (ASTM D3496). In almost all cases some identical modification to the standard specimen preparation methods have been recommended, especially for specimens used in dynamic testing. Some of these recommendations are reported next.

Cylindrical Specimen for Conventional Asphalt Mixtures

For specimens compacted with the kneading compactor, ASTM D1561, the mix is placed in 60°C (140°F) forced draft oven for 15 hours before being reheated to 110°C (230°F) for compaction. The 100 mm (4") by 62.5mm(2.5") specimens are compacted in one layer with 30 blows at

1725kPa psi. After compaction the sample is placed in the oven at 60°C (140°F) for 1.5 hours prior to the application of a 5248.8Kg (11,600 Ib) leveling load.

Specimens of 200 mm (8") by 100 mm (4") are compacted in thirds using the kneading compactor, (i.e. the mold is filled up to 1/3 of the height and is compacted, then the second third of the mold is filled and compacted and so on). Each lift is compacted with 30 blows at 1725kPa (250 psi). After compaction the sample is placed in the oven at 60°C (140°F) for 1.5 hours before the application of a 2262.4kg (5000 Ib) leveling load. The samples are then allowed to cool before being extruded from the molds.

Cylindrical Specimen for Asphalt-Rubber Modified Mixtures

A typical preparation of asphalt rubber mixtures involves heating the aggregate at 149°C (300°F) and the rubberized binder to 177°C (350°F) before mixing and regardless of the base asphalt viscosity. After mixing the aggregate and the modified binder, the samples are placed in a forced draft oven at 60°C (140°F) for 15 hours. The samples are then heated to 149°C (300°F) for compaction. For the 200 mm (8") by 100 mm (4") samples the same compaction procedure as for conventional mixtures is being used with the exception that the 1.5 hours curing time at 60°C (140°F) is extended to 3 hours. Rubberized samples are then allowed to cool before extruded from the molds.

Roberts (1987) used the California kneading compactor for preparing asphalt rubber concrete specimens and it concluded that the samples should be left in the mold for three days since the specimen swell and crack when extracted earlier. Lalwani and Schuler used the Marshall hammer compaction conditions for preparing this type of samples reporting the need to allow the specimens to cool to room temperature before extracting them from the molds (Roberts 1987).

Beam Specimens for Conventional Mixtures

Lai and Minglee (1990) used a static compression load to prepare beam specimens for rutting evaluation using the loaded-wheel test. Based on the required optimum bulk density from Marshall results and the used mold internal volume, the required material weight may be easily evaluated. The beam specimens are then compacted in three layers. After placing the third layer of the mixture the specimen is compacted to approximately the required height. A thick loading plate with size equal to the top area of the mold is placed on the top of the beam and a high pressure is applied so as to compress the mix to the specified height. Generally, it was found that the height of each specimen is slightly higher than the required one since it is too difficult to compress the sample to the desired density.

The mixture in this case is blended according to ASTM D1561. After the mixture is being prepared at the specific temperature, is placed in a forced draft oven at 60°C (140°F) for 15 hours. Then the specimen is reheated to 110°C (230°F) for compaction. After compacting the third lift, the sample is placed in 60°C (140°F) oven for 1.5 hours and consequently a high compression leveling load is applied.

Beam Specimens for Asphalt-Rubber Mixtures

A similar procedure is used for the preparation of the asphalt rubber beam specimens. The mixture is blended by heating the aggregate to 149°C (300°F), and the rubberized binder to 177°C (350°F) before mixing. Then, the samples are placed in a forced draft oven at 60°C (140°F) for 15 hours. The samples are then heated to 149°C (300°F) for compaction. Compaction is similar to the conventional beam specimens with the exception that the 1.5 hours of curing time at 60°C (140°F) is extended to 3 hours. Rubberized samples are then allowed to cool before extruded from the molds.

Moisture Conditioning

When the moisture effects on mixture properties are to be investigated moisture condition may take place. Neil (1991) briefly described the three steps in specimen moisture conditioning using the Lottman's procedure. This procedure consists of immersing the samples in water and applying a vacuum of 24" Hg for 10 minutes to achieve a minimum of 90 % saturation. Samples are wrapped with plastic and placed in a -18°C (0.0°F) freezer for a minimum of 15 hours. The samples are then unwrapped and transferred to a 60°C (140°F) water bath for 24 (± 0.5) hours, and immediately after placed in a 25°C (77°F) water bath for 2 hours so as to cool the samples to the testing temperature. Other methodologies of moisture conditioning have being proposed as well and depend on the testing methodology used for such evaluation.

CHARACTERIZATION OF ASPHALT MIXTURES WITH THE REPEATED-LOAD INDIRECT TENSILE TEST

From the review of previous efforts in characterizing and evaluating the behavior of asphalt mixtures it can be concluded that the indirect tensile test is most promising and favorite test. Comparisons of the results of this test with other available methods indicate that it provides higher testing repeatability and more reliable results, since it better represents the field conditions that the asphalt mixtures may be exposed to, in addition of being simple and quick to perform. Herein are described some of the most important results and conclusions regarding this test from previous investigation. In addition the relationships for evaluating mixture properties are presented.

Relationships of Asphalt Mixture Characteristics to Mixture Structural Properties Using the Indirect Tensile Test

Several studies emphasized the need for developing relationships between binder, aggregate and mixture characteristics to mixture structural properties. These relationships may be based on binder properties and aggregate characteristics, and/or design mixture properties, such as air voids, density, for estimating structural response parameters such as resilient modulus, tensile strength, stability, stiffness. Obviously these relationships are limited to the specific mixtures that were developed with and should calibrated when new type of mixtures are of interest.

One of the most comprehensive studies in evaluating these relationships was undertaken by Baladi (1988), where mixture properties from the Marshall and indirect tensile tests were related to mixture characteristics and testing variables. The study included Marshall test and the indirect static tensile test (INTT) using a standard Marshall load frame and deformation rate. The specimens were conditioned according to the standard Marshall method and tested dry at 9°C (40°F), 25°C (77°F) and 60°C (140°F). The study included indirect constant peak cycle testing (INCCL) using an MTS hydraulic system. The specimens were subjected to constant sustained load followed by a constant peak cyclic load of 226.2Kg (500 Ib). The specimens were subjected to a maximum of 500,000 cycles at a frequency of two cycles per second with a loading time of 0.1 second and a relaxation period of 0.4 second. A third testing method was used, the indirect variable peak cyclic load (INVCL), with the MTS hydraulic system. Basically, this last test is the same as the INCCL test with the difference that after the application of the sustained load, the specimen is subjected to 45.2Kg (100lb), 90.5Kg (200lb), and 226.2kg(500 Ib) peak cyclic loads, and each load applied for only 1000 cycles.

Different type of aggregates and asphalt cements were used in this testing. the three type of aggregates included crushed lime stone, rounded natural aggregate, and a mix of 50 % by weight of crushed lime stone and natural aggregate. Two different aggregate gradations were used and the mineral filler was fly ash. The study included three types of viscosity-graded asphalt contents, AC-10, AC-5, and AC-2.5.

Some of the results of this study are reported in Table 2.10, where the factors that influence mixture properties are shown with a number indicating the order of significance. The indirect tensile parameters were computed from the equations provided previously, while the Marshall equivalent stiffness was obtained from:

Table 2.10 Variables Affecting Mixture Properties (Baladi 1988)

	Specimen Variables				Test Variables			
	AV	*KV*	*ANG*	*GRAD*	*P/CL*	*TT*	R^2	*SE*
Marshall tests								
Stability (*S*)	2	3	4	–	–	1	0.99	0.06
Flow (*F*)	3	–	4	2	–	1	0.53	0.29
Equivalent stiffness (*ES*)	2	4	–	3	–	1	0.93	0.24
INTT								
INCS	2	4	3	–	–	1	0.99	0.08
INTS	2	4	3	–	–	1	0.99	0.08
DV	1	–	2	–	–	–	0.69	0.10
DH	1	2	3	–	–	–	1.00	0.00
INCCL + INVCL								
MR	2	5	4	–	3	1	0.99	0.03
E	2	5	4	6	3	1	0.99	0.03
*CD*1[a]	3	4	–	–	–	1		
*CD*2[a]	3	4	–	–	–	1		

NOTE: dashes = no relationship. *AV* = percentage of air voids (*AV* = 1, 2, 3, etc.); *KV* = kinematic viscosity (cSt); *ANG* = aggregate angularity; *GRAD* = aggregate gradations; *P/CL* = static/cyclic load; *TT* = test temperature; *INCS* = indirect compressive strength; *INTS* = indirect tensile strength; *DV* = indirect vertical deformation; *DH* = indirect horizontal deformation; *MR* = resilient modulus; *E* = total modulus; *CD*1 = cumulative total plastic deformation along the vertical diameter; and *CD*2 = cumulative total tensile plastic deformation along the horizontal diameter.

[a]These values are also functions of the number of load applications.

$$E_s = S / 2F_{0.5S}$$

where: E_s is the equivalent stiffness. Ib/in; S is the Marshall stability, Ib; and $F_{0.5S}$ represents the flow at half the value of the Marshall stability, in.

Based on the testing results, statistical analysis was conducted for relating Marshall stability, flow, equivalent stiffness, indirect compressive and tensile strengths, resilient modulus, and total modulus with mixture material characteristics and testing variables. Also relationships between the mixture property parameters were identified. These relationships are shown in Table 2.11. Some of the conclusion obtained in this study and these relationships are:

1. The effect of TT and KV on the INTT and INCCL results are almost the same
2. The effect of AV on INCS and INTS is the same, but higher than the effects on MR.
3. The effect of ANG on INCS and INTS are almost the same, but slightly higher than on MR.

In addition, it was observed that there is poor correlation between Marshall stability and MR indicating that the Marshall stability can not be used to accurately estimate the structural properties of a mixture. A good correlation was found between MR and INCS or INTS. Among the conclusions of this study Regarding the effectiveness of the testing methods the following conclusions were reported:

1. The resilient and total moduly of asphalt mixes can be expressed in terms of indirect compressive strength of the mixture, the testing temperature, and the magnitude of the applied cyclic load.

2. The Indirect tensile test can be used to obtain an asphalt mix design based on the structural properties of the mixture.

3. The maximum difference between the results of the three tests was only 7 %

Table 2.11. Relationships of Asphalt Mixture Characteristics and Mixture Properties

-Relation of Binder, Aggregate & Testing Characteristics to Mixture Properties

ln (S) = 10.319 - 0.01346 x TT - 0.255 x AV + 0.0006603 x KV + 0.02565 x ANG	(R^2 = 0.99; SE = 0.06)
ln (F) = 1 .588 - 0.00621 x TT + 0.3342 x GRAD - 0.07033 x AV + 0.08257 x ANG	(R^2 = 0.58; SE = 0.29)
ln (ES) = 12.684 - 0.01747 x TT - 0.1355 x AV - 0.3307 x GRAD + 0.002020 x KV	(R^2 = 0.93; SE = 0.24)
ln (INCS) = 9.1350 - 0.03369 x TT - 0.2604 x AV + 0.05223 x ANG + 0.0007399 x KV	(R^2 = 0.99; SE = 0.08)
ln (INTS) = 8.01589 - 0.03363 x TT - 0.2605 x AV + 0.0509 x ANG + 0.0007676 x KV	(R^2 = 0.99; SE = 0.08)
ln (MR) = 16.092 - 0.03658 x TT - 0.1401 x AV - 0.0003409 x CL + 0.04353 x ANG +0.0008793 x KV	(R^2 = 0.99; SE = 0.03)
ln (E) = 16.385 - 0.04529 x TT - 0.1549 x AV - 0.0003339 x CL + 0.04258 x ANG +0.0008364 x KV	(R^2 = 0.99; SE = 0.03)

- Relationships of Poisson's Ration and Resilient and total Moduly with Mixture Properties

ln(MR)=3.408+ln(ES)-0.01911xTT-0.0046 x AV-0.3307 x GRAD- 0.0003409 x CL+0.0011407 x KV+0.04353 x ANG	(R^2=0.997;SE = 0.082)
ln (MR) = 7.1949 + 1.01341 x ln (INCS) - 0.0003409 x CL	(R^2= 0.974;SE=0.220)
ln (MR) = 8.3145 + 1.01511 x ln (INTS) - 0.0003409 x CL	(R^2=0.974;SE= 0.220)
ln (MR) = 6.1776 + 1.08108 x ln (INCS) + 0.14145 x AV - 0.0003409 x CL	(R^2=0.996;SE= 0.085)
ln (MR) = 7.3667 + 1.08335 x ln (INT S) + 0.14218 x AV - 0.0003409 x CL	(R^2=0.996;SE= 0.083)
ln (E) = 7.0327 + 1.0205 x ln (INCS) - 0.1108 x AV - 0.0003339 x CL	(R^2=0.996;SE= 0.080)
ln (E) = 8.1552 + 1.0227 x ln (INTS) - 0.1153 x AV - 0.0003339 x CL	(R^2=0.996;SE= 0.078)
ln (PT) = -0.43228 - 0.01940 x TT - 0.06329 x AV - 0.001332 x KV + 0.0001236 x CL	(R^2=0.913;SE= 0.108)
ln (PR) = - 1.730 - 0.04243 x AV - 0.000885 x KV - 0.004662 x ln (N)+ 0.0004489 x TT	(R^2=0.720;SE= 0.045)

Note: ln = Natural log; S = Marshall stability, Ib: F = Marshall flow, 1/100 in; ES = Equivalent stiffness, Ib/in; INCS = Indirect compressive strength, psi; INTS = Indirect tensile strength, psi; MR = Resilient modulus, psi; E = Total modulus, psi; AV = Percentage of air voids; TT= Test temperature, oF; ANG = Aggregate angularity; GRAD = Aggregate gradation; KV= Kinematic viscosity; CL = Cyclic load, Ib; P = maximum applied load, Ib; R^2 = Coefficient of correlation; SE = Standard error; PT = Total Poisson's ratio; PR = Resilient Poisson's ratio.

Use of Indirect Tensile Test for Fatigue Life Evaluation of Asphalt Mixes

The Diametral repeated-load test was used to evaluate the fatigue characteristics of asphalt mixtures as well (Kim 1991). For such evaluation an MTS setup was used with extensometers as shown in Figure 2.14. In this experiment two asphalt cements and two aggregates were used, see Table 2.12. The mixing temperature for each one of the bitumen were 140°C (284°F) and 149°C (300°F) respectively. The aggregate and asphalt cement was mixed for 4 minutes and placed in the oven at 60°C (140°F) for 15 hours. The specimens were compacted at 116°C (240°F) using the gyratory testing machine for producing 100 mm (4") diameter by 62.5mm (2.5") height specimens at two levels of air voids, 4% and 8 percent, and according to ASTM D1188. The specimens were tested with a haversine load of 0.1 second load duration and 0.5 second rest period, till failure. The stress amplitude was kept constant throughout testing and the horizontal and vertical deformations were recorded after the first 200 cycles. The failure criteria, affecting the fatigue life, has been defined as the 0.28 to 0.36 inch permanent horizontal deformation (Scholz 1991). The failure criteria was set at the 2.5mm (0.1 in). For evaluating the fatigue life of the mixtures the following model was used:

$$Nf = K1\ (1/\varepsilon)^{K2}$$

where: Nf is the number of cycles to failure; ε is the tensile strain in asphalt concrete, in/in; and K1 and K2 are regression constants. Table 2.13 provides the calculated values of these constants. Examples of the fatigue results on these mixtures are shown in Figure 2.15.

Evaluation of Marshall and Hveem Mix Design Procedures Using The Indirect Tensile Test

The indirect tensile test has also been used in evaluating the standard mix design methodologies used by DOTs. Al-Abdul Wahhab (1991), evaluated the Marshall and Hveem mix design procedures using the indirect tensile test. In this study five mixes were used with the materials and gradations shown in Tables 2.14 and 2.15. The mixtures were designed with the Marshall and Hveem mix design methods and the results are shown in Tables 2.16 and 2.17. Then the repeated load indirect tensile test

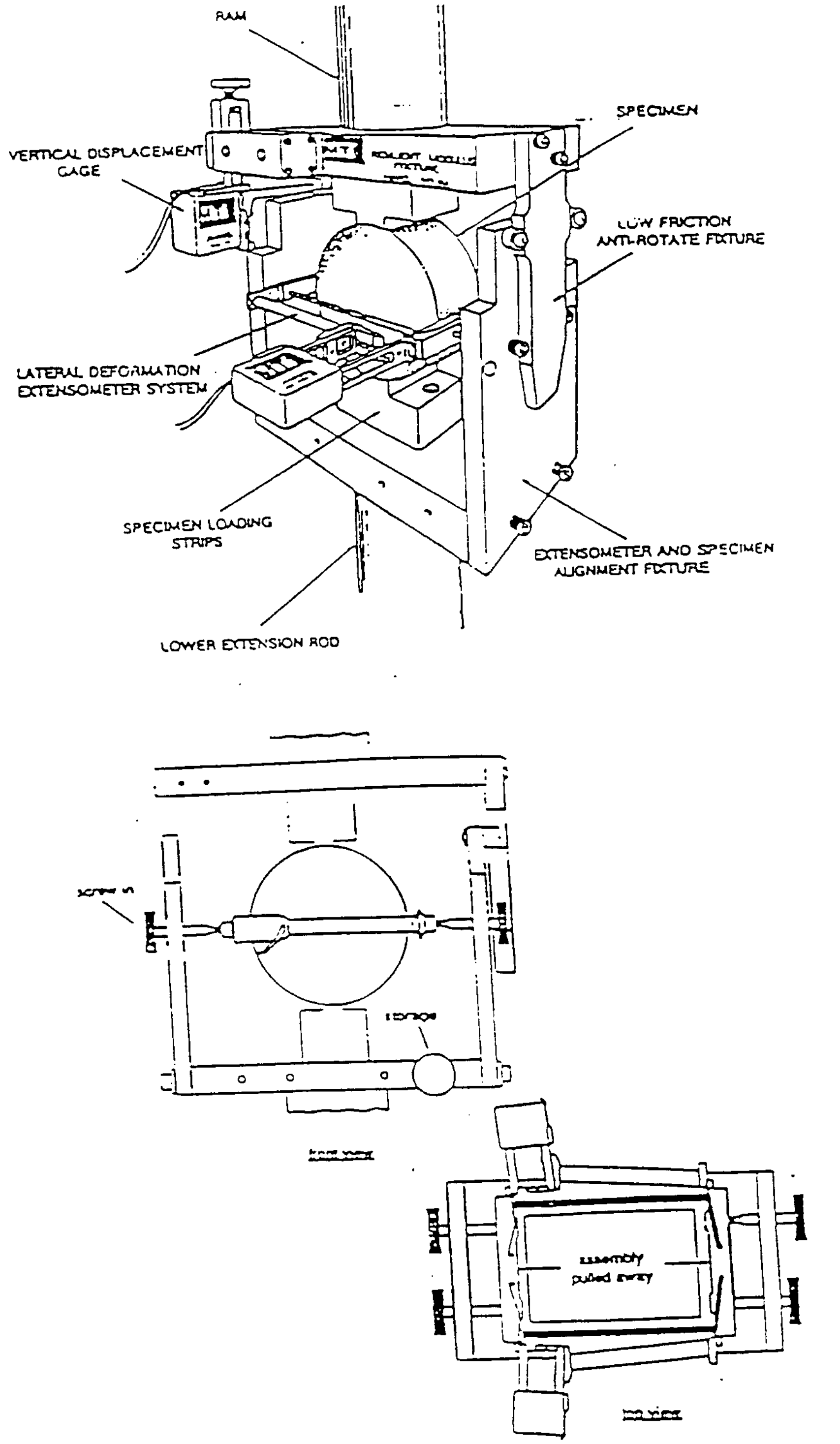


Figure 2.14 Extensometers Setup for Fatigue Testing (Kim 1991)

Table 2.12 Experimental Design for Diametral Fatigue Life (Kim 1991)

Asphalt Type	Asphalt Content	Air Voids	Temperature
0	0	0	0
1	0	0	0
0	1	0	0
1	1	0	0
0	0	1	0
1	0	1	0
0	1	1	0
1	1	1	0
0	0	0	1
1	0	0	1
0	1	0	1
1	1	0	1
0	0	1	1
1	0	1	1
0	1	1	1
1	1	1	1

Asphalt Type: 0 = AAK-1; 1 = AAG-1

Asphalt Content: 0 = Low; 1 = High

Air Voids: 0 = 4 ± 0.5%; 1 = 8 ± 0.5%

Temperature: 0 = 32°F (0°C); 1 = 68°F (20°C)

Table 2.13 Summary of K1 and K2 (Kim 1991)

Sample ID abcd*	K_1	K_2
0000	3.00 x 10[illegible]	3.484
0001	1.90 x 10[illegible]	2.646
0010	3.62 x 10[illegible]	4.184
0011	1.00 x 10[illegible]	3.145
0100	1.01 x 10[illegible]	5.208
0101	7.71 x 10[illegible]	3.546
0110	2.07 x 10[illegible]	4.808
0111	1.40 x 10[illegible]	3.125
1000	5.40 x 10[illegible]	4.405
1001	1.64 x 10[illegible]	3.704
1010	7.51 x 10[illegible]	3.802
1011	2.85 x 10[illegible]	2.564
1100	7.95 x 10[illegible]	5.952
1101	5.76 x 10[illegible]	3.559
1110	2.72 x 10[illegible]	6.579
1111	8.70 x 10[illegible]	3.436

Note: *a = Asphalt Type; b = Asphalt Content; c = Air Voids; d = Temperature

Levels of each factor are presented in Table 1.

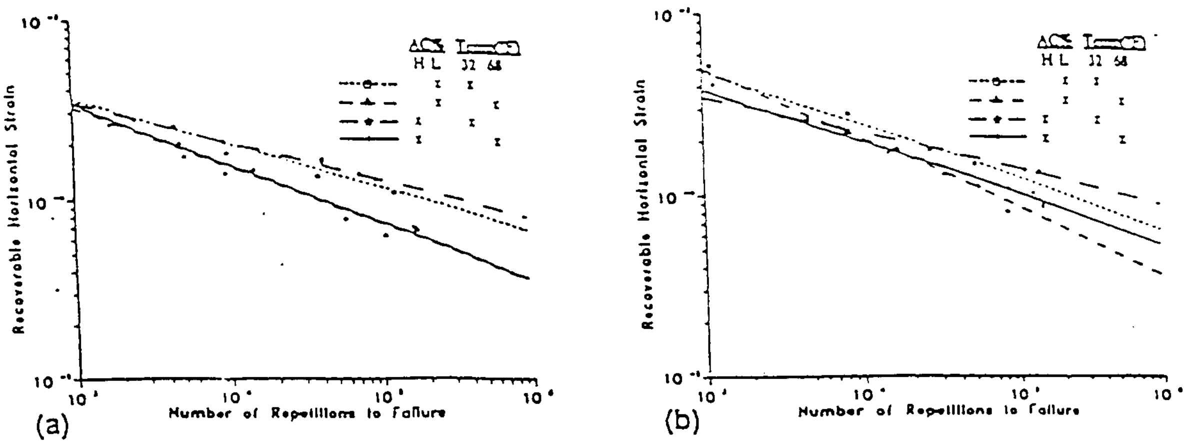


Figure 2.15 Diametral Fatigue Results for Mixtures with High (a) and Low (b) Air void Content (Kim 1991)

Table 2.14 Aggregate Gradations for Wearing and Base Courses (Al-Abdul Wahhab 1991)

Sieve Size	Wearing Course (W) Gradation Designation			Base Course (B) Gradation Designation	
	W-1	W-2	W-3	B-1	B-2
1½ inch	-	-	-	100	100
1 inch	-	-	-	100	90
3/4 inch	100	100	-	90	80
1/2 inch	87.5	90	100	-	-
3/8 inch	-	-	90	70	65
No.4	55	60	65	55	53
No.10	38.5	39.5	39.5	40	40.5
No.40	26	21	21	23.5	21.5
No.80	13	14	14	-	-
No.200	6	7	7	6.5	6.5

Saudi Arabian Ministry of Communications Specification

Table 2.15 Aggregate and Asphalt Binder Properties (Al-Abdul Wahhab 1991)

Material	Physical Properties	Mix Designation					Saudi Arabian Ministry of Communications Specification Limits
		W1	W2	W3	B1	B2	
Aggregate	L.A. Abrasion	22	22	22	23	23	30
	Soundness 5 cycles - Coarse aggregate	2.86	2.86	2.63	3.2	3.4	10 max
	- Fine aggregate	1.96	1.96	1.95	2.2	2.3	10 max
	Bulk Specific Gravity (Sat. surface dry)	2.597	2.611	2.592	2.598	2.60	
	Water Absorption - Coarse aggregate	2.625	2.63	2.64	2.6	2.56	4 max
	- Fine aggregate	3.83	3.82	3.83	3.62	3.61	
Asphalt	Flash Point, Cleveland open cup	628°F					450 minimum
	Penetration, 77°F, 100 gm, 5 sec.	61					60-70
	Specific Gravity, 25°C	1.043					—
	Solubility in Trichloro-ethylene	99.9					99.9 min

Table 2.16 Marshall Properties at Optimum AC Content (Al-Abdul Wahhab 1991)

Mix Designation	Optimum Asphalt Content	1/2 hr Stability kgs	Flow 0.25 mm	Air Voids %	VMA %	24 hrs Stability kgs	Strength Index %
W-1	5.5	2090	15.3	4.2	11.5	2060	98.6
W-2	5.6	2060	14.5	4.3	11.8	1953	94.8
W-3	5.9	1940	16.9	3.5	12.1	1832	94.4
B-1	5.2	2307	17.5	3.9	12.3	1960	85.0
B-2	4.9	2120	14.8	4.1	11.5	2040	96.2
MOC Specification		Minimum 700	10-16	4-7			Minimum 80

Table 2.17 Hveem Properties at Optimum AC Content (Al-Abdul Wahhab 1991)

Mix Designation	Optimum Asphalt	Hveem Stability	Air Voids %
W-1	5.0	47	4.3
W-2	5.0	52	4.2
W-3	5.5	37	4.0
B-1	4.5	49	4.4
B-2	4.5	45	4.6
MOC Specifications		40	4-7%

was used for evaluating the resilient modulus of the mixtures. Testing was conducted at 51°C (122°F) and with a load frequency of 1 Hz, loading time of 0.1 second, and a static load of 4.5Kg (10 Ib) for holding the specimen in place. The maximum applied load, (ranges from 10 to 30% of the static indirect tensile strength)and the horizontal elastic tensile deformation were recorded to determine the MR values according to:

$$MR\ (MPa) = 1000\ P\ (0.9974\ U + 0.2692) / (h\ D)$$

where: P is the applied load, KN; h represents the specimen thickness, mm; D is the recoverable horizontal deformation across the sample, mm; and U is the Poisson's ratio. The static split tensile strength was evaluated with the static indirect tensile test with a loading rate of 2 in/minute and a testing temperature of 50 oC. The split tensile strength was calculated according to:

$$ST = 2\ Pmax / \Pi\ h\ D$$

where: ST is the split tensile strength, psi; Pmax is the Load at failure, Ib, D is the sample diameter; and h is the sample thickness, in.

The results indicated that the optimum asphalt contents according to the Marshall mix design method were about 0.5 percent higher than those of the Hveem method, see Tables 2.16 and 2.17. Also, differences in bulk density, air void content, resilient modulus, stiffness, creep compliance, and split tensile strength were observed. The resilient modulus and the split tensile strength of the mixtures designed according to Hveem and Marshall are shown in Figures 2.16 and 2.17. As it can be seen from these results mixtures designed with the Hveem method provide higher values of MR and tensile strength, thus, identifying the deficiency of the Marshall method.

The effectiveness of the indirect tensile test in evaluating tender mixtures, versus the Marshall mix design was investigated by Khosla and Omer (1985). As it can be seen from Figure 2.18, Marshall stability did not correlate to the mixture tenderness, while good correlation between splitting tensile

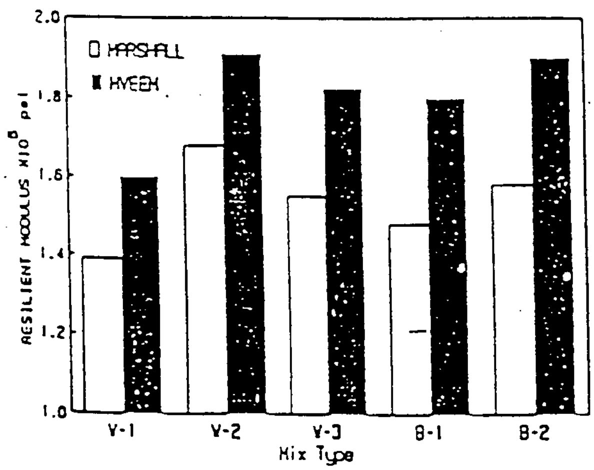


Figure 2.16 Resilient Modulus at Marshall and Hveem Optimum Mix Design Tested at 50°C (Al-Abdul Wahhab 1991)

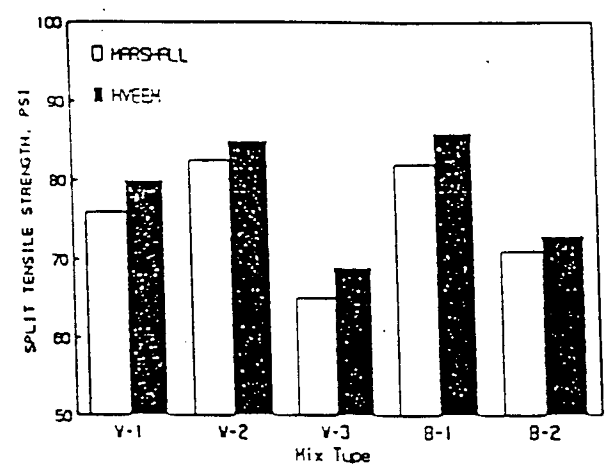


Figure 2.17 Split Tensile Strength at Marshall and Hveem Optimum Mix Design Tested at 50°C (Al-Abdul Wahhab 1991)

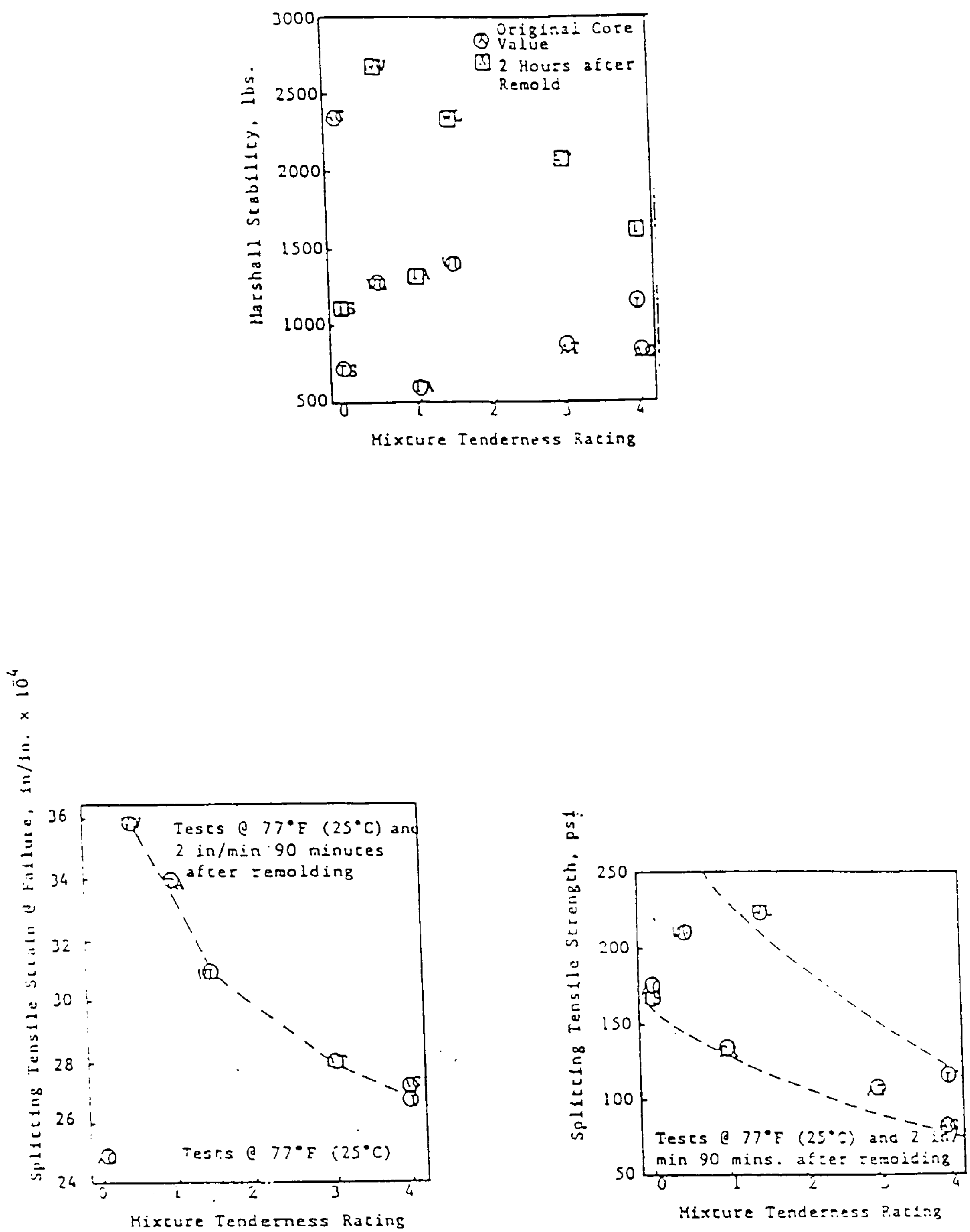


Figure 2.18 Relationship Between Mixture Tenderness with Marshall stability, Tensile Strain and Strength (Khosla 1985).

strain and strength with mixture tenderness were found. indicating that ITT better represent mixture characteristics.

Improvement of Indirect Tensile Test Device and Creep Testing

Some investigators examined the development of an improved indirect tension apparatus. Mohammed et al (1992), investigated the effectiveness of an improved apparatus, Louisiana Modified (LM), designed according to recommendations of Federal Highway Administration, and compared to the previous version of the apparatus. Louisiana Transportation Center device (LTRC). The modified device is shown in Figure 2.19.

Evaluation of the new device was conducted by preparing 168 specimens, of 100 mm (4 in) diameter by 62.5mm (2.5 in) height , and tested with the LTRC and LM devices. The specimens represent a typical Type 1 Louisiana mixture with 65 percent crushed gravel, by weight, 25 percent coarse sand, 10 percent fine sand and an AC-30. The specimens were prepared using 75 blows per face of the Marshall hammer and tested with the static indirect tensile test at 4°C (40°F) and 25°C (77°F) according to AASHTO T245-82. The test specimens were loaded to failure at a deformation rate of 2 in/minute and the load to failure and the total horizontal and vertical deformations were recorded. In addition, the repeated load ITT was conducted at 4°C (40°F) and 25°C (77°F) and 40°C (104°F) according to ASTM D4123. During this test the samples were seated with a sustained load of 22.6kg (50 Ib) and tested with a cyclic haversine load with peak load equal to 10 percent of the indirect tensile strength. Conditioning of the sample was conducted with the sustained load until the deformation rate of the specimen was essentially constant. The load frequency was 2 Hz with 0.1 second loading time and 0.4 second relaxation period, and the data acquisition system rate was 500 Hz. The response curves (load, vertical deformation, and horizontal deformation) over two cycles were digitized. The data from these curves were then scanned to determine the instantaneous and total recoverable horizontal and vertical deformation. Data associated with the beginning of the relaxation period were used to compute instantaneous properties. Values associated with the end of the relaxation period were used to compute total properties. Each specimen was tested at the three temperatures starting with the

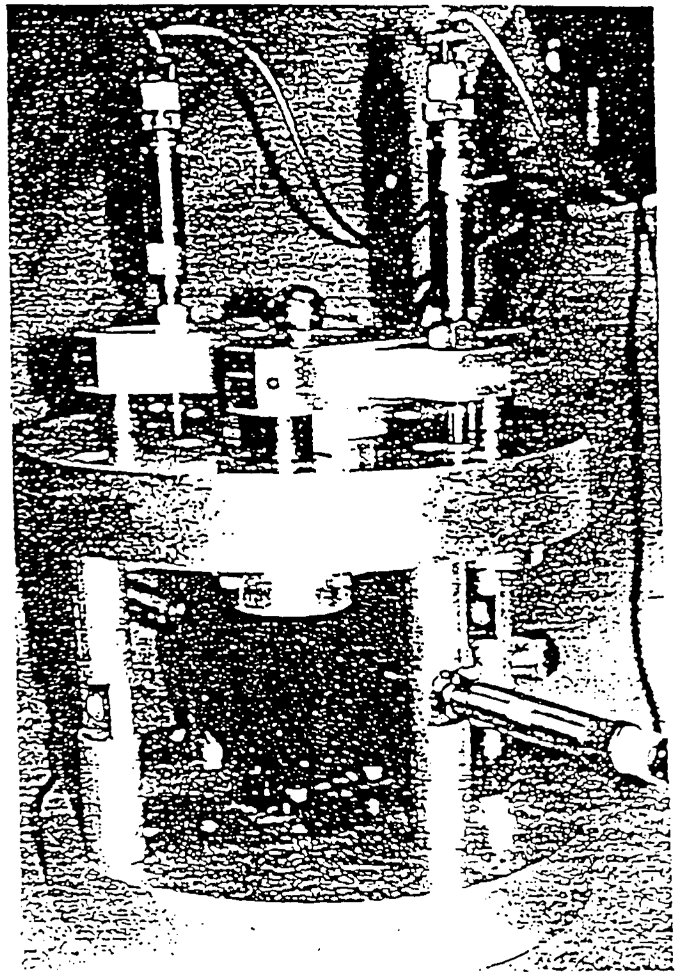


Figure 2.19 Louisiana Modified, LM, Apparatus (Mohammad 1992)

lowest temperature to minimize permanent damage to the sample. At each temperature, the sample was tested twice; after the first test, the sample was rotated approximately 90 degrees and the test was repeated.

Creep tests were also conducted at 25°C (77°F) using a ramp of load, (i.e., a load changing linearly increasing with time) of maximum load equal to 113.1kg (250 Ib) applied as quickly as possible, and using the stress controlled mode of the MTS machine. The load and vertical and horizontal deformations were monitored continuously with the data acquisition system. The test was terminated either after 60 minutes of load duration or at failure. The mean horizontal deformations and the mean vertical deformations were computed for each set of specimens. The creep modulus was computed from the measured deformations as follows:

$$S(t)= P_o(\mu + 0.27)/ t\, \delta H(t)$$

where: S(t) is the creep modulus at time t; Po is the applied vertical load; μ is the Poisson's ratio; t is the sample thickness; and $\delta H(t)$ represents the horizontal deformation at time t.

Table 2.18 presents a comparison of the test results for the indirect tensile strength test at two different testing temperatures and for the two devices, LM and LTRC. As it can be seen from the Table both devices meet the ASTM C670 repeatability requirements. Overall, the repeatability of the indirect tensile test with such devices is high, with a range of 2 to 9 percent. The repeatability of the LM device was relatively higher in relation to the previous version of the device. In addition, the difference in test results due to different operators was not significant, Table 2.19, (the same letter on columns indicates that no significant difference exist between row variables). A comparison between the values of the indirect tensile strength obtained with the two devices at the two testing temperature indicated that the LTRC device provides higher values at 4°C(40°F) than LM device while no significant differences were observed at 25°C (77°F), see Table 2.20.

Table 2.18 Comparison of LM and LTRC Indirect Tensile Devices Based on Present Serviceability Index (Mohammad 1992)

Test Device	LM		LTRC	
Temperature (°F)	40	77	40	77
Operator 1	265	78	328	77
	260	75	318	68
	276	77	311	76
	268	79	307	83
	260	78	314	74
	275	76	310	77
Mean	267	77	315	76
STD	6	1	7	4
(%) CV	2	2	2	6
Repeatability	Y	Y	Y	Y
Operator 2	291	72	374	57
	278	74	347	72
	249	75	321	72
	267	68	363	74
	271	76	313	77
	262	80	295	70
Mean	270	74	336	70
STD	13	4	28	6
(%)CV	5	5	8	9
Repeatability	Y	Y	Y	Y

STD : Standard Deviation

CV : Coefficient of Variation

Y : Indicates Test is Repeatable as per ASTM C670.

Table 2.19 Effect of Operator on Mean Values of Indirect Tensile Strength Values for LM and LTRC (Mohammad 1992)

Test Device	LM		LTRC	
Temperature (°F)	40	77	40	77
Operator 1	A	A	A	A
Operator 2	A	A	A	A

Table 2.20 Effect of Testing Device and Testing Temperature on Mean Values of Indirect Tensile Strength (Mohammad 1992)

Mechanical Property	Test Device	Temperature (°F)	
		40	77
Indirect Tensile Strength	LTRC	A	A
	LM	B	A

Similar evaluation was conducted from the results of the diametral resilient modulus testing. Table 2.21 indicates that the two devices provide different response at the low testing temperature. It may be concluded that overall the effect of operator was significant in the evaluation of the instantaneous and total resilient modulus and Poisson's ration, see Table 2.22. The means of the diametral resilient modulus obtained with the two devices, Table 2.23, were found to be significantly different. Finally the effect of the measuring system used in evaluating the diametral resilient modulus and Poisson's ratio, Table 2.24, was significant only for the evaluation of the later parameter.

The creep modulus was calculated using either a computed Poisson's ratio from the measured horizontal and vertical deformations or with an assumed value of 0.35. The indirect tension creep modulus was statistically analyzed at time intervals of 5, 10, 100, and 500 seconds. see Table 2.25. The test results were repeatable at those time intervals and the operator did not have any effect on the measurements. Similarly the effect of test device was found to be insignificant, with the exception of 500 seconds interval, for the case of calculated Poisson's ration, Table 2.26. When the Poisson's ration was assumed the devices provided different results.

Finally, the potential effectiveness of the use of indirect tensile test for creep testing as compared to the axial compression test was investigated by Harold (1989) based on the work of Khosla and Omer (1985). The results of the study indicated that meaningful creep behavior information can be obtained using the indirect tensile test, Figure 2.20. As it can be seen from this Figure, for short loading duration, no significant difference was found between the direct compression and indirect tension stiffness evaluation whereas, for long load duration, a significant difference was observed. Since, long load duration are typical of thermal loading conditions and short duration are typical of wheel loading conditions its use in evaluating creep potential of asphalt mixture under highway loading is possible.

Table 2.21 Effect of LM and LTRC on Mean Values of Indirect Tensile Strength Values (Mohammad 1992)

Operator	1		2	
Temperature (°F)	40	77	40	77
LM Device	A	A	A	A
LTRC Device	B	A	B	A

Table 2.22 Effect of Operator on Mean Values of Diametral resilient Modulus (Mohammad 1992)

DEVICE	LM											
Measurement Device	Extensometer				LVDT-N				LVDT-O			
Mechanical Properties	MRI	MRT	MUI	MUT	MRI	MRT	MUI	MUT	MRI	MRT	MUI	MUT
Operator 1	A	A	A	A	A	A	A	A	A	A	A	A
Operator 2	A	A	A	A	A	A	A	A	A	A	A	A

DEVICE	LTRC							
Measurement Device	LVDT-N				LVDT-O			
Mechanical Properties	MRI	MRT	MUI	MUT	MRI	MRT	MUI	MUT
Operator 1	A	A	A	A	A	A	A	A
Operator 2	A	A	A	A	B	A	A	A

MRI = Instantaneous Resilient Modulus
MRT = Total Resilient Modulus
MUI = Instantaneous Poisson's Ratio
MUT = Total Poisson's Ratio

Table 2.23 Effect of Testing Device and Measuring System on Mean Values of Diametral Resilient Modulus (Mohammad 1992)

Measurement System	LVDT - N				LVDT - O			
Mechanical Properties	MRI	MRT	MUI	MUT	MRI	MRT	MUI	MUT
LM	A	A	A	A	A	A	A	A
LTRC	B	B	B	B	B	B	B	B

MRI = Instantaneous Resilient Modulus
MRT = Total Resilient Modulus
MUI = Instantaneous Poisson's Ratio
MUT = Total Poisson's Ratio

Table 2.24 Effect of Measuring System on Mean Values of Diametral Resilient Modulus (Mohammad 1992)

Test Device	LM				LTRC			
Mechanical Properties	MRI	MRT	MUI	MUT	MRI	MRT	MUI	MUT
LVDT – C	A	A	B	B	A	A	B	A
LVDT – H	A	A	A	A	A	A	A	A
Extensometer	A	A	A	B	N/A	N/A	N/A	N/A

MRI = Instantaneous Resilient Modulus
MRT = Total Resilient Modulus
MUI = Instantaneous Poisson's Ratio
MUT = Total Poisson's Ratio

Table 2.25 Effect of Operator Error on Mean Values of Creep Modulus (Mohammad 1992)

Creep Modulus	Calculated Poisson's Ratio				Assumed Poisson's Ratio, MU = 0.35			
Time (secs)	5	10	100	500	5	10	100	500
Operator 1	A	A	A	A	A	A	A	A
Operator 2	A	A	A	A	A	A	A	A

Table 2.26 Effect of Testing Device on Mean Values of Creep Modulus (Mohammad 1992)

Mechanical Properties	Test Device	Time (seconds)			
		5	10	100	500
Calculated Creep Modulus	LTRC	A	A	A	A
	LM	A	A	A	B
Creep Modulus with Assumed MU = 0.35	LTRC	A	A	A	A
	LM	B	B	B	B

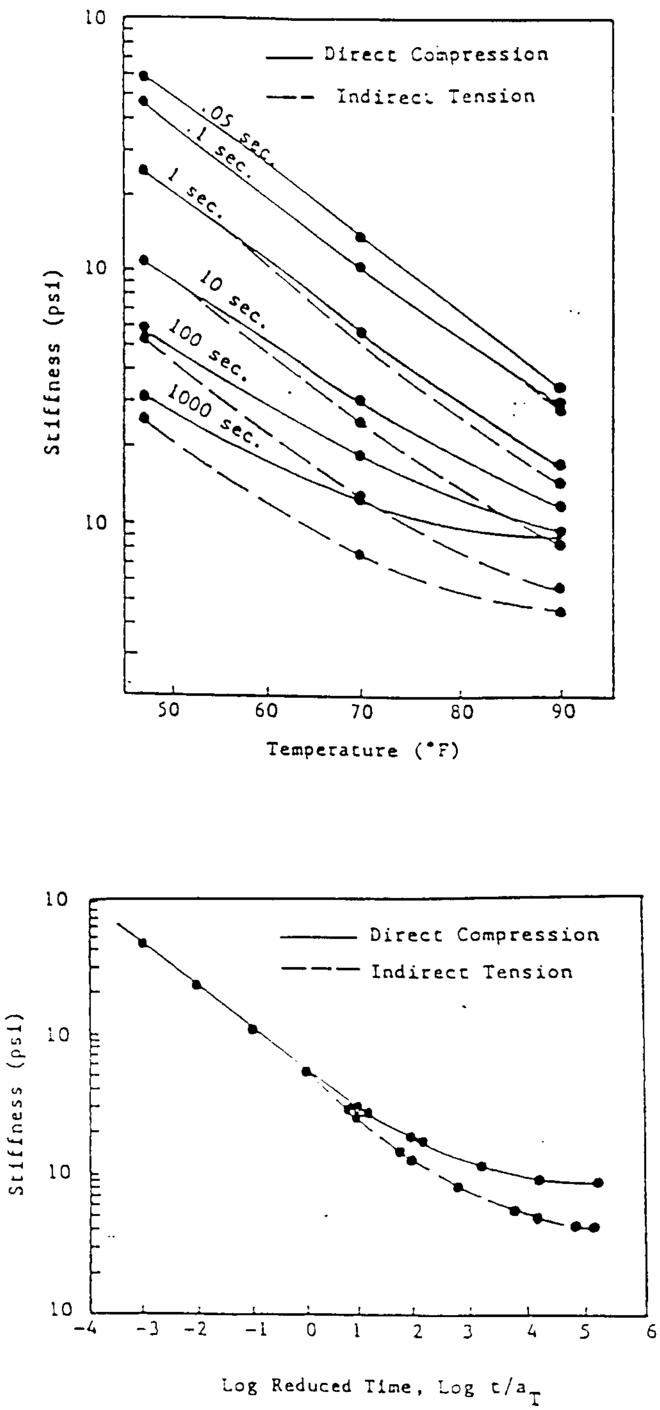


Figure 2.20 Stiffness Values for US-64 (Layer 1) Using Different Test Methods (Khosla 1985)

SELECTION OF REPEATED-LOAD INDIRECT TENSILE TEST FOR ASPHALT-RUBBER MIXTURE CHARACTERIZATION

It appears that the repeated load indirect tensile represents one of the best testing methods for characterizing asphalt mixtures and thus could be used in the design and evaluation of asphalt rubber mixtures. The ITT test uses similar specimens to the standard Marshall method, (i.e., same specimen dimensions with some modifications in mix preparation and compaction), describes better mixture behavior than the standard mix design methods, and provides the possibility of evaluating creep/rutting, fatigue, and low temperature cracking, the three typical modes of asphalt mixture failure, that could eventually be incorporated as criteria in an integrated mix design methodology.

The variety of recommended testing conditions proposed in ASTM D4123, have been evaluated by several studies, (Boudreau 1992, Almudalteem1991, Kim1991). Some of the best results, in terms of testing repeatability and the ability of the test to better describe mixture behavior, were obtained with a load duration of 0.1 second, load frequency of 0.33 Hz, testing temperature at 15°C (60°F), an induced strain of 50-75 microstrain, and a load magnitude equal to 50% of the static indirect tensile strength. Typically, 50-200 load repetitions are applied before measurements are taken, and a 4.5kg (10lb)- 22.62Kg (50 Ib) static load is being applied to keep specimen in place. The failure criteria for the test is being commonly selected at 0.1" total horizontal deformation. The selection of these testing conditions often depends on the characteristics of the mixtures, and thus a preliminary evaluation will identify the best values.

Overall, some of the advantages reported on previous studies using the ITT test include: the indirect tensile test appears to be practical and rather simple test method for characterizing the asphalt concrete properties, and failure caused by tensile stresses; the testing specimens are easy to fabricate; it is possible to conduct the test with Marshall and an MTS apparatus for static and repeated load respectively; no high rate of load is required for the test; the test provides the opportunity to evaluate the modulus of elasticity, Poisson's ratio, fatigue and permanent deformation characteristics, and provides the opportunity to quantify low temperature cracking and moisture susceptibility; the test can be conducted under a variety of test conditions, including temperature, moisture conditions, and loading characteristics, for determining the engineering properties for elastic or viscoelastic analyses; the variation in testing results is rather low compared to other types of tests; the test results are not affected by surface condition; failure occurs in an area of a relatively uniform tensile stress

FACTORS AFFECTING ASPHALT-RUBBER MIXTURE PROPERTIES

Dry Process

Several studies investigated the factors affecting asphalt rubber mixture properties. A variety of material considerations and testing conditions have been used in evaluating such effects on asphalt rubber mixtures. the evaluation of these effect will aid in the development of the mix design methodology and the design criteria. Thus, some of the main results from previous studies on asphalt rubber mixtures are being reviewed herein.

The effects of mix variations on the properties of rubber-modified mixes were looked by Takallou (1980). The effects of aggregate characteristics and gradation, rubber content, air void content, mixing temperature, curing time and surcharge load were investigated in this study. The recommended aggregate specifications for asphalt rubber mixtures are shown in Table 2.27. For the specific study conditions and materials to be used in the dry process, a "gap" in the gradation curve of the aggregate, primary in the 1/8 to 1/4 in size range was recommended, as illustrated in Figure 2.21 and Table 2.28, for providing space for the rubber particles, Table 2.29. The asphalt cement used in this study was an AC-5. Twenty different mix combinations were prepared and tested for diametral modulus according to ASTM D4123, and fatigue at -6°C and + 10°C, see Table 2.30. The mixture variables included two air void contents, two rubber contents, three rubber gradations, two mix temperatures and curing times, and the use of surcharge load, Table 2.31.

Table 2.27 Recommended Specifications for Rubber-Asphalt Mixtures Depending on Traffic Levels (Takallou 1980)

	Mix Designation		
	A	B	C
Average daily traffic	2,500	2,500–10,000	10,000
Minimum thickness (in.)	1.0	1.5	1.75
Sieve size (% aggregate passing)			
3/4 in.			100
5/8 in.		100	
1/2 in.			
3/8 in.	100	60–80	50–62
1/4 in.	60–80	30–44	30–44
No. 10	23–38	19–32	19–32
No. 30	15–27	13–25	12–23
No. 200	8–12	8–12	7–11
1/4-in. to No. 10 size fraction		12 max	12 max
Preliminary mix design criteria			
Rubber, % of total mix by			
Weight	3.0	3.0	3.0
Volume (approx.)	6.7	6.7	6.7
Asphalt (% of total mix by weight)	8–9.5	7.5–9.0	7.5–9.0
Maximum voids (%)	2.0	2.0	4.0

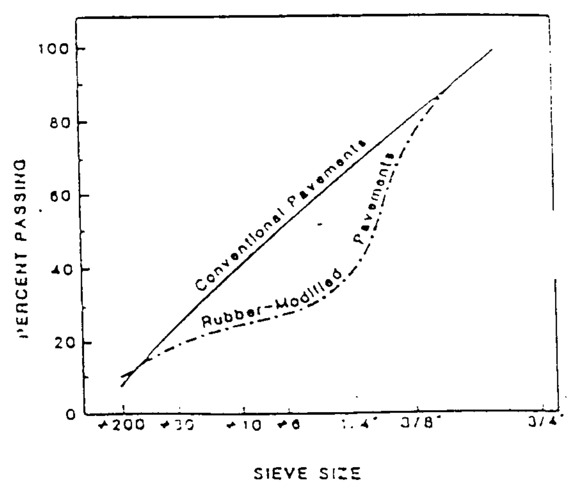


Figure 2.21 Aggregate Gradation for Conventional and Rubber Modified Mixtures (Takallou 1980)

Table 2.28 Aggregate Gradation for Medium Traffic Level Mixtures (B) (Takallou 1980)

Sieve Size	Percentage Passing: Gap Graded	Percentage Passing: Dense Graded	Specification for B Mix
3/4 in.		10	
5/8 in.	100		100
3/8 in.	70	76	60–80
1/4 in.	37		30–44
No. 4		55	
No. 10	[illegible]	36	19–32
No. 30	[illegible]		13–25
No. 40		[illegible]	
No. 200	[illegible]	7	8–12

Table 2.29 Rubber Particle Size Specification (Takallou 1980)

Sieve Size	Percentage Passing: Coarse Rubber	Percentage Passing: Fine Rubber	Percentage Passing: 80/20 Rubber Blend[a]
1/4 in.	100		100
No. 4	70–[illegible]		76–92
No. 10	10–20	100	28–36
No. 20	0–5	50–100	10–24

[a] The 80/20 is 80 percent coarse and 20 percent fine rubber in combination.

Table 2.30 Characteristics of Rubber-Asphalt Mixtures Specification (Takallou 1980)

Table (7) Specimen Identification
(After Takallou 1980)

Specimen	Rubber Content (%)	Rubber Blend (% fine/ % coarse)	Mixing/ Compaction Temperature (°F)	Asphalt Content (%)	Aggregate Gradation	Cure Time (hr)	Surcharge (lb)
A	3	80/20	375/265	9.3	Gap	0	0
B	3	80/20	375/265	9.3	Gap	2	0
C	3	80/20	375/265	9.3	Gap	0	5
D	3	80/20	425/265	9.3	Gap	0	0
E	3	80/20	425/265	9.3	Gap	2	0
F	3	80/20	425/265	9.3	Gap	0	5
G	3	80/20	375/210	9.3	Gap	0	0
H	3	60/40	375/265	7.5	Gap	0	0
I	3	0/100	375/265	7.5	Gap	0	0
J	3	80/20	425/210	9.3	Gap	0	0
K	2	80/20	375/265	8.0	Gap	0	0
L	2	60/40	375/265	7.2	Gap	0	0
M	2	0/100	375/265	7.0	Gap	0	0
N	3	80/20	375/265	7.5	Dense	0	0
O	3	80/20	375/265	7.5	Dense	2	0
P	3	80/20	375/265	7.5	Dense	0	5
Q	3	80/20	425/265	7.5	Dense	0	0
R	3	80/20	425/265	7.5	Dense	0	0
S	3	80/20	375/210	7.5	Dense	0	0
T	0	No rubber	375/265	5.5	Dense	0	0
U	3	0/100	375/265	7.0	Dense	0	0

Table 2.31 Sample Preparation Characteristics Specification (Takallou 1980)

Variables	Level of Treatment
Air Voids, %	2, 4,
Rubber Content, %	2, 3
Rubber Gradation (Coarse/Fine)	Coarse (80/20), Medium (60/40), Fine (0/100)
Mix/Compaction Treatment, °F	375/265, 425/265
Mix Curing at 375°F and 425°F	0, 2 hrs
Aggregate Gradation	gap-graded, dense-graded
Surcharge	0, 5 lb

The mixtures were designed with the Marshall method and the resilient modulus and fatigue life evaluation was conducted at the optimum binder level. The resilient modulus was calculated according to:

$$Mr = f(u + 0.2734) / t(dh)$$

where: Mr is the resilient modulus, psi; f represents the dynamic load, Ib; u is the Poisson's ratio in this case assumed equal to 0.4; t is the specimen thickness, in; and dh is the total elastic horizontal deformation, in. The fatigue equation was given by:

$$Nf = a\,(1 / \varepsilon)^{b}$$

where: Nf is the number of repetitions to failure; a is the antilog of the intercept of the logarithmic relationship; ε is the initial tensile strain, in/in; and b is the slope of the logarithmic relationship between fatigue life and initial strain.

The laboratory mix design results are shown in Table 2.32. As it can be seen from this Table, the asphalt content required to reach a certain minimum voids level for rubber modified mixes depends on aggregate and rubber gradation and content. Typically, the coarser the rubber more binder reach mixtures with lower stability and higher flow are obtained. Similarly, higher the rubber content higher the asphalt binder and lower stability is observed. The modulus and fatigue results are shown in Table 2.33 while the effects of aggregate gradation, rubber gradation and content, curing time and mixing temperature, and surcharge weight on resilient modulus and fatigue life are shown in Figures 2.22 through 2.28. As it can be seen from these Figures, the use of surcharge did not have any impact on resilient modulus, while it improve the fatigue life of the gap graded mixtures. Curing time significantly increased the resilient modulus of dense graded mixtures while a small decrease in fatigue life was observed for both gap graded and dense graded mixtures. The use of finer rubber in gap graded mixtures significantly increased resilient modulus while it decreased fatigue life. Gap graded mixtures

Table 2.32 Design Characteristics of Rubber-Asphalt Mixtures

(Takallou 1980)

Aggregate Gradation	Rubber Content (%)	Rubber Gradation (% coarse/ % fine)	Design Asphalt Content (%)	Marshall Stability (lb)	Flow (0.01 in.)
Gap graded	2	0/000	7.0	920	15
		60/40	7.2	690	21
		80/20	8.0	665	23
	3	0/100	7.5	600	19
		60/40	7.5	650	22
		80/20	9.3	[illegible]	33
Dense graded	0	No rubber	5.5	1,500	8
	3	80/20	7.5	550	22

Table 2.33 Resilient Modulus and Fatigue Life of Rubber-Asphalt Mixtures

(Takallou 1980)

Mix	No. of Samples Used in Calculations	Average Value of Air Voids		Average Value of MR		N_f	
		Percentage	SD	ksi	SD	Average Value	SD
A	3	2.17	0.06	1,872	27	29,237	3,629
B	3	2.19	0.12	2,044	128	29,736	2,991
C	3	2.18	0.08	2,084	83	25,070	7,600
D	3	2.14	0.08	2,165	18	22,515	1,504
E	3	2.09	0.03	2,149	52	24,174	1,996
F	4	2.13	0.12	2,047	58	20,768	3,887
G	3	4.08	0.27	1,713	194	46,751	20,326
H	3	2.05	0.08	2,356	175	47,990	256
I	4	2.24	0.09	2,149	74	41,194	5,471
J	3	4.02	0.17	1,787	113	43,271	4,617
K	3	2.12	0.07	2,351	50	89,062	7,012
L	3	2.22	0.05	2,488	127	75,325	4,920
M	2	2.33	0.16	2,588	34	41,788	2,075
N	3	2.22	0.19	2,414	212	118,186	15,670
O	3	2.15	0.24	2,592	161	97,032	18,825
P	3	2.21	0.09	2,225	100	84,153	5,007
Q	3	2.12	0.05	2,116	94	93,651	4,198
R	3	2.02	0.11	1,939	133	81,141	8,354
S	3	4.50	0.23	1,443	177	127,682	24,996
T	3	2.25	0.13	3,163	133	15,536	2,562

NOTE: SD = standard deviation. Specimen U was not tested for MR and N_f. Test temperature was −6°C; strain level was 100 microstrain.

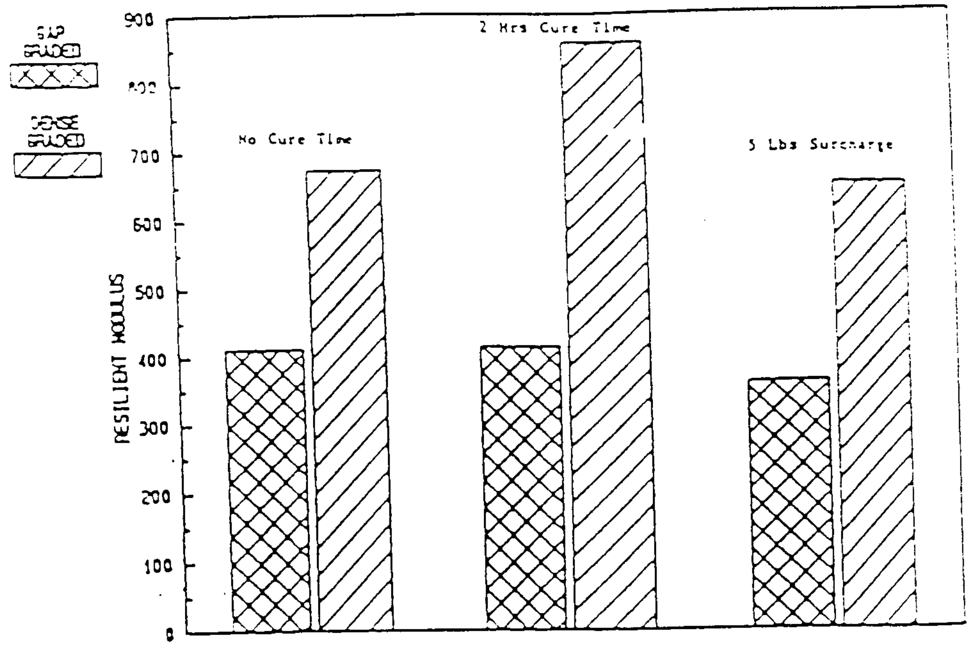


Figure 2.22 Effect of Curing Time and Surcharge on Resilient Modulus of Rubber Asphalt Mixtures, at 10°C, 3% Rubber and 80/20 Blend (Takallou 1980)

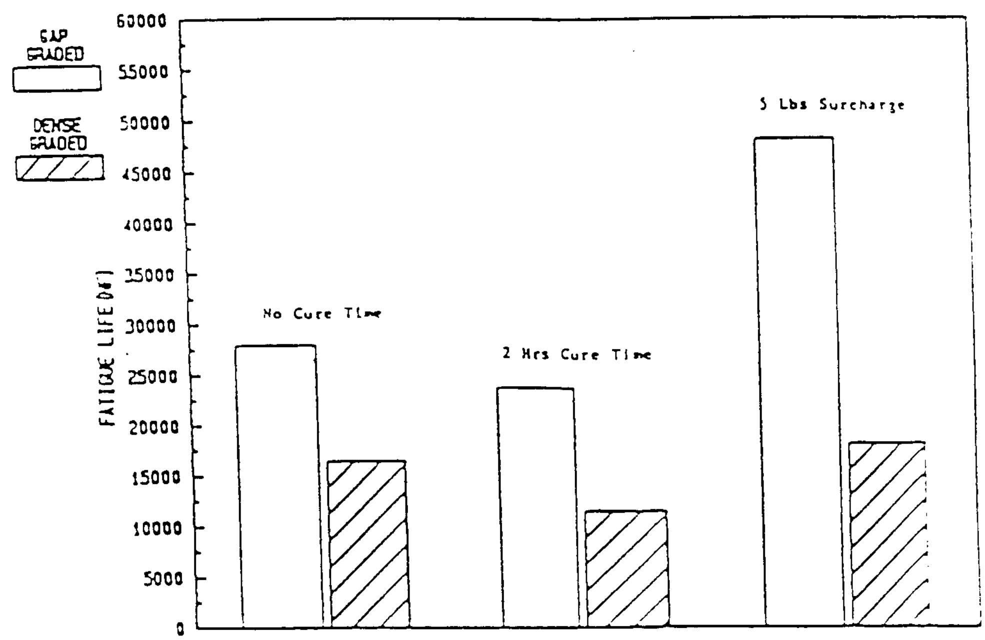


Figure 2.23 Effect of Curing Time and Surcharge on Fatigue Life of Rubber-Asphalt Mixtures, at 10°C, 3% Rubber and 80/20 Blend (Takallou 1980)

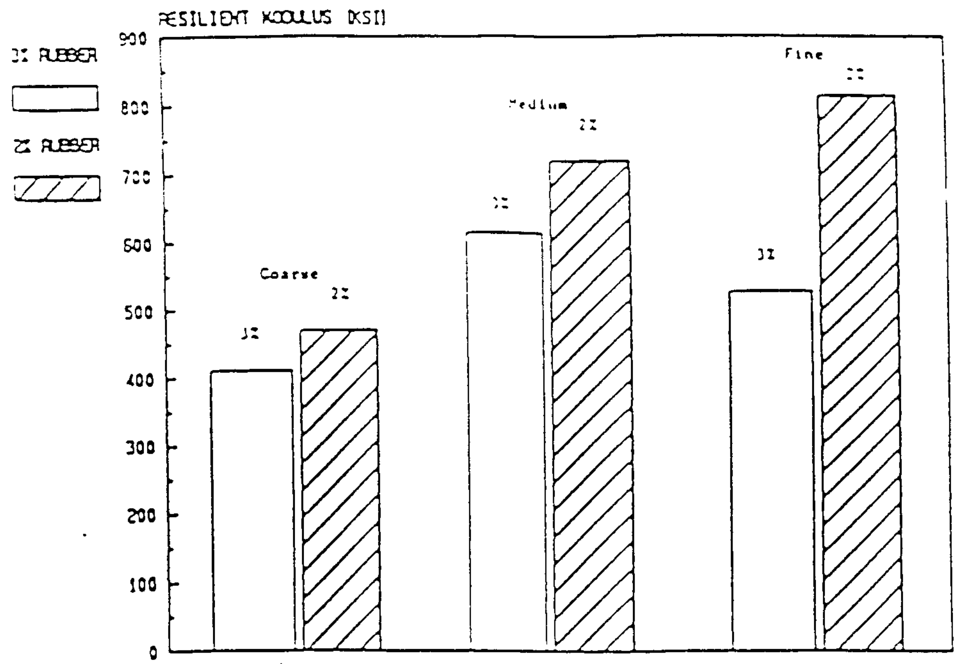


Figure 2.24 Effect of Rubber Content and Gradation on Resilient Modulus of Gap Graded Mixtures at 10°C (Takallou 1980)

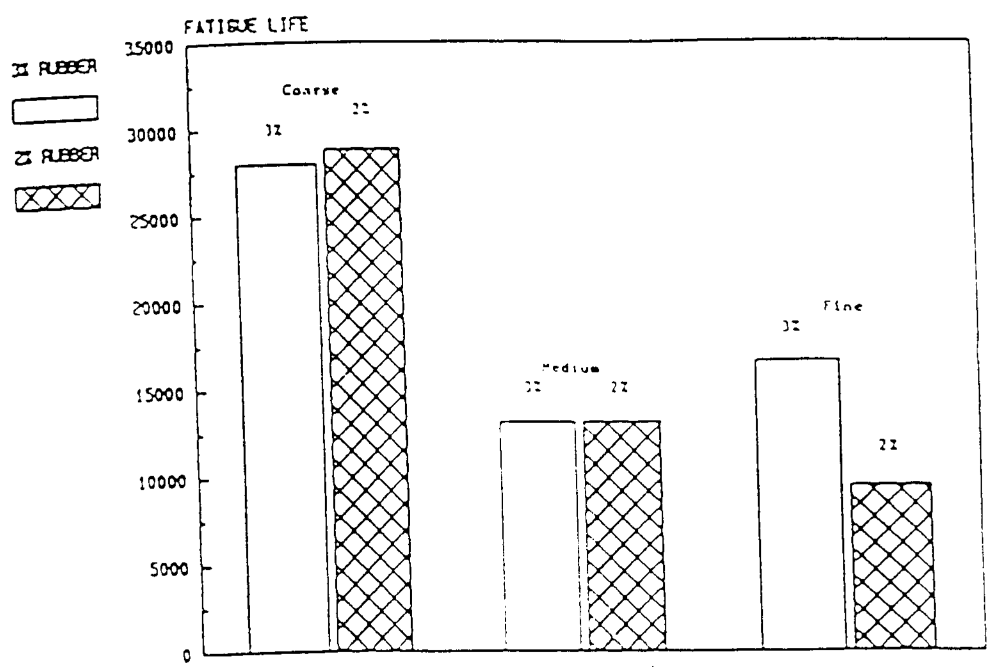


Figure 2.25 Effect of Rubber Content and Gradation on Fatigue Life Gap Graded Mixtures at 10°C (Takallou 1980)

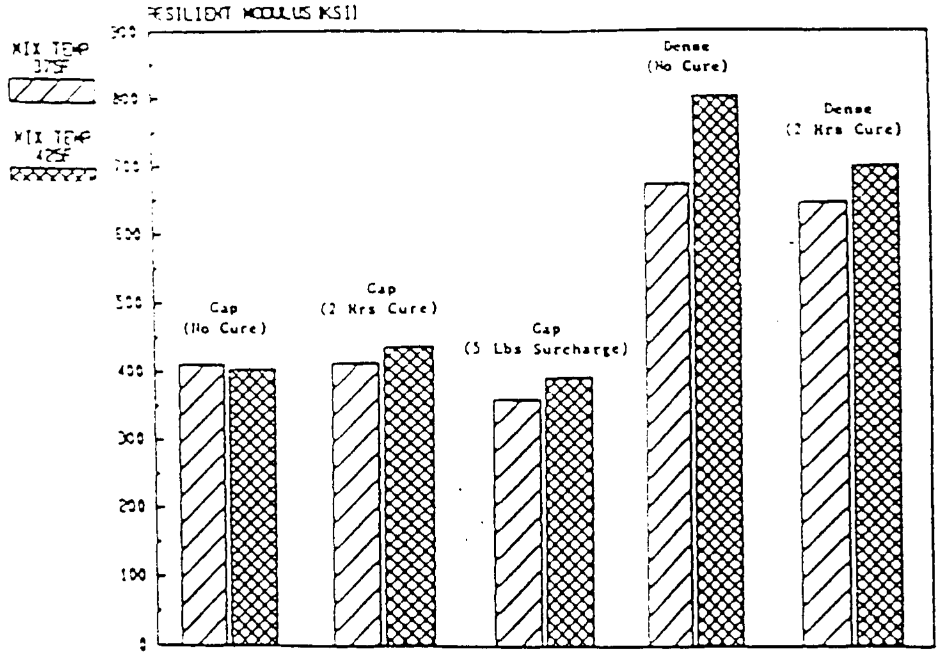


Figure 2.26 Effect of Mixing Temperature on Resilient Modulus, at 10°C 3% Rubber and 80/20 Blend (Takallou 1980)

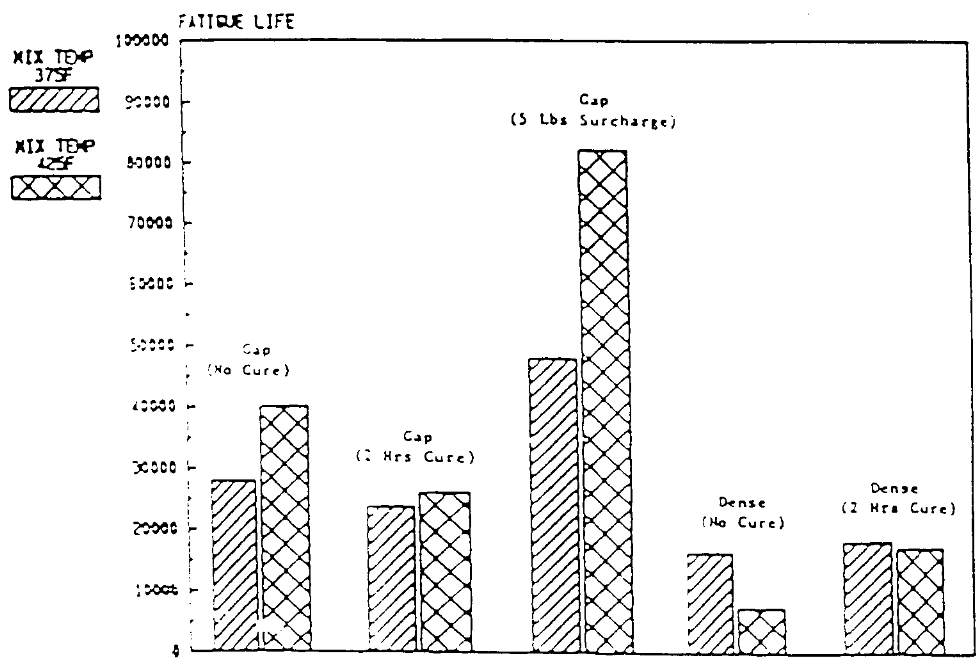


Figure 2.27 Effect of Mixing Temperature on Fatigue Life, at 10°C 3% Rubber and 80/20 Blend (Takallou 1980)

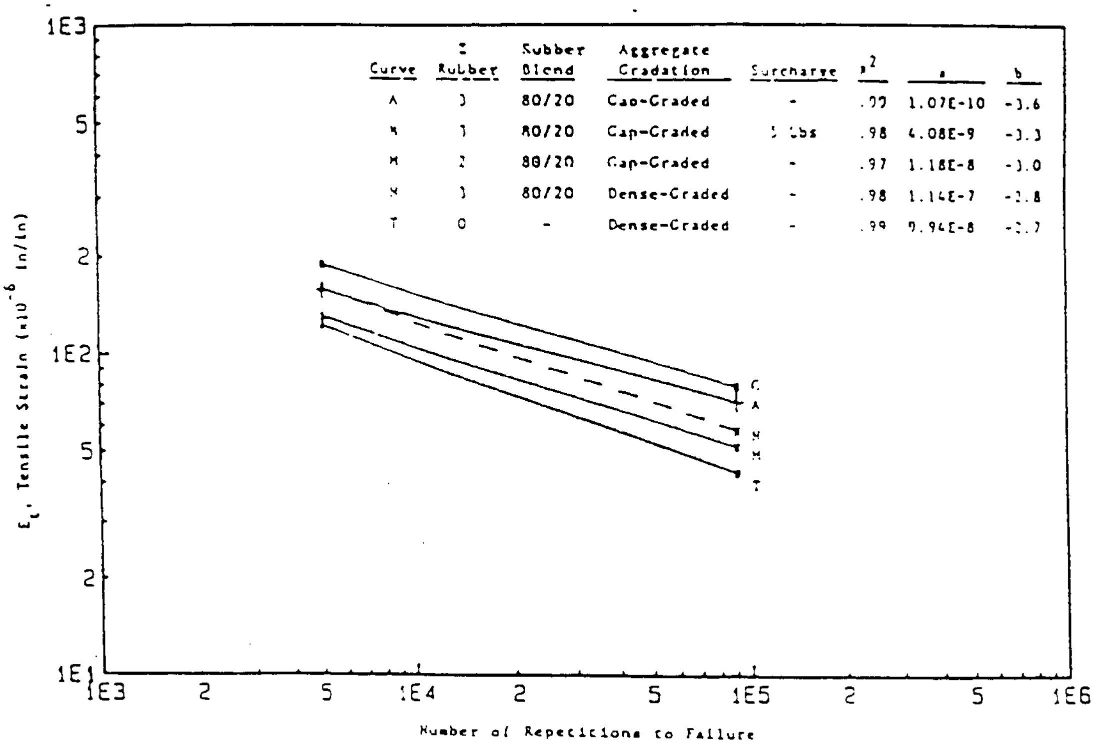


Figure 2.28 Fatigue Results at 10°C (Takallou 1980)

with higher rubber content had a lower modulus, while mixing temperature had some effect on the modulus of the dense graded mixtures, and the fatigue of gap graded mixtures.

Wet Process

The permanent deformation characteristics of asphalt-rubber mixtures were investigated by Krutz (1992). For such evaluation the static and repeated-load creep test were used with mixtures prepared with six different binders. The aggregate used, a crushed granite, had the gradation shown in Table 2.34. Three asphalt binders were included in the study, AC-5, AC-20, and AC-40, with the first two modified with crumb rubber with the characteristics shown in Table 2.35. The AC-5 was modified with the use of an extender oil, and the optimum binder contents were 8.5, 8.3 and 7.9% of the total mix for the mixtures with AC-5, AC-5 and extender oil and AC-20 respectively. The factorial experiment of the study is shown in Table 2.36. The 100 mm (4") by 200 mm (8") samples were prepared allowed to cool overnight at room temperature before evaluating their bulk specific gravity and height according to ASTM D2726 and D3515 respectively. The samples were cured for 24 to 36 hours in a temperature control chamber at 25°C (77°F) and 40°C (104°F) before evaluating permanent deformation using the static and repeated-load creep test. Figures 2.29 and 2.30 illustrate the static test results at 25 °C and 40 °C respectively. Also, Figures 2.31 and 2.32 show the repeated load creep test results at 25 °C and 40 °C respectively. The relative ranking of strain is changed for both testing conditions when 25 °C test results were compared to 40 °C test results. The 25 °C test results were useful to assess the loss in stiffness when compared to testing at 40 °C; however, because of low testing temperature, they did not appear to be appropriate for characterization of permanent deformations. The static test results at 40 °C were not able to discriminate well between mixtures. The repeated load testing at 40 °C indicated, in a concrete manner, the difference that exist between the different binders. Thus the following conclusions were drawn: permanent deformation testing should be carried out at elevated temperature. Since rutting occur primarily at high temperatures and the modified mixtures react differently at lower temperatures; and the repeated-loading test should be used for permanent deformation evaluation since the static test is not able to discriminate well between mixtures.

Table 2.34 Aggregate Gradation and Specifications (Krutz 1992)

Sieve Size	Laboratory Gradation	ASTM D3515 3/4" Dense	Nevada Type II	California 3/4" Medium
	Cumulative Percent Passing			
3/4"	100	100	90-100	100
1/2"	98	90-100	—	89-100
3/8"	85	—	63-85	75-100
#4	58	44-74	45-63	51-74
#8	40	28-58	—	35-57
#16	28	—	—	—
#30	20	—	—	14-35
#50	14	5-21	—	—
#100	9	—	—	—
#200	5	2-10	3-9	0-11

Table 2.35 Gradation of Ground Tire Rubber (Krutz 1992)

Sieve Size	Baker IGR-24	Manufacturer Recommendations
	Cumulative Percent Passing	
#10	100	100
#16	100	100
#30	78	70-100
#40	49	—
#50	27	—
#80	9	0-20
#100	7	—
#200	0.2	0-5

Table 2.36 Factorial for Permanent Deformation Evaluation

(Krutz 1992)

	Binder	AC5 Ext. Oil		AC5		AC20		AC40	
	Modifier	Orig.	Rubber Added	Orig.	Rubber Added	Orig.	Rubber Added	Orig.	Rubber Added
Static Load	77°F		X	X	X	X	X	X	
	104°F		X	X	X	X	X	X	
Repeat. Load	77°F		X	X	X	X	X	X	
	104°F		X	X	X	X	X	X	

TOTAL OF 72 SAMPLES (each X denotes 3 samples)

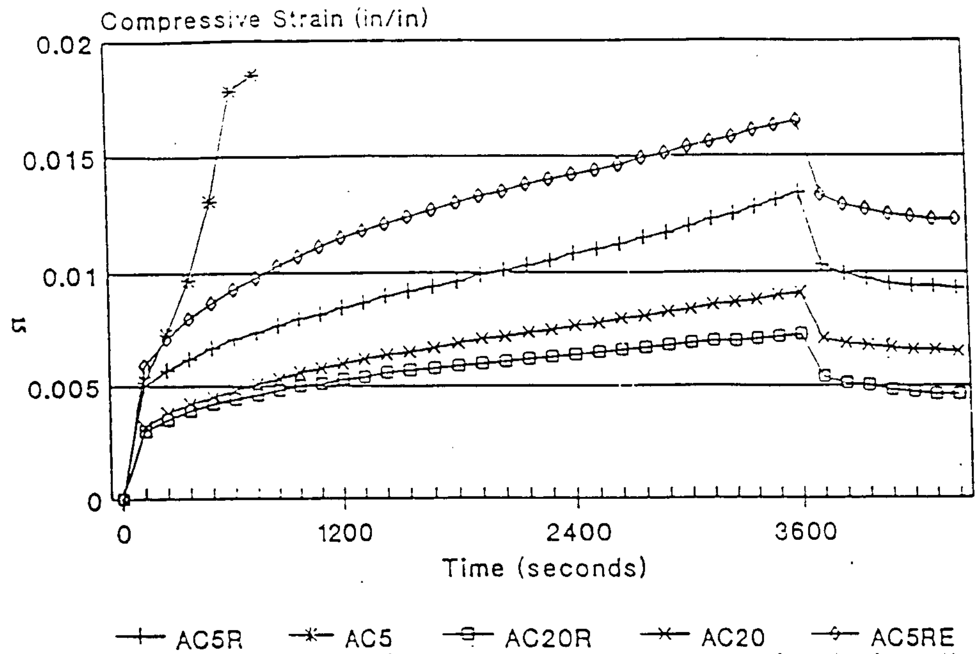


Figure 2.29 Compressive Strain Versus Time for Static Loading Conducted at 25°C (Krutz 1992)

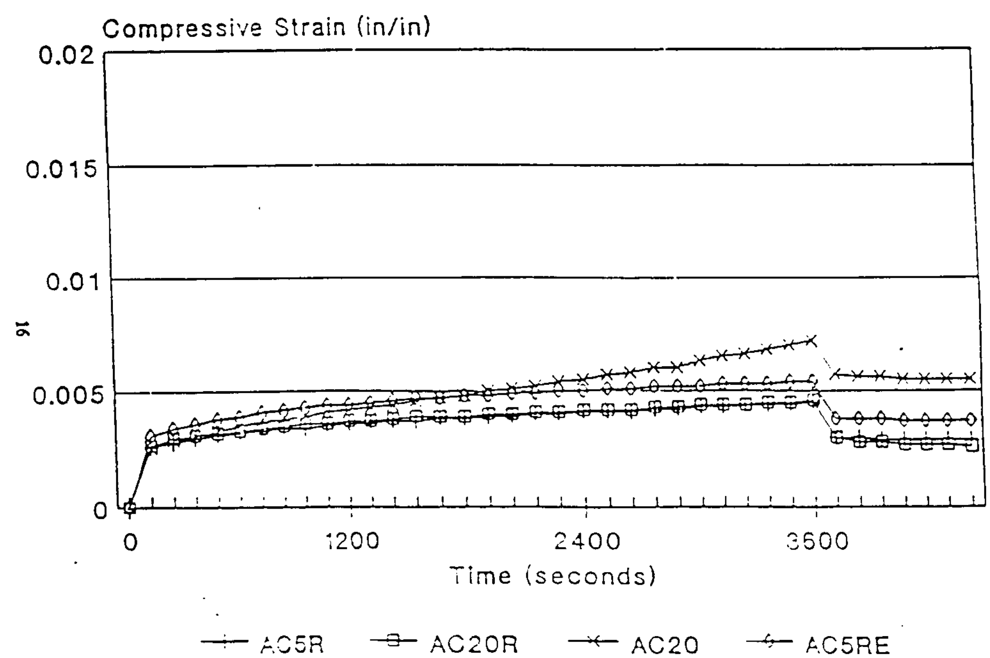


Figure 2.30 Compressive Strain Versus Time for Static Loading Conducted at40°C (Krutz 1992)

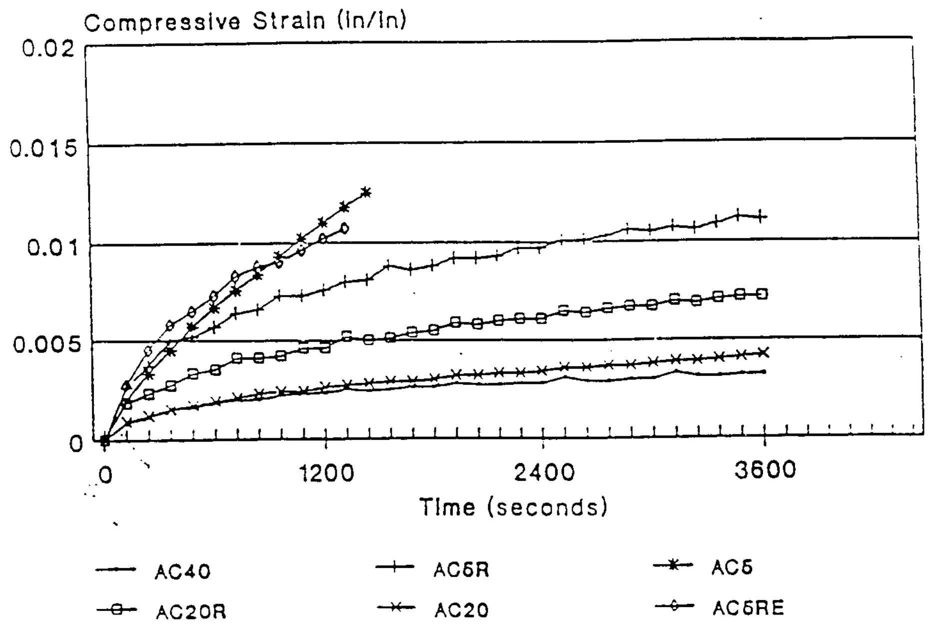


Figure 2.31 Compressive Strain Versus Time for Repeated Loading Conducted at 25°C (Krutz 1992)

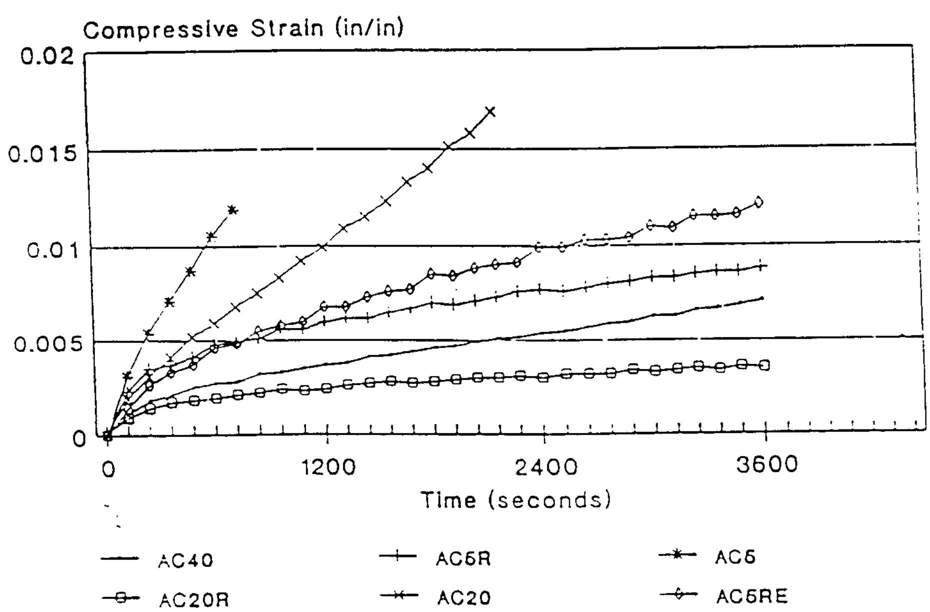


Figure 2.32 Compressive Strain Versus Time for Repeated Loading Conducted at40°C (Krutz 1992)

IMPROVED DESIGN PROCEDURES FOR ASPHALT-RUBBER MIXTURE

Mix design procedures used today, such as the Marshall and Hveem, are based on empirically based procedures for selecting the binder content for specific aggregate characteristics meeting criteria selected on the basis of experience. Investigations were performed over the years for modifying these empirical based methods so as to improve mixture selection for specific traffic and environmental conditions. The search for improving mixture designs was oriented either to modify the existing procedures or define new mixture design methodologies.

Significant progress is being achieved over the years in measuring fundamental mixture properties, (able to describe mixture behavior and relate to mixture distress failures), and correlating them to mixture and material parameters. Monismith (1985), among other investigators, considered the design of mixtures for specific traffic and environmental conditions which will mitigate the modes of distress most commonly associated with asphalt pavements. Some of the parameters considered in this effort were: mixture stiffness; resistance to permanent deformation; durability; fatigue resistance; low temperature response (including stiffness at long loading times and fracture characteristics; permeability. On the other hand, efforts investigating the definition of new mixture design procedures include requirements on: fatigue cracking, permanent deformation; thermal cracking; and consideration on limiting subgrade excessive deformation.

For the design of dense graded asphalt rubber mixtures, prepared according to the wet process, the Marshall and Hveem methodologies have been typically used. The open graded mixtures prepared with asphalt rubber binders involves usually the determination of asphalt content, void capacity, fine aggregate content, mixing temperature, and retained strength. Generally, the amount of crumb rubber in asphalt rubber mixtures ranges from 15-25 percent by weight of the asphalt cement.

The rubber modified mixtures prepared with the dry process are gap graded mixtures. The use of Marshall or Hveem methods for these mixtures often did not provide satisfactory results.

Traditional specimen preparation methods are used for these mixtures and the design is often governed by the air void content, with values between 2 to 4 percent. The asphalt binder content for these mixtures range from 7.5 to 9 percent, by weight of the mixture.

CHAPTER 3. FACTORIAL EXPERIMENTS

INTRODUCTION

In order to achieve the objectives of this study, the laboratory investigation included several experiments. These experiments were focused towards: evaluation of the conventional and rubber modified binder; design of the conventional and rubber modified mixtures with the standard NJDOT mix design method, these data were useful in coupling this method with material behavior parameters so as to enhance the current methodology; evaluation of the modified mixtures static indirect tensile characteristics; resilient modulus evaluation; creep behavior at different temperatures and stresses; and determination of the indirect tensile fatigue life at different stress and loading conditions. The binder, rubber aggregate and mixture characteristics considered in this study were selected from NJDOT experimental projects with rubber modified mixtures.

NEW JERSEY DOT ASPHALT RUBBER PROJECTS

Several asphalt rubber projects were built during the recent years by New Jersey Department of Transportation. The NJDOT experimental projects included mixtures prepared with both the wet and dry processes. Specifically the following asphalt-rubber mixtures, prepared with the wet process, were used by NJDOT: dense graded rubberized bituminous mixture for surface and base course; open graded friction course rubberized bituminous mixture; rubberized mix with glass addition for surface and base courses; rubberized mix with Reclaimed Asphalt Pavement (RAP) addition, for surface and base courses. The dry process mixtures included: gap graded rubberized bituminous mix, Plus Ride II, for surface and base course; rubberized mix with glass addition, for surface and base courses; and rubberized mix with RAP addition, for surface and base courses. Available project information with material and mixture characteristics are provided in Tables 3.1 and 3.2.

Table 3.1. NJDOT Paving Projects with Asphalt-Rubber Mixtures - Wet Process

Project Name	Route I 95	Asphalt Mixture Gradation	Dense Graded Rubberized Bituminous Mixture
Construction Date	April 1994	Asphalt Mixture Type :	Surface Course
Section Name :	21F3K	Asphalt Mixture Code Number	I-4

Wet Process

Variables Of	Bin #	Used Materials	Type	Supplier	Sieve Size	2"	3/2"	1"	3/4"	1/2"	3/8"	# 4	# 8	# 16	# 30	# 50	# 200	Percentages
Aggregate	1	Washed Sand		Clayton Sand	% Passing							100	94.2	78.3	59.1	36	12.2	34
	2	#10 Sand		Trap Rock Industries							100	60	8	1.6	1.2	1	0.4	25
	3	#8 Agg.		Better Materials					100	99.6	73	4.3	1.2	0.8	0.4	0.3	0.2	28
	4	1" Agg.		Trap Rock Industries				100	90	28	2.6	1.9	1	0.8	0.6	0.5	0.4	13
	5																	
	Filler																	
	Composite Gradation							100	99	90	80	51	34.5	27	21	13	4.4	

Variables Of			
Asphalt Cement	1. Supplier		Koch Materials Co.
	2. Grade		AC-10
	3. Others		
Crumb Rubber	1. Supplier		Asphalt Rubber System
	2. Type	A	Baker TR-24
		B	Baker TBS-20

Crumb Rubber 3. Gradation	1/4"	# 4	# 8	# 10	# 16	# 30	# 40	# 50	# 60	# 80	# 10	# 200	Percentages
A				100	100	73				14		3.3	89
B				100	100	56				7		0.9	11
Composite				100	100	71				13		3	

Variables Of			
Crumb Rubber	4. Extender Oil Supplier		Asphalt Rubber System
	5. Extender Oil Type		Sundex 790
Asphalt-Rubber Binder	1. Asphalt Cement Percent		75 (%)
	2. Rubber Percent	A	16 (%)
		B	2 (%)
	3. Extender Oil Percent		7 (%)
	4. Mixing Temperature		350 F
	5. Mixing Time		
	6. Curing Temperature		
	7. Curing Time		
Modified Mixture	1. Binder Percent		5.9 (%) by total weight of mix
	2. Mixing Temperature		280 F (laydown)
	3. Compaction Level (Air Void %)		

Table 3.1. NJDOT Paving Projects with Asphalt-Rubber Mixtures - Wet Process (Continue)

Project Name : Route I-95
Construction Date : April 1994
Section Name : 2H/3K

Asphalt Mixture Gradation : Dense Graded Rubberized Bituminous Mixture
Asphalt Mixture Type : Base Course
Asphalt Mixture Code Number : I-2

Wet Process

Variables Of	Bin #	Used Materials	Type	Supplier	Sieve Size	2"	3/2"	1"	3/4"	1/2"	3/8"	# 4	# 8	# 16	# 30	# 50	# 200	Percentages
Aggregate	1				% Passing													34.5
	2																	18
	3																	13.5
	4																	34
	5																	
	Filler					100	100	100		73		47	35			14	5.4	
	Composite Gradation																	

Variables Of			
Asphalt Cement	1. Supplier		Koch Materials
	2. Grade		AC-10
	3. Others		Asphalt Rubber System
Crumb Rubber	1. Supplier	A	Baker TR-24
	2. Type	B	Baker TBS-20
	3. Gradation		(see below)
	4. Extender Oil Supplier		Asphalt Rubber System
	5. Extender Oil Type		Sundex 790
Asphalt-Rubber Binder	1. Asphalt Cement Percent		75%
	2. Rubber Percent	A	16%
		B	2%
	3. Extender Oil Percent		7%
	4. Mixing Temperature		350 F
	5. Mixing Time		
	6. Curing Temperature		
	7. Curing Time		
Modified Mixture	1. Binder Percent		5.2 (%) by total weight of mix
	2. Mixing Temperature		280 F (laydown)
	3. Compaction Level (Air Void %)		

Crumb Rubber – 3. Gradation

Sieve Size	1/4"	# 4	# 8	# 10	# 16	# 30	# 40	# 50	# 60	# 80	# 10	# 200	Percentages
A				100	100	73				14		3.3	89
B				100	100	56				7		0.9	11
Composite				100	100	71				13		3	

Table 3.1. NJDOT Paving Projects with Asphalt-Rubber Mixtures - Wet Process (Continue)

Project Name : Route I-195 (in Allentown)
Construction Date : August 1992
Section Name : Westbound

Asphalt Mixture Gradation : Open Graded Rubberized Bituminous Mixture
Asphalt Mixture Type : Friction Course
Asphalt Mixture Code Number :

Variables Of	Wet Process																	
Aggregate	Bin #	Used Materials	Type	Supplier	Sieve Size	2"	3/2"	1"	3/4"	1/2"	3/8"	# 4	# 8	# 16	# 30	# 50	# 200	Percentages
	1				% Passing													
	2																	
	3																	
	4																	
	5																	
	Filler																	
	Composite Gradation																	
Asphalt Cement	1. Supplier																	
	2. Grade																	
	3. Others																	
Crumb Rubber	1. Supplier																	
	2. Type			A		80 mesh size												
				B														
	3. Gradation			Sieve Size		1/4"	# 4	# 8	# 10	# 16	# 30	# 40	# 50	# 60	# 80	# 10	# 200	Percentages
				A														
				B														
				Composite														
	4. Extender Oil Supplier																	
	5. Extender Oil Type																	
Asphalt-Rubber Binder	1. Asphalt Cement Percent																	
	2. Rubber Percent			A		15 % by weight of asphalt cement												
				B														
	3. Extender Oil Percent																	
	4. Mixing Temperature																	
	5. Mixing Time																	
	6. Curing Temperature																	
	7. Curing Time																	
Modified Mixture	1. Binder Percent																	
	2. Mixing Temperature																	
	3. Compaction Level (Air Void %)			Air Void is 15 % and the laydown temperature is 280 F														

Table 3.1. NJDOT Paving Projects with Asphalt-Rubber Mixtures - Wet Process (Continue)

Project Name : Route 9 (in Little Egg Harbor Township)
Construction Date : October 1993
Section Name :

Asphalt Mixture Gradation : Open Graded Rubberized Bituminous Mixture
Asphalt Mixture Type : Friction Course
Asphalt Mixture Code Number :

Variables Of	Wet Process																
Aggregate	Bin #	Used Materials	Type	Supplier	Sieve Size	2"	3/2"	1"	3/4"	1/2"	3/8"	# 4	# 8	# 16	# 30	# 50	# 200	Percentages
	1				% Passing													
	2																	
	3																	
	4																	
	5																	
	Filler																	
	Composite Gradation																	
Asphalt Cement	1. Supplier																	
	2. Grade																	
	3. Others																	
Crumb Rubber	1. Supplier																	
	2. Type			A	40 mesh size													
				B														
	3. Gradation			Sieve Size	1/4"	# 4	# 8	# 10	# 16	# 30	# 40	# 50	# 60	# 80	# 10	# 200	Percentages	
				A														
				B														
				Composite														
	4. Extender Oil Supplier																	
	5. Extender Oil Type																	
Asphalt-Rubber Binder	1. Asphalt Cement Percent																	
	2. Rubber Percent			A	15 (%) by weight of asphalt cement													
				B														
	3. Extender Oil Percent																	
	4. Mixing Temperature																	
	5. Mixing Time																	
	6. Curing Temperature																	
	7. Curing Time																	
Modified Mixture	1. Binder Percent				290-320 F													
	2. Mixing Temperature				280 F (laydown)													
	3. Compaction Level (Air Void %)																	

Table 3.2. NJDOT Paving Projects with Rubber-Filled Mixtures - PlusRide II

Project Name :	Route 130 (in Logan Township)	Asphalt Mixture Gradation :	Gap Graded Rubberized Bituminous Mix, Plus Ride II
Construction Date :	September 1993	Asphalt Mixture Type :	Surface Course
Section Name :	Southbound	Asphalt Mixture Code Number :	Mix 12

Variables Of	Dry Process																	
					Sieve Size	2"	3/2"	1"	3/4"	1/2"	3/8"	# 4	# 8	# 16	# 30	# 50	# 200	Percentages
Aggregate	Bin #	Used Materials	Type	Supplier	% Passing													
	1																	
	2																	
	3																	
	4																	
	5																	
	Filler									100	60-80	27-41	21-33		13-25		8-12	
	Composite Gradation																	
Asphalt Cement	1. Supplier																	
	2. Grade																	
	3. Others																	
Crumb Rubber	1. Supplier																	
	2. Type			A														
				B														
	3. Gradation			Sieve Size		1/4"	# 4	# 8	# 10	# 16	# 20	# 40	# 50	# 60	# 80	# 10	# 200	Percentages
				A														
				B														
				Composite		100	76-100		28-42		16-24							
Modified Mixture	1. Asphalt Cement Percent																	
	2. Crumb Rubber Percent			A	3 % by total weight of mix													
				B														
	3. Mixing Temperature																	
	4. Compaction Level (Air Void %)																	

Table 3.2. NJDOT Paving Projects with Rubber-Filled Mixtures - PlusRide II (Continue)

Project Name : Route 130 (in Logan Township)
Construction Date : September 1993
Section Name : Southbound

Asphalt Mixture Gradation : Gap Graded Rubberized Bituminous Mix, Plus Ride II
Asphalt Mixture Type : Base Course
Asphalt Mixture Code Number : Mix 16

Variables Of	Dry Process																	
Aggregate	Bin #	Used Materials	Type	Supplier	Sieve Size	2"	3/2"	1"	3/4"	1/2"	3/8"	# 4	# 8	# 16	# 30	# 50	# 200	Percentages
	1				% Passing													
	2																	
	3																	
	4																	
	5																	
	Filler																	
	Composite Gradation								100		50-62	27-41	21-33		12-23		7-11	
Asphalt Cement	1. Supplier																	
	2. Grade																	
	3. Others																	
Crumb Rubber	1. Supplier																	
	2. Type			A														
				B														
	3. Gradation			Sieve Size	1/4"	# 4	# 8	# 10	# 16	# 20	# 40	# 50	# 60	# 80	# 10	# 200	Percentages	
				A														
				B														
				Composite	100	76-100		28-42		16-24								
Modified Mixture	1. Asphalt Cement Percent				3 % by total weight of mix													
	2. Crumb Rubber Percent			A														
				B														
	3. Mixing Temperature																	
	4. Compaction Level (Air Void %)																	

Table 3.2. NJDOT Paving Projects with Rubber-Filled Mixtures - PlusRide II (Continue)

Project Name : Route 41 (in Cherry Hill)
Construction Date :
Section Name :

Asphalt Mixture Gradation : Gap Graded Rubberized Bituminous Mix. Plus Ride II
Asphalt Mixture Type : Surface Course
Asphalt Mixture Code Number :

Variables Of	Dry Process																	
					Sieve Size	2"	3/2"	1"	3/4"	1/2"	3/8"	# 4	# 8	# 16	# 30	# 50	# 200	Percentages
Aggregate	Bin #	Used Materials	Type	Supplier	% Passing													
	1																	
	2																	
	3																	
	4																	
	5																	
	Filler									100	60-80	27-41	21-33		13-25		8-12	
	Composite Gradation																	
Asphalt Cement	1. Supplier																	
	2. Grade																	
	3. Others																	
Crumb Rubber	1. Supplier			A														
	2. Type			B														
	3. Gradation			Sieve Size		1/4"	# 4	# 8	# 10	# 16	# 20	# 40	# 50	# 60	# 80	# 10	# 200	Percentages
				A														
				B														
				Composite		100	76-100		28-42		16-24							
Modified Mixture	1. Asphalt Cement Percent				3 % by total weight of mix													
	2. Crumb Rubber Percent			A														
				B														
	3. Mixing Temperature																	
	4. Compaction Level (Air Void %)																	

In the wet process mixtures extender oil was used with the following characteristics: 80 poises minimum viscosity at 99 °C (210 °F) with ASTM D216; 199 °C (390 °F) minimum flash point with AASHTO T48-84; 10 percent maximum asphaltene weight; and 55 percent minimum aromatic oil by weight of asphalt. The recommendations for asphalt rubber binder preparation included: heating asphalt cement, before mixing, at 177 °C (350 °F) to 218 °C (425 °F); evaluate reaction time at 163 °C (325 °F) to 205 °C (400 °F); minimum mixing time 45 minutes; 1500-4000 cp viscosity at 177 °C (350 °F) with ASTM D2669; 50-150 tenths of millimeter cone penetration at 25 °C (77 °F) with ASTM D3407; 43 °C (110°F) minimum softening point with ASTM D36; and 10 percent minimum resilience, 25 °C (77 °F) with ASTM D3407.

AGGREGATE, RUBBER AND ASPHALT CHARACTERISTICS

The trap rock aggregate was obtained from the Trap Rock Industries, in New Jersey, a NJDOT supplier. Four different sizes of raw aggregates were obtained for building the desired gradation: a 18.7mm (3/4") and a 9.37mm (3/8") maximum nominal size aggregates; screening material; and sand. The sieve analysis on these materials was conducted according to ASTM C-136. The gradations of these aggregates are shown in Table 3.3, and depicted in Figure 3.1. These aggregates were then separated into ten fractions so as to prepare the recommended NJDOT gradation for wet and dry processes. The sieves used for splitting the aggregates included: 25.4 mm (1") , 19.0 mm (3/4"), 12.5 mm (1/2"), 9.5 mm (3/8"), 4.76 mm (# 4) , 2.38 mm (# 8), 1.19 mm (# 16), 0.6 mm (# 30), 0.3 mm (# 50), and 0.075 mm (# 200). The wet process is used with dense graded mixture for wearing course as the one used for Route I-95 project. The dense graded mixture gradation with NJDOT specification limits is shown in Table 3.4 and Figure 3.2 . The dry process (PlusRide II) is used with gap graded mixture for surface course. The selected gap graded mixture gradation and NJDOT specification limits (as used in Route 130, Logan Township, project) are illustrated in Table 3.5 and Figure 3.3.

Table 3.3 Trap Rock Aggregate Characteristics

Sieve Size inch (mm.)	% Passing			
	Coarse Agg. 19.0 mm (3/4")	Fine Agg. 12.5 mm (3/8")	Sand	Screening
1" (25.4)	100	100	100	100
3/4" (19.0)	65.3	100	100	100
1/2" (12.5)	13.6	100	100	100
3/8" (9.5)	2.4	71.7	100	100
# 4 (4.76)	-	56.1	98.4	100
# 8 (2.38)	-	47.5	67.6	99.4
# 16 (1.19)	-	22.6	50.4	96.6
# 30 (0.6)	-	13.9	39.8	83
# 50 (0.3)	-	6.9	30.4	22.3
# 200 (0.075)	-	3.6	9.8	1.3

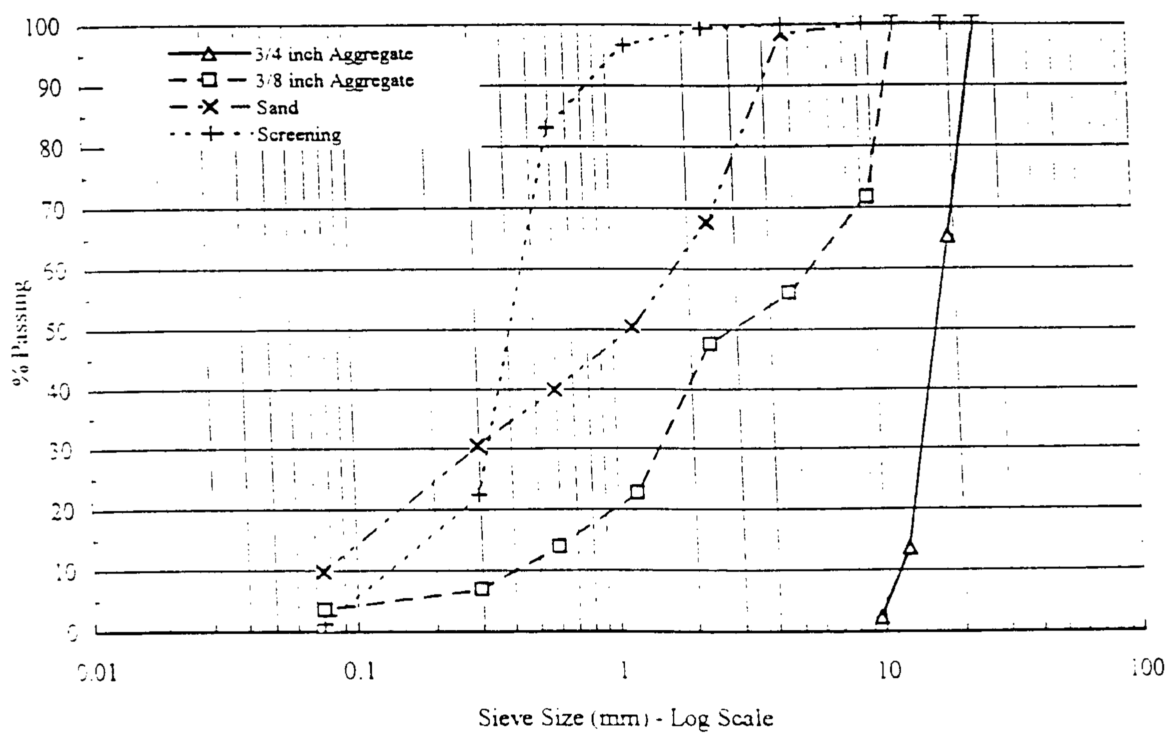


Figure 3.1 Trap rock Aggregate Gradations

Table 3.4 Aggregate Gradation and NJDOT Specifications for Wet Process

sieve size inch (mm.)	% Passing	
	Mixture Gradation	Spec. Limits
1" (25.4)	100	100
3/4" (19.0)	99	98/100
1/2" (12.5)	90	88/98
3/8" (9.5)	80	65/88
# 4 (4.76)	51	35/65
# 8 (2.38)	34.5	25/50
# 16 (1.19)	27	18/40
# 30 (0.6)	21	12/30
# 50 (0.3)	13	10/25
# 200 (0.075)	4.4	3/10

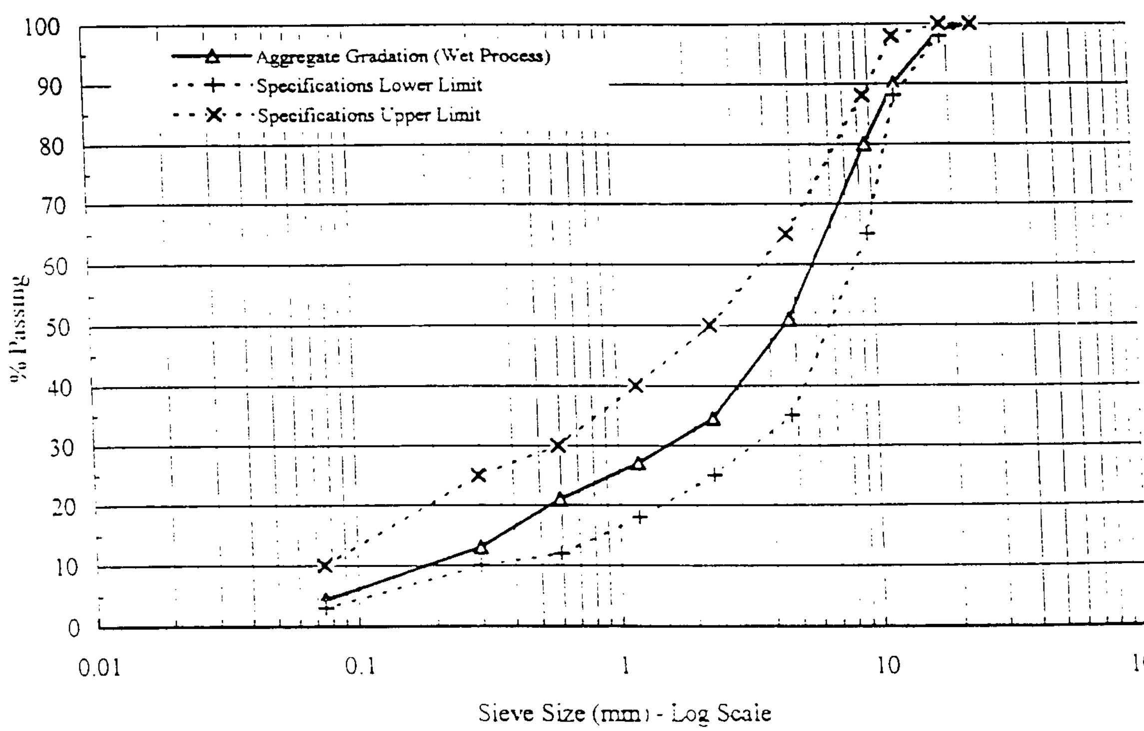


Figure 3.2 Aggregate Gradation and NJDOT Specifications for Wet Process

Table 3.5 Aggregate Gradation and NJDOT Specifications for PlusRide II

sieve size inch (mm.)	% Passing	
	Mixture Gradation	Spec. Limits
1/2" (12.5)	100	100
3/8" (9.5)	70	60/80
# 4 (4.76)	34	27/41
# 8 (2.38)	27	21/33
# 16 (1.19)	23	-
# 30 (0.6)	19	13/25
# 50 (0.3)	16	-
# 200 (0.075)	10	8/12

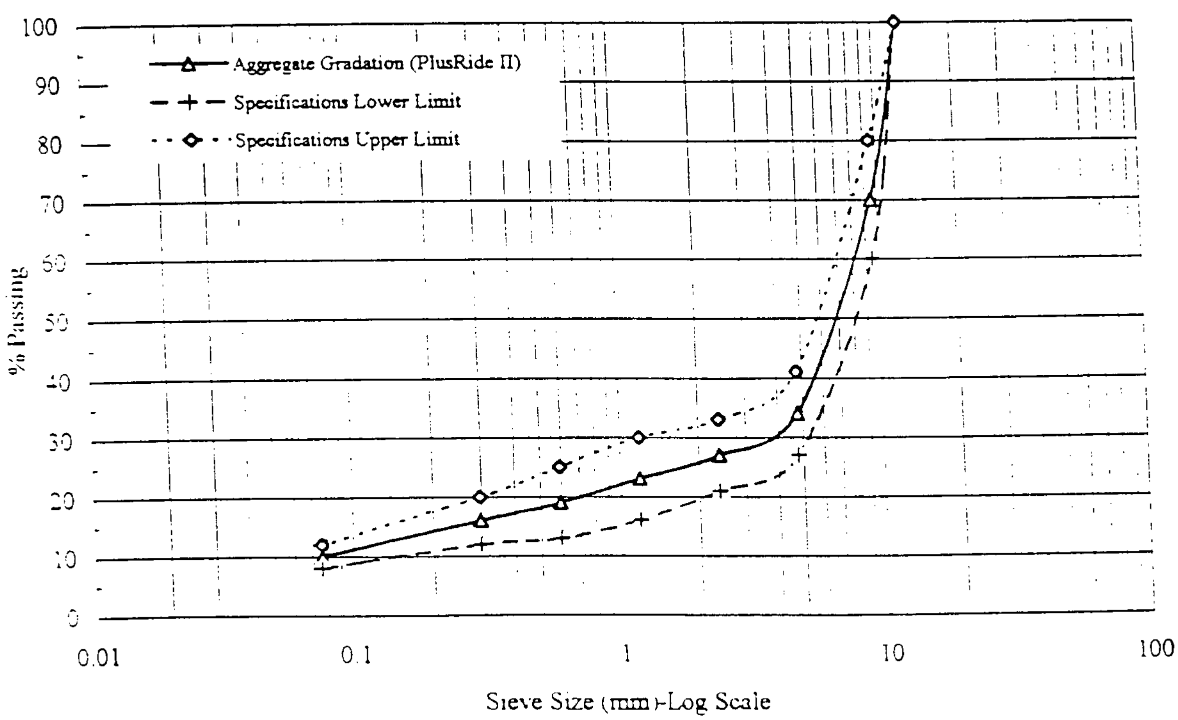


Figure 3.3 Aggregate Gradation and NJDOT Specifications For Plus Ride II

The bulk, saturated surface dry, and effective specific gravities, and water absorption of the coarse aggregate fractions were evaluated according to ASTM C-127. Similarly, the specific gravities of the fine aggregate fractions were evaluated according to ASTM C-128, see Table 3.6.

Table 3.6 Aggregate Specific Gravity

Aggregate Fraction		Specific Gravity		
Passing From	Retained On	Bulk	SSD*	Apparent
25.4 (1")	19.0 (3/4 ")	2.747	2.781	2.845
19.0 (3/4 ")	12.5 (1/2 ")	2.757	2.790	2.850
12.5 (1/2 ")	9.5 (3/8 ")	2.695	2.730	2.794
9.5 (3/8 ")	4.76 (# 4)	2.607	2.638	2.691
4.76 (# 4)	2.38 (# 8)	2.669	2.749	2.901
2.38 (# 8)	1.19 (# 16)	2.658	-	-
1.19 (# 16)	0.6 (# 30)	2.475	-	-
0.6 (# 30)	0.3 (# 50)	2.151	-	-
0.3 (# 50)	0.075 (# 200)	2.794	-	-
0.075 (# 200)	Pan	2.683	-	-

* SSD = Saturated Surface Dry

In the I-95 surface course project, two types of crumb rubber were used. As for the construction projects, these two crumb rubbers were obtained from Baker Rubber Inc. in Pennsylvania. The first one was a TR-24 while the second was a MAT-20 identical to the TBS-20 used in the I-95 project. The gradations of these crumb rubber types along with the NJDOT specifications are shown in Table 3.7 and Figure 3.4. The NJDOT specification requirements for the crumb rubber was met by using 89 percent of TR-24 and 11 percent Mat-20. An extender oil, Sundex 790, was also used, and obtained from Asphalt Rubber System Inc.

Table 3.7 Crumb Rubber Characteristics and NJDOT Specifications for Wet Process

Sieve size (mm)	% Passing		
	TR-24	MAT-20	Composite
# 10 (2.0)	100	100	100
# 16 (1.19)	100	100	100
# 30 (0.595)	73	56	71
# 80 (0.177)	14	7	13
# 200 (0.075)	3.3	0.9	3
Percentage	89	11	

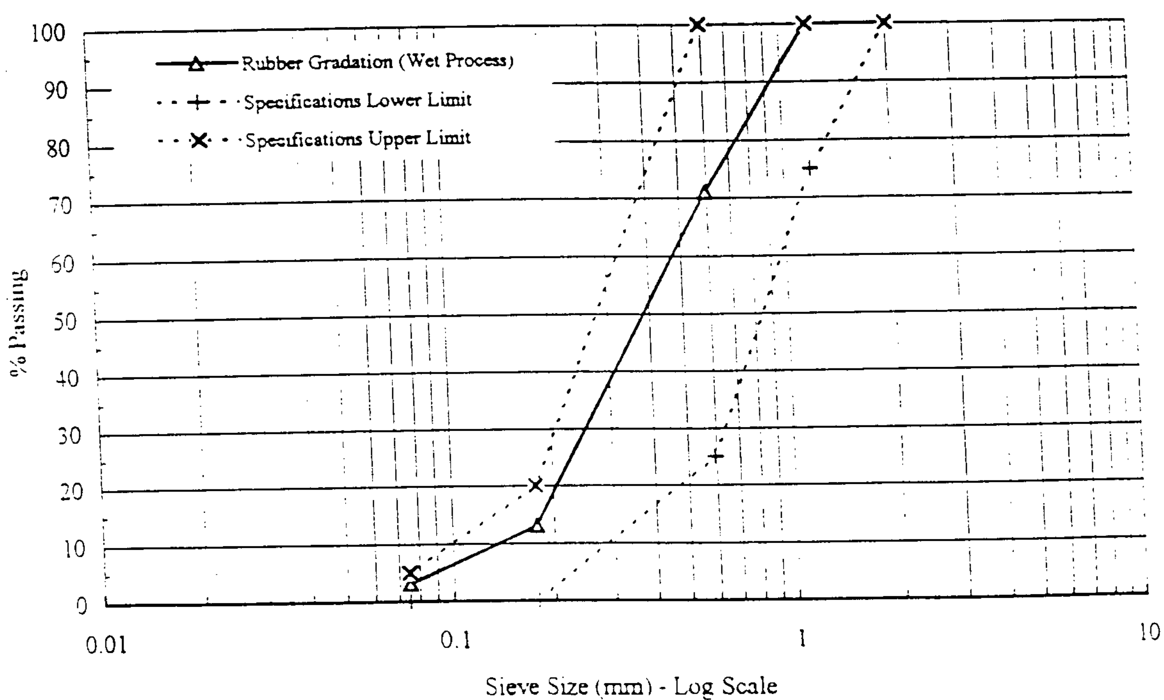


Figure 3.4 Rubber Gradation and NJDOT Specifications for Wet Process

PlusRide II crumb rubber was obtained from Tires Into Recycled & Supplies Inc. in Winston-Salem, North Carolina. Four different sizes of crumb rubber are provided for building the desired gradation: 6.25 mm (1/4"), and a 4.74 mm (3/16") maximum nominal size rubbers; 10-mesh rubber; and ABD-215 fine crumb rubber. These crumb rubber materials were then separated into four fractions so as to prepare the recommended NJDOT gradation for PlusRide II. The sieves used were 6.25 mm (1/4"), 4.76 mm (#4), 2.0 mm (#10), and 1.02 mm (#20). The crumb rubber gradation and the NJDOT specification limits are shown in Table 3.8 and Figure 3.5.

The asphalt cement used in this mixture was an AC-10 provided by Koch Materials Co., in New Jersey. The physical properties of this binder were evaluated and presented in the next chapter along with the modified binder evaluation.

ASPHALT RUBBER BINDER PREPARATION (WET PROCESS)

The steps undertaken for preparing the asphalt rubber binder in the laboratory, and suggested by Asphalt Rubber Systems Inc., included: i) the asphalt cement, AC-10, was heated with the extender oil, Sundex 790, to 205°C (400°F), and poured into the mixer pan; ii) the binder and extender oil were then mixed with a variable speed mixer till 500 rpm; at a temperature of 190°C (375°F) the crumb rubber (TR-24 and MAT-20) was added at once; the blend was mixed for 45 minutes, starting from the time the rubber was added, while the temperature was kept constant during mixing at 177°C (350°F); after the 45 minutes of blending time, the mixer was stopped and the temperature of the binder was kept constant until it was used for specimen preparation. The composition of the rubber modified binder was thus: 75 percent asphalt cement AC-10; 7 percent extender oil Sundex 790; 16 percent of Baker Rubber TR-20; and 2 percent of Baker Rubber MAT-20.

Table 3.8 Crumb Rubber Characteristics and NJDOT Specifications for PlusRide II

sieve size in (mm)	% Passing	
	Rubber Gradation	Spec. Limits
1/4" (6.25)	100	100
# 4 (4.76)	78	60/80
# 10 (2.00)	34	27/41
# 20 (1.02)	27	21/33

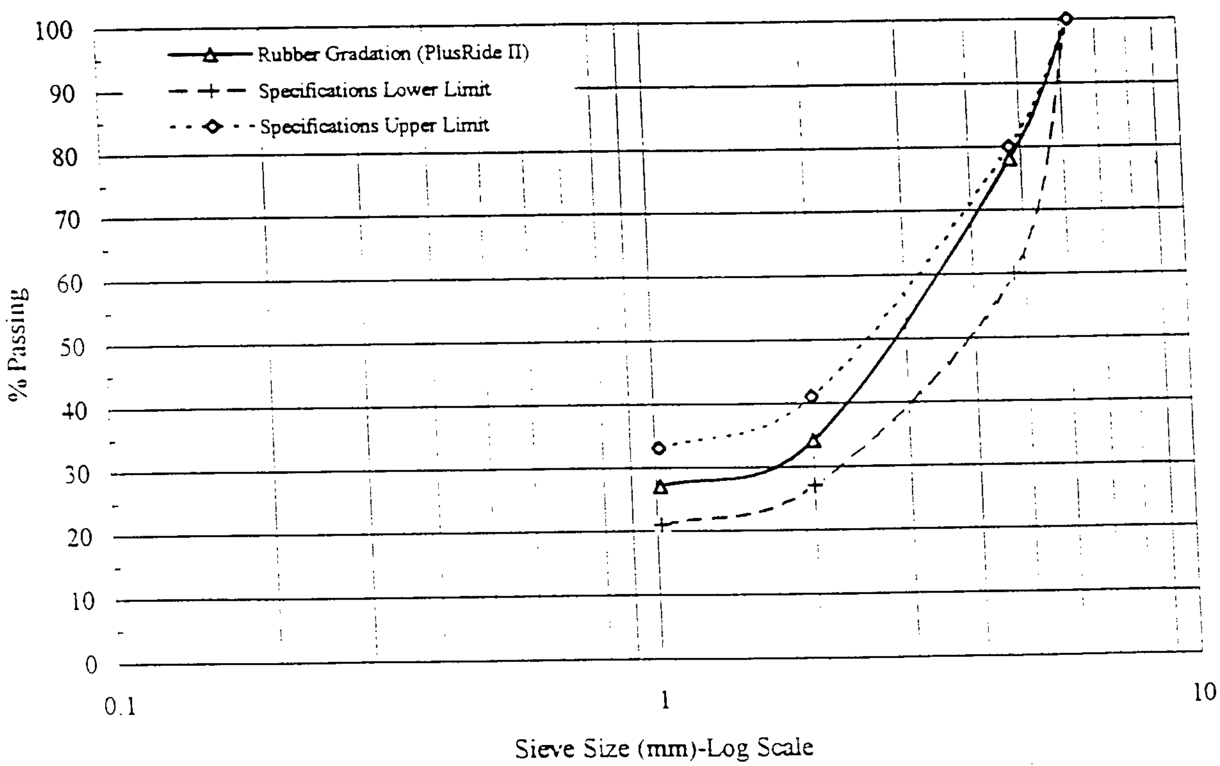


Figure 3.5 Rubber Gradation and NJDOT Specifications for PlusRide II

EXPERIMENTAL DESIGN AND TESTING

During the first phase of the study the following major areas were investigated: evaluation of the asphalt - rubber binder properties with mixing time so as to evaluate the reaction time of the modified binder and the effects of aging; design and analysis of conventional and asphalt-rubber mixtures using the Marshall method of mix design, and data collection on mixture behavior so as to investigate coupling of the Marshall method with mixture behavior and performance related variables; evaluation of the static indirect tensile and repeated-load indirect tensile characteristics of conventional and asphalt rubber with varying testing conditions, so as to identify the best testing method and testing conditions to be used in the development of the integrated mix design method. In the second phase of this study PlusRide II mixtures were prepared and tested following the same testing program of the wet process. Fatigue and rutting characteristics of the wet process mixture and PlusRide II were also investigated in this phase. In order to achieve these objectives the following testing was conducted.

Asphalt-Rubber Binder Experiments

In order to evaluate the asphalt rubber binder properties with increasing blending time the factorial of Table 3.9 was defined. The recommended mixing time for the asphalt rubber binder used in the wet process mixtures by NJDOT was 45 minutes. Thus, from the asphalt-rubber binder, prepared according to the laboratory method identified by Asphalt Rubber Systems Inc., samples at 20, 40, 45, 50, 60, and 90 minutes of mixing time were taken and tested. The first set of experiments included evaluation of the penetration, with the standard needle at 4°C (39.2°F) and 25°C (77°F) and cone at 25°C (77°F), kinematic viscosity at 177°C (350°F), softening point, flash point, and specific gravity. To be noticed that viscosity evaluation with the Brookfield viscometer was not possible at this time due to equipment constraints. The conventional binder properties were also evaluated for comparative analysis.

Table 3.9 Conventional and Asphalt Rubber Binder Factorial Experiment

Test	AC-10	Asphalt-Rubber Binder Mixing Time (minutes)					
		20	40	45	50	60	90
Penetration Test. Needle. AASHTO T-49 (ASTM D-5). At 25 °C (77 °F). 100g. and 5 Sec.	x	x	x	x	x	x	x
Penetration Test. Needle. AASHTO T-49 (ASTM D-5). at 4 °C (39.2 °F). 200g. and 60 Sec.	x	x	x	x	x	x	x
Cone Penetration Test. ASTM D-3205 at 25 °C (77 °F). 200g. and 5 Sec.	x	x	x	x	x	x	x
Kinematic Viscosity Test. AASHTO T-201(ASTM D-2170) at 177 °C (350 °F). Test Temperature	x	x	x	x	x	x	x
Softening Point Test. AASHTO T-53 (ASTM D-36)	x	x	x	x	x	x	x
Flash Point Test. AASHTO T-79 (ASTM D-3143)	x			x			
Specific Gravity Test. AASHTO T-228 (ASTM D-70)	x	x	x	x	x	x	x
Thin Film Oven Test AASHTO T-179 (ASTM D-1754)	x		x	x	x	x	

In order to examine the aging effects on the asphalt rubber binder penetration and viscosity on aged samples, Table 3.10, with the thin film oven test were conducted. For this investigation, samples at

Table 3.10 Factorial Experiment for Aged Samples

Test	AC-10	Asphalt-Rubber Binder Mixing Time (minutes)			
		40	45	50	60
Penetration Test. Needle. AASHTO T-49 (ASTM D-5). At 25 °C (77 °F). 100g. and 5 Sec.	x	x	x	x	x
Cone Penetration Test. ASTM D-3205 at 25 °C (77 °F). 200g. and 5 Sec.	x	x	x	x	x
Kinematic Viscosity Test. AASHTO T-201(ASTM D-2170) at 177 °C (350 °F). 350 F Test Temperature	x	x	x	x	x
Softening Point Test. AASHTO T-53 (ASTM D36)	x	x	x	x	x

Note all samples aged with Thin Film Oven Test AASHTO T-179 (ASTM D-1754)

40, 45, 50, and 60 minutes of mixing time were taken and tested in penetration, with the standard needle and cone at 25°C (77°F), kinematic viscosity at 177°C (350°F), and softening point.

Marshall Mix Design and Behaviors

In order to examine the design of the conventional and rubber modified mixtures, and generate the necessary data on mixture behavior and performance related variables to be used in improving the current Marshall method of mix design, the factorial of Table 3.11 was defined. Three sets of asphalt mixture specimens were prepared and tested: conventional mixture (dense graded for wearing course); asphalt-rubber mixture (wet process), and third with rubber-filled mixture (dry process). The samples, at different levels of binder content, were prepared according to the standard Marshall method, ASTM D-1559, with 75 blows per each face of the specimen. The 100mm (4") diameter by 62.5mm (2.5") height specimens were prepared according to the following steps: the aggregate was heated at 149°C (300°F); at the same temperature the aggregate was mixed with the asphalt cement to produce the conventional mixtures; the heated aggregate was mixed with the modified asphalt-rubber binder prepared at

Table 3.11 Marshall Design of Conventional and Modified Mixtures

Binder Content (% by Total Weight)	Conventional Mixtures	Asphalt-Rubber Mixtures (wet Process)	Rubber-Filled Mixtures (PlusRide II)
4.0	*		
4.5	*	*	*
5.0	*	*	*
5.5	*	*	*
6.0	*	*	*
6.5		*	

Note: three replicates n =3

177°C (350°F) to produce the asphalt-rubber mixtures (wet process); the heated aggregate was blended with 3 percent treated crumb rubber, by mixture weight (the crumb rubber was treated with 10 percent Sundex 790, by crumb rubber weight, prior of mixing with the aggregate) and the heated asphalt cement was then added and mixed at 149°C (300°F) to the produce asphalt-rubber mixtures (PlusRide II). The mixtures were compacted using the standard Marshall hammer with 75 blows per face at compaction temperatures of 300°F for both conventional and PlusRide II samples and 325°F for the wet process samples. the conventional specimens were allowed to cool at room temperature before extracting them from the molds, while the asphalt-rubber specimens (wet process and PlusRide II) were left in the molds for 15 hours before extracting them to avoid any specimen damage, swell, and/or cracking.

In order to study the behavior of the mixtures during Marshall testing the stability and flow characteristics need to be monitored with time. Thus, the mechanical dial gauges of the apparatus were replaced with Linear Variable Differential Transducers, (LVDTs). In this study, the proving ring mechanical dial gauge was replaced with an LVDT Model 351-0006, and with a sensitivity of 4.2873 VDC/ inch/Volt Input. The flow mechanical dial gauge was substituted with an LVDT, Model LD600-25, with sensitivity of 16.5 mV/mm/Volt Input. The LVDTs were connected to a data acquisition system and a power supply was used to produce the DC input voltage for the LVDTs. The data over time were collected with a data acquisition software, Notebook, at a sampling rate of 4 Hz (4 readings per second). The input voltage for the LVDTs was set at 25 Volts.

Static Indirect Tensile Testing

The tensile strength characteristics of the asphalt-rubber mixtures was looked at with the static indirect tensile test. In this experiment specimens of 100mm (4") diameter and 62.5mm (2.5") thickness were prepared with the materials and characteristics of the wet and dry process mixtures, previously described. Mixture compaction was similar to the one recommended in ASTM D-1561, and using the Material Testing System, MTS, machine with a compactor foot and round-nose steel rod locally fabricated. The mixture preparation and compaction steps follow: in the wet process, the aggregate was heated at 177°C (350 °F) and mixed with the asphalt rubber binder to produce mixtures at different binder content, see Table 3.12; in PlusRide II, the aggregate was heated to 149°C (300F) and blended with 3 percent of pre-treated crumb rubber, by mixture weight, before mixing it with the heated asphalt cement at the temperature of 177°C; both wet process and PlusRide II mixtures were cured at 60°C (140°F) for 15 hours, and then heated at 177°C (325°F) and 149°C (300°F), respectively, for compaction. First, one half of the mixture placed into the molds was rodded, 20 times in the center and 20 times around mold edges, and then the second half was added and rodded with the same technique. The specimens were then compacted with the MTS by applying 25 tamping blows at 36kPa (250 psi) followed by another 150 tamping blows at 72 Kpa (500psi) pressure on the total cross area of the specimen and by rotating the mold under the piston at 40° apart After compaction the specimens (wet process and PlusRide II) were kept at 60°C (140 °F) for 3 hours and a leveling load of 5701Kg (12600 Ib) was used. Finally, the specimens were allowed to cool for 15 hours before extracting them from the molds for avoiding cracking.

Table 3.12 Mixtures for Static Indirect Tensile Test Evaluation

Binder Content (% by total weight)	Asphalt-Rubber Mixture (Wet Process)	Rubber-Filled Mixture (PlusRide II)
4.0		*
4.5		*
5.0	*	*
5.5	*	*
6.0	*	*
6.5	*	

Note: * Three replicates n=3

Repeated-Load Indirect Tensile Testing and Experimental Design

In the effort to evaluate the effectiveness of repeated-load indirect tensile test for characterizing asphalt-rubber mixtures, so as to develop an integrated mix design procedure, the wet process and PlusRide II mixtures were prepared with the same technique as for the static indirect tensile test, and tested, see Table 3.13. The evaluation of the mixtures with the repeated-load indirect tensile test may be conducted at different testing conditions, ASTM D-4123. Based on the results of past studies and asphalt rubber mixtures with a loading frequency of 1 Hz, with a load duration of 0.1 seconds (representing the actual field loading conditions), and a resting period of 0.9 seconds were chosen. Two levels of loads were investigated with each asphalt rubber mixture: 10 percent and 30 percent of the indirect tensile strength, and two testing temperatures, of 5°C (41°F), and 25°C (77°F), see Table 3.14.

Table 3.13 Mixtures for Repeated Load Indirect Tensile Test

Binder Content (% by total weight)	Asphalt-Rubber Mixture (Wet Process)	Rubber-Filled Mixture (PlusRide II)
4.0		*
4.5		*
5.0	*	*
5.5	*	*
6.0	*	*
6.5	*	

Note: * Three replicates n=3

Table 3.14 Repeated-Load Indirect Tensile Testing Conditions

Load Frequency (HZ)	Load Duration (Sec)	Testing Temperature (°F)			
		41		77	
		Applied Load (% of INTS*)			
		10	30	10	30
1	0.1	x	x	x	x

* INTS = Indirect Tensile Strength x = three specimen n= 3

Diametrical Fatigue Test Specimens and Experiment Design

Fatigue behavior of both wet process and PlusRide II mixtures were investigated using the repeated-load diametrical fatigue test. Mixtures with the same technique as for the static indirect tensile test specimens were prepared and used for testing. Samples, 101.6mm diameter by 63.5mm height (4" diameter by 2.5" height), were prepared with binder contents ranging ± 0.5 percent around the Marshall optimum binder content, see Table 3.15. Fatigue life of asphalt mixtures may be conducted at different testing conditions. Based on past studies a loading frequency of 1 Hz with a load duration of 0.1 seconds (representing the actual field loading conditions), and a resting period of 0.9 seconds were chosen (similar to the repeated-load indirect tensile test conditions). Two levels of loads were investigated with each asphalt rubber mixture, 15 percent and 30 percent of the maximum static indirect tensile strength, at 25 °C (77 °F) testing temperature (representing the actual field in-service temperature). A seating load of 15 Lb was used during testing and a cumulative permanent horizontal deformation of 2.5 mm (0.1 in) was used as the failure criteria.

Table 3.15 Mixtures for Repeated Load Diametrical Fatigue Testing

Binder Content (% by total weight)	Asphalt-Rubber Mixtures (Wet Process)	Rubber-Filled Mixtures (PlusRide II)
Marshall Optimum + 0.5	*	*
Marshall Optimum	*	*
Marshall Optimum - 0.5	*	*

* Three specimens n =3

Repeated Unconfined Triaxial Creep Test and Experimental Design

Creep behavior of both wet process and PlusRide II asphalt-rubber mixtures were evaluated using the repeated-load unconfined triaxial test. Mixtures with the same technique as for the static indirect tensile test specimens were prepared and used for testing. Samples, 101.6mm diameter by 203.2mm height (4" diameter by 8" height), were prepared with the same binder content as for the fatigue test samples, Table 3.15. The samples were compacted in three layers using the MTS machine and the same compaction foot, locally fabricated, as recommended by ASTM D-1561. A Haversine with constant stress mode pulses with 0.5 Hz frequency and 0.2 sec load duration was used in compacting the samples. Each lift is compacted prior to adding the next. The first (i.e. the bottom) lift was compacted with 18 blows at 250 psi followed by another 90 blows at 500 psi. the second (i.e. the middle) was compacted with 24 blows at 250 psi followed by 120 blows at 500 psi and the third (i.e. the top) with 30 blows at 250 psi followed by 150 blows at 500 psi. In the beginning trial samples were compacted with different numbers of blows per each layer and the air void of each layer was evaluated. The above number of blows were found appropriate to obtain uniform density samples.

Based on early studies, the repeated-load creep test was conducted at two temperatures, 25 °C (77 °F) and 40 °C (104 °F), with two stress levels, 345 Kpa (50 psi) and 138 Kpa (20 Psi) respectively, see Table 3.16. Loading frequency of 1 Hz with a load duration of 0.1 seconds (representing the actual field loading conditions), and a resting period of 0.9 seconds were chosen similar to the repeated-load indirect tensile and fatigue testing conditions. Sample seating load of 10 Lb was used.

Table 3.16 Repeated-Load Unconfined Triaxial Creep Testing Conditions

Load Frequency (HZ)	Load Duration (Sec)	Testing Temperature °C (°F)	
		25 (77)	40 (104)
		Applied Stress, Kpa (psi)	
1	0.1	138 (20)	345 (50)
		*	*

* Three specimens n =3

CHAPTER 4. ASPHALT RUBBER BINDER AND MARSHALL RESULTS

INTRODUCTION

One of the objectives in incorporating rubber into asphalt mixtures is to improve the physical and mechanical properties. For asphalt rubber mixtures prepared with the wet process the asphalt cement is preblended with the rubber at high temperature and specific blending conditions. Binder, and consequently mixture properties, are enhanced if compatible materials and appropriate blending, mixing and curing conditions are used. In the wet-process, when sufficient blending time and heat are provided a partially polymer modified asphalt is achieved, in which rubber is slowly depolymerized. High degree of interaction between the asphalt and the rubber is thus desired in order to accelerate the depolymerization of rubber particles. Such interaction is influenced by several factors. For example, finer rubber gradations provide higher surface area and thus are more reactive with asphalt. Rougher rubber surface and with high natural rubber content or rubber hydrocarbons accelerates the rubber-asphalt interaction. Blending temperature also affects the rate of interaction, and typically a temperature between 149°C (300°F) to 204°C (400°F) is being used. As a result the time of blending, depending on the degree of this interaction, may vary with material composition and characteristics and blending conditions. Therefore, it is essential to examine the asphalt binder interaction for the specific combination of asphalt-rubber used. Thus, one of the objectives of this study was to examine the viscosity versus time relationship, and the effect of blending time on other binder properties. For this evaluation both modified and modified-aged binder samples were prepared according to the binder proportioning and specifications identified and used by NJDOT.

In the lack of improved and proven mix design methods several agencies, including NJDOT, use the traditional mix design method, Marshall. In this investigation conventional and asphalt rubber dense and gap graded mixtures were prepared and tested. The mixtures were prepared with typical materials and characteristics representing NJDOT mixtures. The asphalt rubber mixture (wet process) represents the surface course material used in experimental field sections on IH-95 and route I-295. NJDOT specifications for surface course aggregate and crumb rubber gradations are used in asphalt rubber mixture (dry process) preparation.

ASPHALT-RUBBER BINDER CHARACTERIZATION

The kinematic viscosity test results for aged and non-aged samples are shown in Table 4.1. Since the Brookfield Viscometer, suggested by FHWA for modified binder evaluation, was not available at the time of testing, additional standard binder tests were considered for verifying the results of kinematic viscosity. These tests included cone and standard needle penetration tests, and softening point. Tables 4.2 through 4.5 provide these test results at different blending times while Table 4.6 presents the specific gravity of non-aged samples. The plots of these data versus blending time are shown in Figures 4.1 through 4.4. As it can be seen from Figure 4.1, showing the effect of blending time on the kinematic viscosity of non-aged rubber asphalt binder, the viscosity increases during the first 45 minutes of blending. During this period the crumb rubber reacts with the asphalt cement, and the crumb rubber absorbs the aromatic oils (polymer swell). Such increase in viscosity has been also attributed to the swelling and adhesive characteristics of the crumb rubber modifier. The viscosity reaches its maximum value, while with blending time exceeding the 45 minutes the rubber starts to breakdown and the viscosity is significantly reduced. An equivalent effect was observed on the penetration results with the standard needle or cone as shown in Figures 4.2 and 4.3. As it can be seen from these Figures, penetration reaches a minimum at about 45 minutes of blending time. The softening point plot, Figure 4.4, indicates a maximum value at 45 minutes of blending. The effects of aging, see Tables 4.1 through 4.5, indicated that a higher penetration and lower viscosity than the non-aged samples, was obtained for the same blending time. This might be indicative of the binder deterioration with continuous exposure to heat, in this case representing the aging effects of production and laying operations, thus providing a softer binder, possibly in the degradation side of the reaction curve. In order to limit such effects shorter blending time, providing a partially reacted rubber and asphalt, might be appropriate so that aging effects during production and laying operations provides the desired binder properties and stiffness.

From a comparison of the properties of the conventional binder, AC-10, Table 4.7, and the asphalt rubber binder, it can be concluded that the penetration of the modified binder was reduced, while the viscosity and softening point increased, indicating that a harder binder was achieved, with potentially better rutting resistance potential.

Table 4.1 Kinematic Viscosity of Asphalt-Rubber binder (177 °C)

Asphalt-Rubber Binder Blending Time (Minutes)	Kinematic Viscosity mm^2/Sec (cSt)	
	Non-Aged Samples	Aged Samples
20	1183.2	-
40	1370.5	1277.2
45	1625.3	1552.1
50	1253.2	-
60	1187.8	-
90	1105.2	-

Table 4.2 Cone Penetration of Asphalt-Rubber Binder (200 gm, 5 sec., and 25 °C)

Asphalt-Rubber Binder Blending Time (Minutes)	Cone Penetration (0.1 mm)	
	Non-Aged Samples	Aged Samples
20	93.6	-
40	90.6	94.6
45	86.6	88.6
50	90.2	-
60	96.2	-
90	104.8	-

Table 4.3 Needle Penetration of Asphalt-Rubber binder

(100 gm, 5 sec., and 25 °C)

Asphalt-Rubber Binder Blending Time (Minutes)	Needle Penetration (0.1 mm)	
	Non-Aged Samples	Aged Samples
20	85.3	-
40	84.0	91.2
45	82.2	85.8
50	83.8	-
60	86.0	-
90	88.6	-

Table 4.4 Needle Penetration of Asphalt-Rubber Binder

(200 gm, 60 sec., and 4 °C)

Asphalt-Rubber Binder Blending Time (Minutes)	Needle Penetration of Non-Aged Samples (0.1 mm)
20	63
40	59
45	54
50	56
60	65
90	69

Table 4.5 Softening Point of Asphalt-Rubber binder

Asphalt-Rubber Binder Blending Time (Minutes)	Softening Point (°C)	
	Non-Aged Samples	Aged Samples
20	45.50	-
40	47.55	46.98
45	48.80	47.58
50	48.35	-
60	48.15	-
90	48.13	-

Table 4.6 Specific Gravity of Asphalt-Rubber Binder

Asphalt-Rubber Binder Blending Time (Minutes)	Specific Gravity of Non-Aged Samples
20	1.038
40	1.038
45	1.040
50	1.039
60	1.039
90	1.039

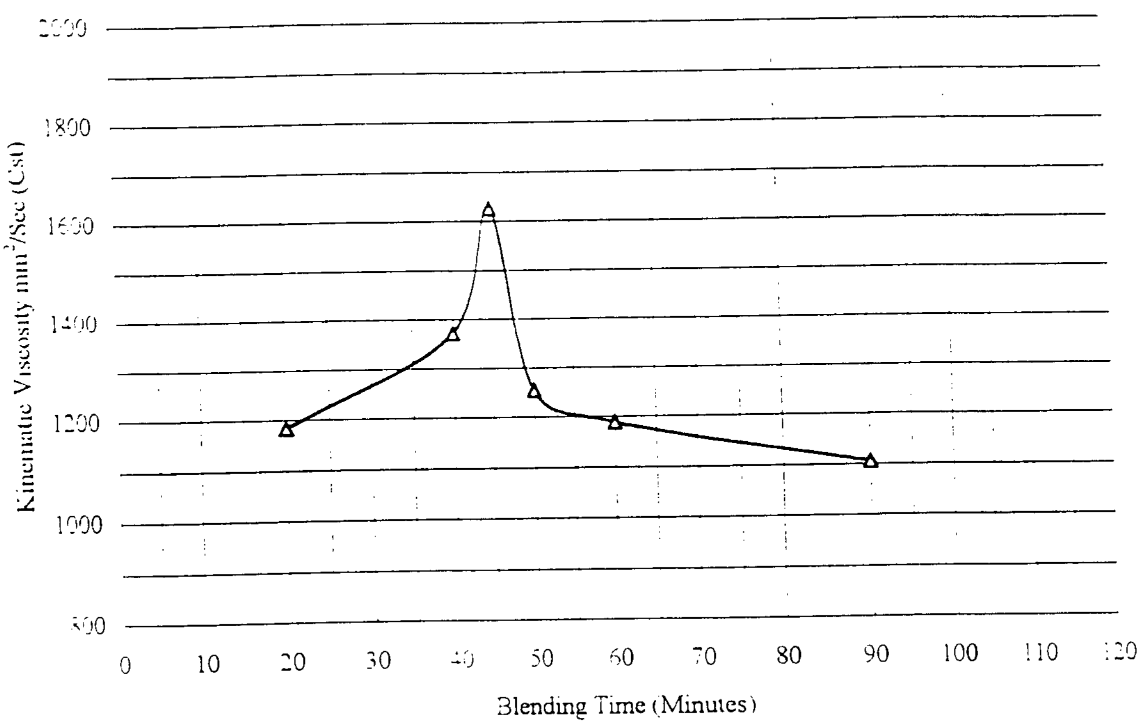


Figure 4.1 Kinematic Viscosity Versus Blending Time for Asphalt-Rubber Binder (177 ° C).

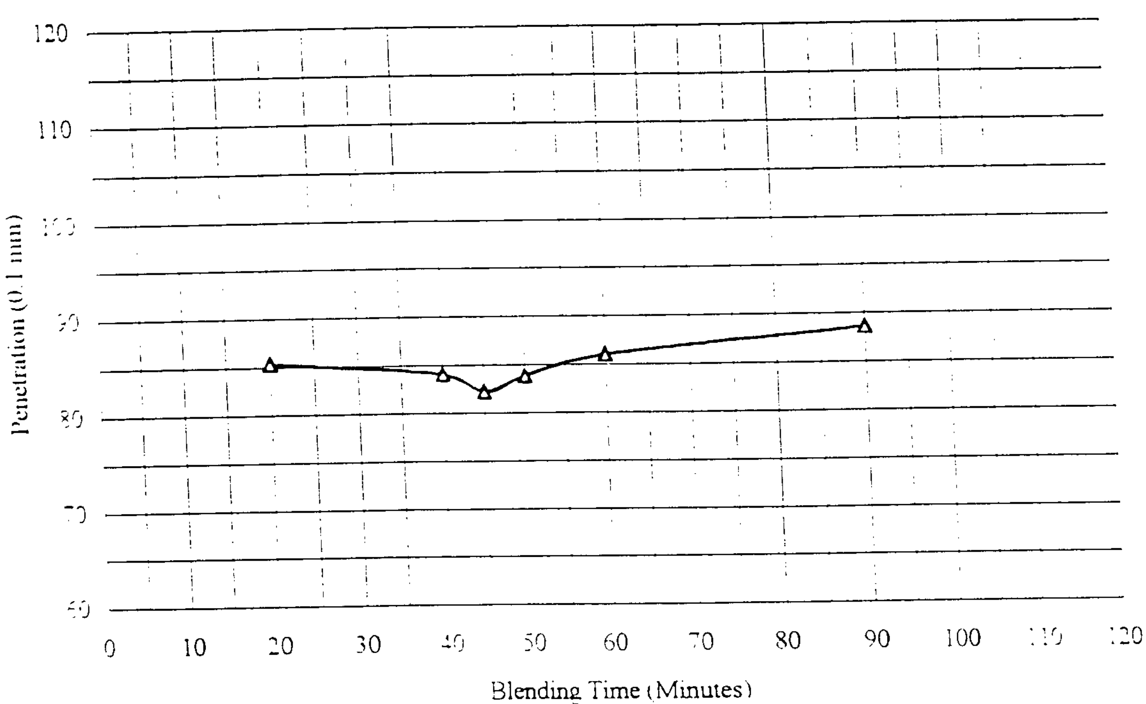


Figure 4.2 Needle Penetration Versus Blending Time for Asphalt-Rubber Binder (100gm, 5sec., and 25° C).

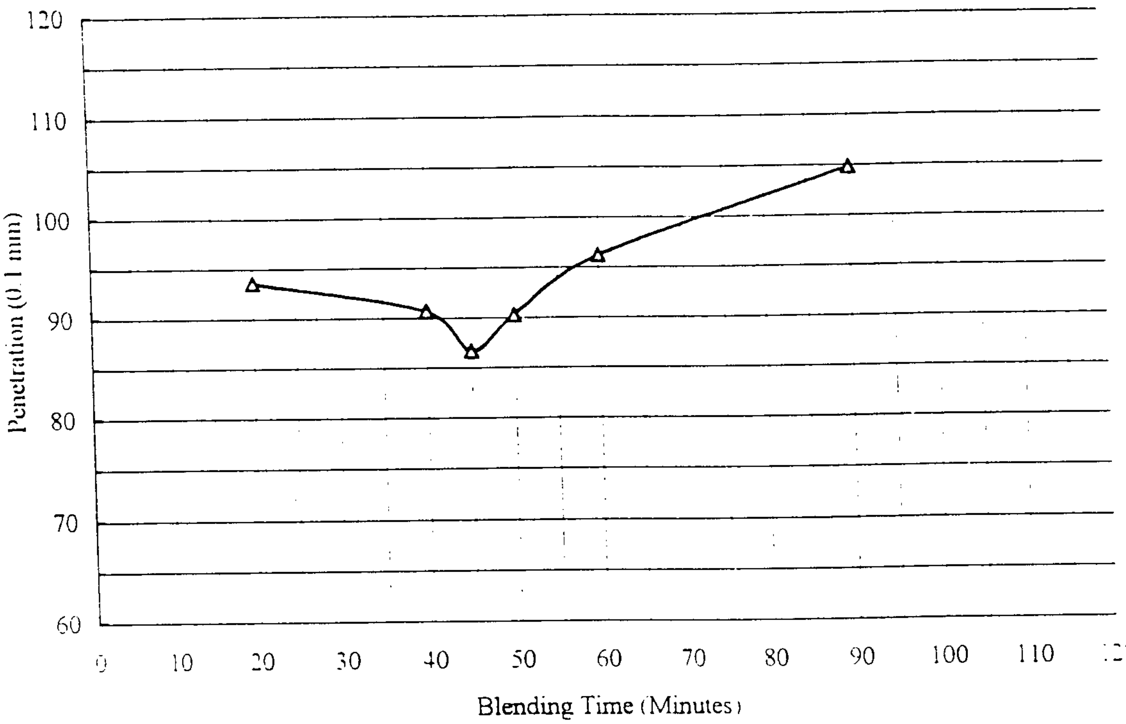


Figure 4.3 Cone Penetration Versus Blending Time for Asphalt-Rubber Binder (200gm, 5sec., and 25° C).

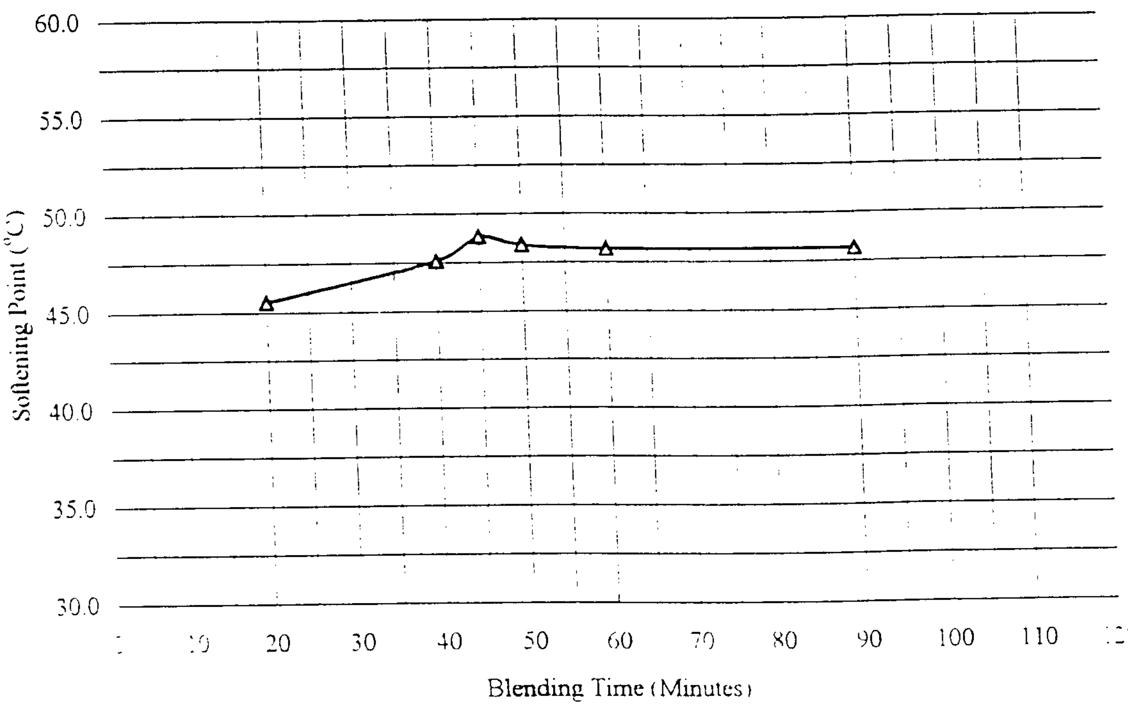


Figure 4.4 Softening Point Versus Blending Time for Asphalt-Rubber Binder.

Table 4.7 Asphalt Cement, AC-10, Characteristics

Test	Value
Non-Aged Samples	
Penetration Test, Needle, AASHTO T-49 (ASTM D-5), At 25 °C (77 °F), 100g, and 5 Sec.	98.20
Penetration Test, Needle, AASHTO T-49 (ASTM D-5), at 4 °C (39.2 °F), 200g, and 60 Sec.	46.00
Cone Penetration Test, ASTM D-3205 at 25 °C (77 °F), 200g, and 5 Sec.	103.40
Kinematic Viscosity Test, AASHTO T-201(ASTM D-2170) at 177 °C (350 °F), Test Temperature	39.13
Softening Point Test, AASHTO T-53 (ASTM D-36)	39.10
Thin Film Oven Test AASHTO T-179 (ASTM D-1754)	1.027
Aged Samples	
Penetration Test, Needle, AASHTO T-49 (ASTM D-5), At 25 °C (77 °F), 100g, and 5 Sec.	55.40
Cone Penetration Test, ASTM D-3205 at 25 °C (77 °F), 200g, and 5 Sec.	57.00
Kinematic Viscosity Test, AASHTO T-201(ASTM D-2170) at 177 °C (350 °F), 350 F Test Temperature	51.64
Softening Point Test, AASHTO T-53 (ASTM D36)	44.10

MARSHALL TEST RESULTS AND ANALYSIS

The Marshall test results for the conventional and asphalt-rubber mixtures are shown in Tables 4.8, while Figures 4.5 through 4.10 illustrate the effects of binder content on stability, mixture density, air voids, voids in the mineral aggregate, flow and voids filled with binder. According to the standard Marshall method, the optimum binder content, based on stability, 4% air voids and density, is found to be 5.03, 6.13, and 5.0 percent by the total weight of the mixture, for the conventional, the wet process, and the dry-process asphalt rubber mixtures, respectively. Mixture characteristics at the optimum binder content are shown in Table 4.9. The optimum binder content of the asphalt rubber mixture (wet process) is higher than the conventional one since a higher viscosity asphalt rubber binder is being used in relation to the conventional asphalt cement. With a higher viscosity binder, the coated film is expected to be thicker. Since the same aggregate type and gradation is being used for both conventional and asphalt rubber mixture (wet process) and with the same aggregate surface area, a higher percentage of asphalt-rubber binder is required for coating the aggregate. Also, the thicker binder film with the higher viscosity will result in a lower density for the asphalt rubber mixture (wet process). A mixture with higher binder content will provide a lower mixture density since the specific gravity of the binder in relation to the aggregate is lower, 1.06 versus 2.61 respectively. In addition, the use of higher viscosity asphalt rubber binder makes it more difficult to fill with the binder additional voids during compaction. The results of density, voids filled with binder and air voids of the conventional and asphalt-rubber mixtures (wet process) reinforce this phenomenon. The thicker coating film of asphalt-rubber binder will also decrease the aggregate interlock between the aggregate particles, resulting thus in a lower stability, and higher flow.

Aggregate gradation is the most important difference between the conventional and asphalt rubber mixture (dry process). To provide space for the rubber particles, it is necessary to create a gap in the aggregate gradation curve, primarily in the 3.18 mm (1/8") to 6.25 mm (1/4") size range, see aggregates characteristics in chapter 3, Figure 3.3. The rubber particles replace a portion of the aggregate particles that normally occupy the same size range. This makes the density of the asphalt rubber mixture (dry process) lower than that of the conventional mixtures since the specific gravity of the crumb rubber particles is much lower than that of the aggregate particles. The crumb rubber particles make the asphalt rubber mixtures (dry process) more compressible during Marshall testing, creating thus higher flow. The aggregate particles interlock reduction, due to the gradation gap, makes the stability lower than that of both conventional and asphalt rubber (wet process) mixtures. Also, the rubber particles elasticity increases the sample bounce back during compaction creating higher air voids than that of the conventional mixture. The optimum binder contents of the conventional and asphalt-rubber mixture (dry process) are almost the same since the aggregate mixture coated surface areas and the asphalt grade are similar.

Table 4.8 Marshall Test Results for Conventional and Asphalt-Rubber Mixtures

a. Conventional Mixtures

Binder Content (%)	Density (Kg/m^3)	Air Void (%)	VMA (%)	Voids Filled with Binder (%)	Stability (N)	Flow (0.25 mm)
4.0	2334	5.9	14.2	58.8	8305	14.6
4.5	2350	4.5	14.1	68.1	8996	16.9
5.0	2362	3.3	14.1	76.5	7031	18.8
5.5	2374	2.1	14.1	85.0	7018	19.6
6.0	2385	1.0	14.2	93.2	7611	21.5

b. Asphalt-Rubber Mixtures (Wet Process)

Binder Content (%)	Density (Kg/m^3)	Air Void (%)	VMA (%)	Voids Filled with Binder (%)	Stability (N)	Flow (0.25 mm)
5.0	2257	8.0	17.9	55.7	5946	18.0
5.5	2271	6.7	17.9	62.3	6258	22.7
6.0	2288	5.4	17.7	69.4	7493	23.4
6.5	2328	3.1	16.7	81.6	6323	25.3

c. Asphalt-Rubber Mixtures (Dry Process)

Binder Content (%)	Density (Kg/m^3)	Air Void (%)	VMA (%)	Voids Filled with Binder (%)	Stability (N)	Flow (0.25 mm)
4.5	2292	4.0	19.5	70.1	6522	23.2
5.0	2272	4.2	20.7	71.4	6660	32.4
5.5	2259	4.1	21.6	73.9	5385	36.4
6.0	2239	4.2	22.7	74.9	5304	35.6

Table 4.9 Mixture Characteristics at Optimum Binder Contents

Mixture	Binder Content (%)	Density (Kg/m^3)	Air Void (%)	VMA (%)	Voids Filled with Binder (%)	Stability (N)	Flow (0.25 mm)
Conventional	5.03	2370	3.3	14.1	76.5	8550	18.3
Wet Process	6.13	2305	4.5	17.3	73.2	6750	24.2
PlusRide II	5.00	2280	4.1	20.5	72.0	6660	27.0

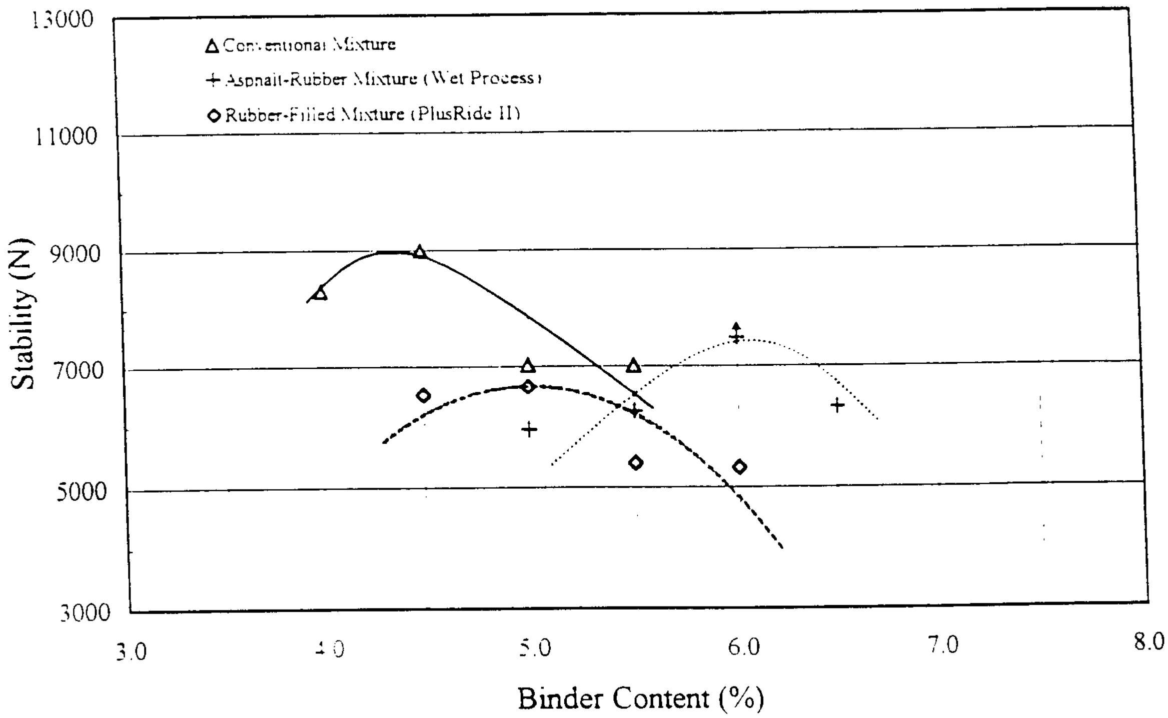


Figure 4.5 Stability Results for Conventional and Asphalt-Rubber Mixtures

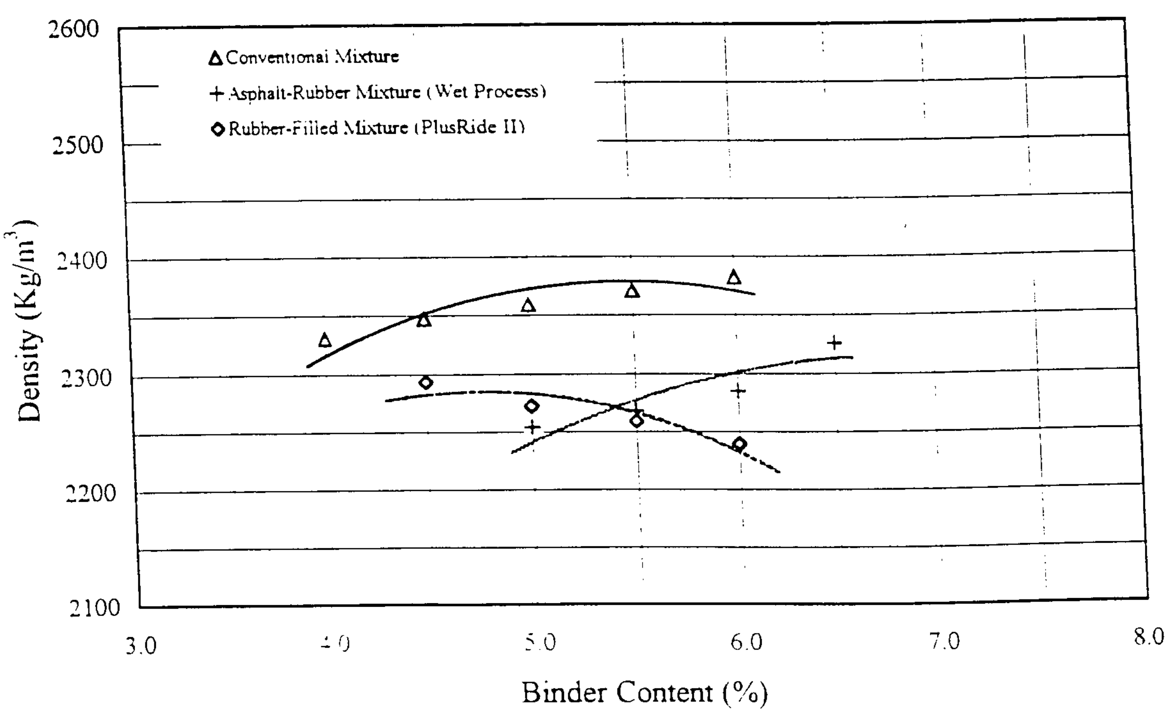


Figure 4.6 Density of Conventional and Asphalt-Rubber Mixtures

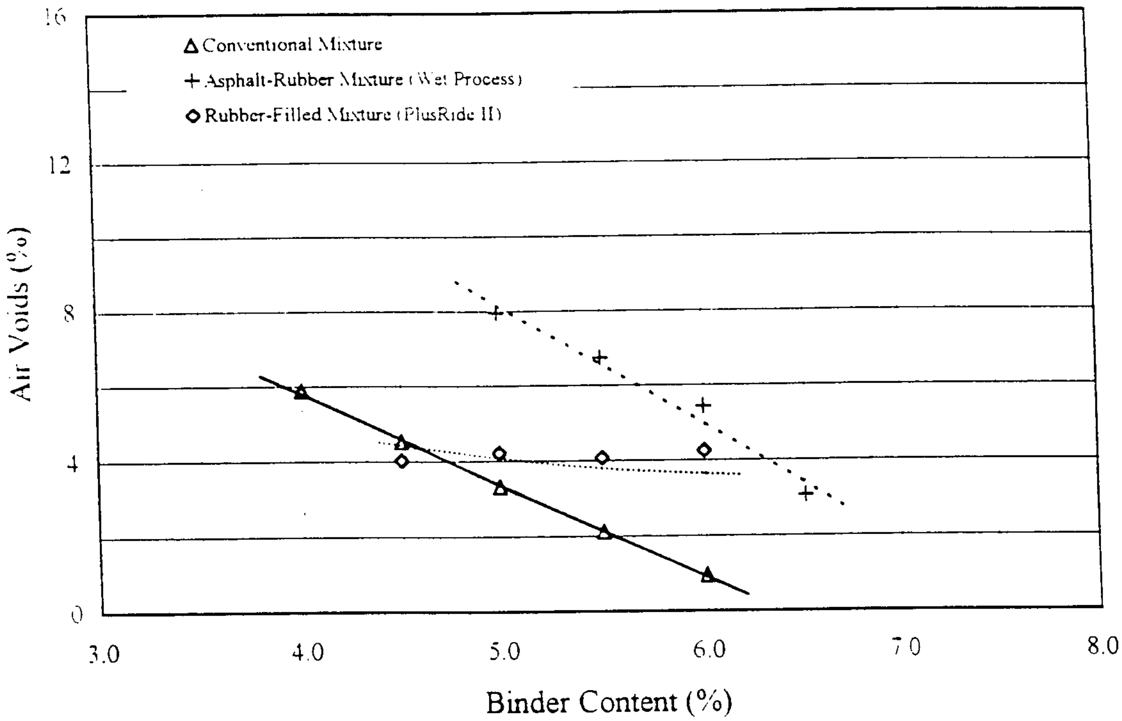


Figure 4.7 Air Void Content of Conventional and Asphalt-Rubber Mixtures

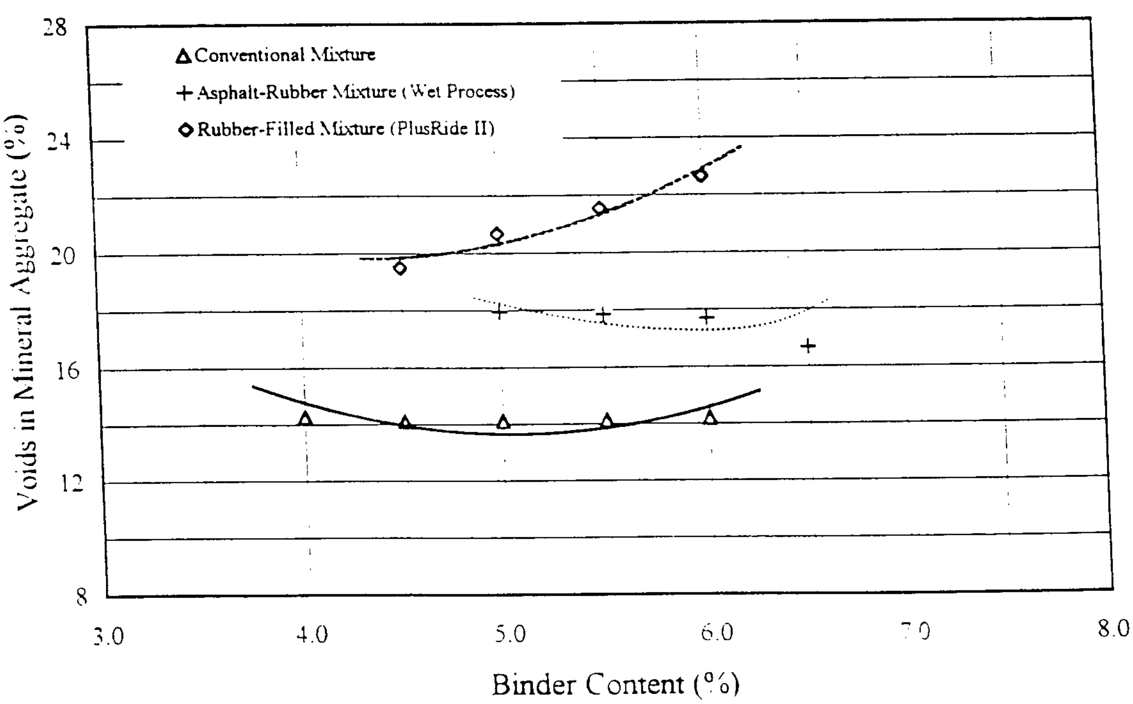


Figure 4.8 Void in Mineral Aggregate of Conventional and Asphalt-Rubber Mixtures

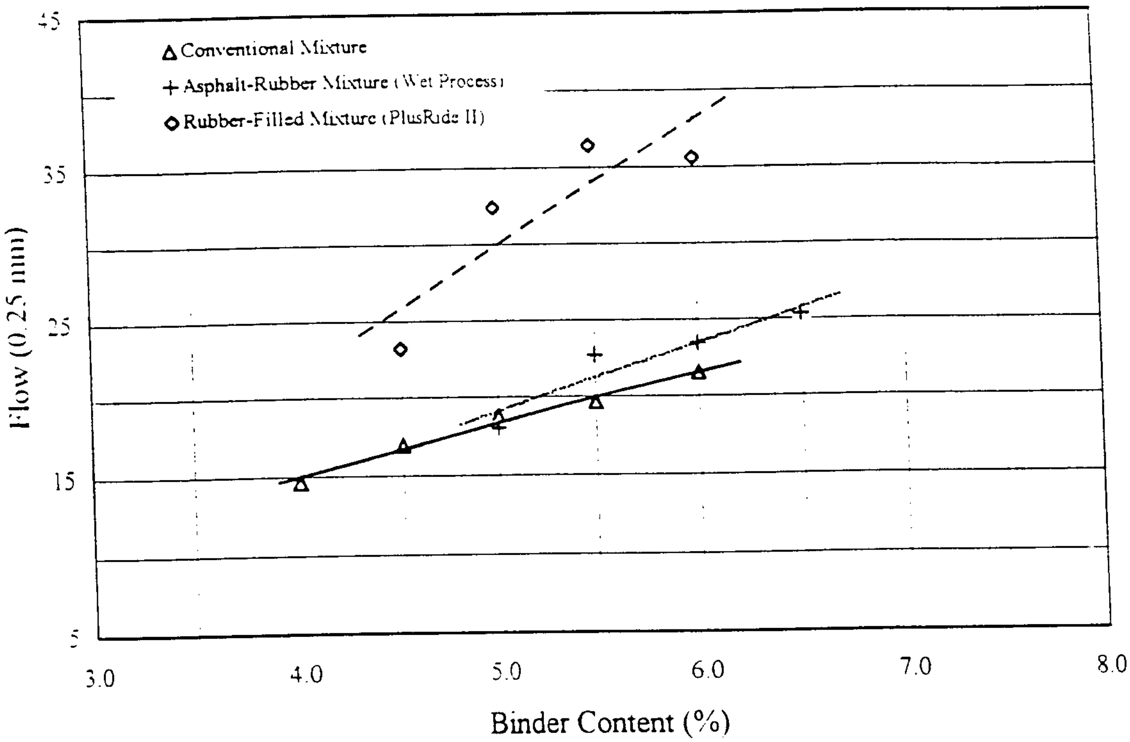


Figure 4.9 Flow Results of Conventional and Asphalt-Rubber Mixtures

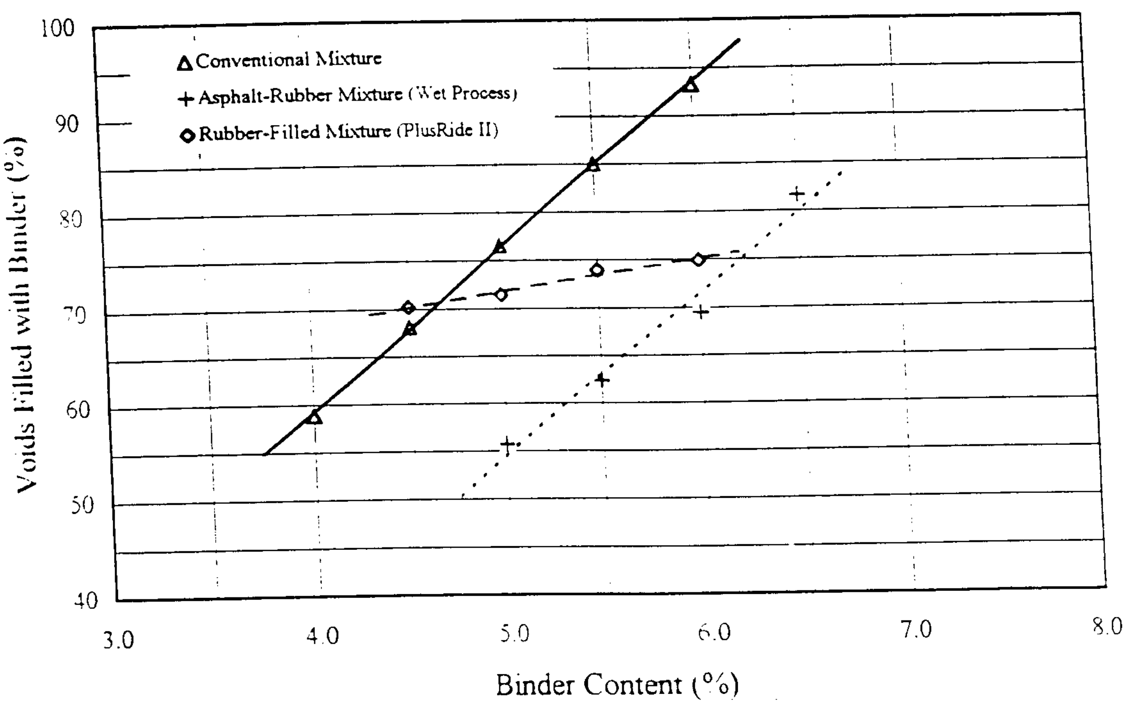


Figure 4.10 Voids Filled with Binder of Conventional and Asphalt-Rubber Mixtures

CHAPTER 5. MIXTURE BEHAVIOR AND ENHANCEMENT OF MARSHALL MIX DESIGN METHOD

INTRODUCTION

Mixture design establishes the proportion of binder to aggregate that produces a paving mixture that will serve the longest possible time without serious pavement distress. Mixture design is often a compromise between high stiffness, (which insures strength and resistance to permanent deformation), and flexibility, (which aids fatigue and fracture resistance). Several methods have been developed over the years to design asphalt mixtures, with the Marshall method becoming the most popular. This mix design methodology evolved by the Corps of Engineers from an effort initiated at the U.S. Army Engineering Tulsa District in the period 1941 to 1944, for addressing the design of asphalt pavement mixtures for heavy bombers. Subsequently, the effort was assigned to the U.S. Army Engineer Waterways Experiment Station, and attention focused on the Hubbard-Field and the Marshall procedures (Brown 1989). While the latter method was adopted by the Mississippi Highway Department, the Marshall method was recommended for specific purpose since, among other: it provided satisfactory results in the few years it had been in use; the testing apparatus was portable and could be readily used in the field and adapted to the existing California bearing ratio (CBR) testing equipment; and the method measured asphalt properties similar to those measured by the Hubbard-Field apparatus;

The conventional Marshall apparatus has two mechanical dial gauges. One is attached to the proving ring to measure the stability while the second is attached to the Marshall breaking head to record the specimen flow. With this setup, only the maximum values of the test output can be recorded. In the effort to couple the Marshall results with mixture behavior parameters, for selecting an optimum mixture, the testing output should be monitored over the entire testing period. Thus, the Marshall apparatus was modified with the data acquisition system and LVDTs, as explained in Chapter 3. Some of the mixture behavior parameters that might be able to monitor, and potentially use for enhancing the Marshall method results, include: mixture toughness and stiffness, and the rate of absorbed energy.

STIFFNESS OF CONVENTIONAL AND ASPHALT-RUBBER MIXTURES

Stiffness is the relationship between stress and strain as a function of time of loading and temperature. In many applications of asphalt mixtures, stiffness characteristics must be known not only to assess the behavior of the mix itself, but also to evaluate the performance of an engineering structure of which the mix is a part. At high service temperature, increased binder stiffness is desirable, while at low service temperature high binder stiffness is primarily responsible for low temperature cracking.

Among the most frequent methods that were used in evaluating mixture stiffness is the axial, diametral, and flexural repeated-load tests. In this stage of the study, and in an effort to enhance mixture selection with the Marshall mix design methodology, stiffness analysis was used (with the results from both conventional and asphalt-rubber mixtures). In order to be able to evaluate mixture stiffness the stress strain relationship is needed. The specimen flow, monitored at any time during testing, divided by the specimen diameter may represent an average mixture strain, since the strain might not be equal due to non-uniform stress applied over the contact area of the breaking head. On the other hand the stability divided by the contact area of the Marshall breaking head, may be possibly thought as an equivalent uniform stress distributed over the contact area between the head and the sample. Thus, at any time during testing, the calculated stress over the determined strain represents the asphalt mixture stiffness given by:

$$S_{mix}(t) = \sigma/\varepsilon$$

where: $S_{mix}(t)$ is the asphalt mixture stiffness at time t, Kpa; σ is the calculated Stress, (Stability / breaking head contact area) at time t, Kpa; and ε is the strain, (specimen flow/specimen diameter, 101.6 mm) at time t, mm/mm.

The stiffness results versus time of loading for conventional and asphalt-rubber mixtures are shown in Figures 5.1 though 5.3, for different binder contents. For both conventional and modified mixtures, stiffness value increases with increasing loading time, up to a maximum, after which the stiffness decreases. In addition, as it can be seen from these Figures, the stiffness of conventional and modified mixtures of the optimum binder content have present a higher value. The conventional mixture has about 1.6 times higher stiffness than the asphalt rubber mixtures indicating that modified mixtures may have a higher flexibility and consequently may provide higher resistance to fatigue and low temperature cracking than the conventional. The average values (replicates n=3) of maximum stiffness at different binder content for the conventional and asphalt-rubber mixtures, are shown in Table 5.1.

A similar method empirically determined for defining mixture stiffness is based on Marshall stability and flow (Baladi 1988):

$$S_{mix} = S/[2F_{(0.5\,S)}]$$

where: S_{mix} represents the mixture stiffness (N/mm); S is the Marshall stability (N); and $F_{(0.5\,S)}$ is the flow at 50 percent Stability (mm). The maximum stability values, and the flow at 50 percent of stability for each sample were used in evaluating stiffness. The average maximum values for both conventional and asphalt rubber mixtures are given in Table 5.2, for the various binder contents. Similarly to the previous method of evaluating mixture stiffness, the results indicate that both the conventional and asphalt rubber mixtures had the Maximum stiffness close to their optimum binder content. The stiffness of the asphalt rubber mixture was lower than that of the conventional at the same binder content, with the exception of the 6.0% binder content representing the optimum level for the asphalt-rubber mixture (wet process). With this method of calculating stiffness, the conventional mixture had about 1.8 times higher stiffness than the asphalt rubber mixtures.

The Stiffness values versus binder content for these two methods were plotted in Figures 5.4 and 5.5. In both cases the maximum stiffness for the conventional mixtures is obtained at about 4.5% binder content, while for the asphalt rubber mixtures, wet process and PlusRide II, at about 6.0% and 5.08 %, respectively. Thus, the use of either method for evaluating mixture stiffness is expected to provide the same results.

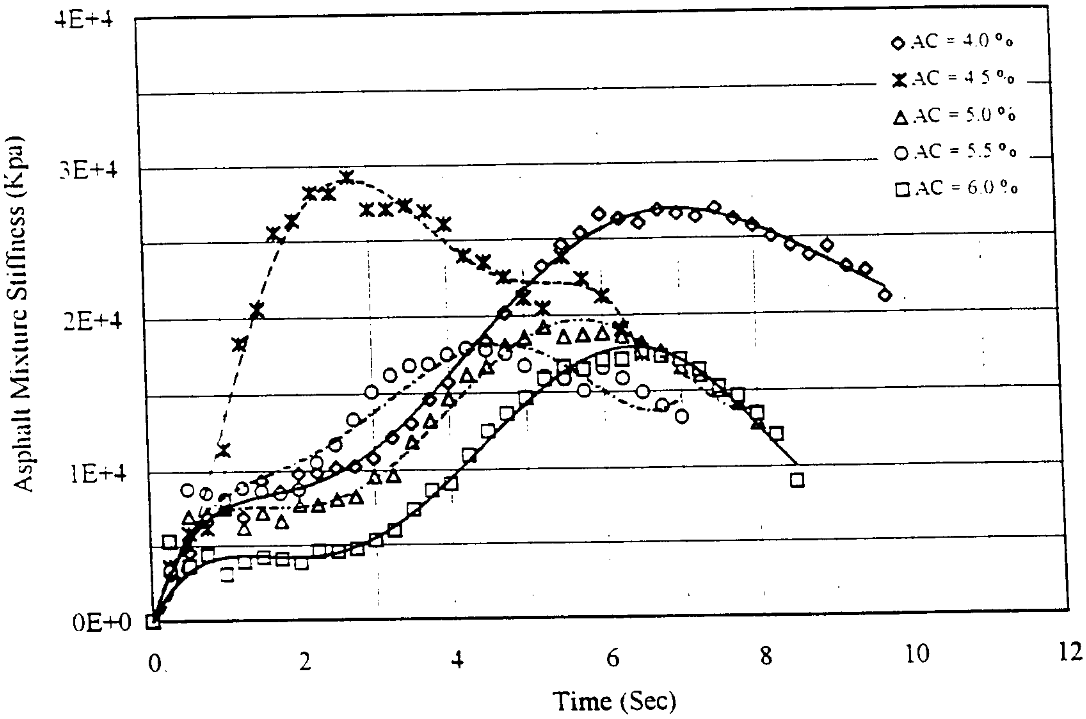


Figure 5.1 Stiffness Versus Time for Conventional Mixtures

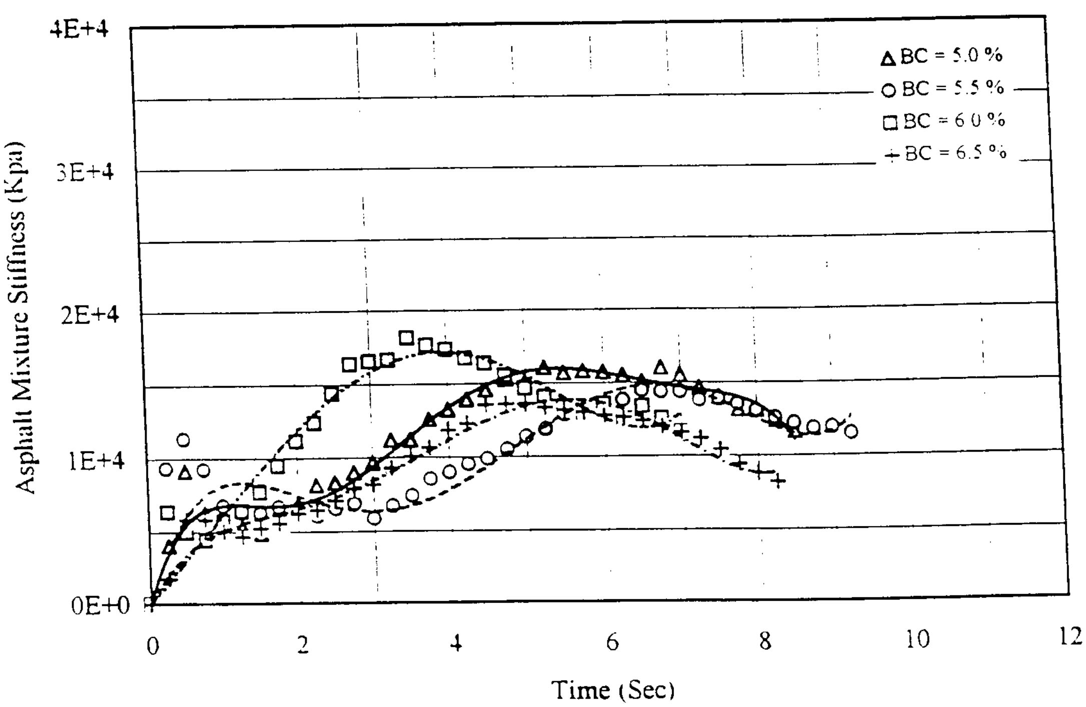


Figure 5.2 Stiffness Versus Time for Asphalt-Rubber Mixtures (Wet Process)

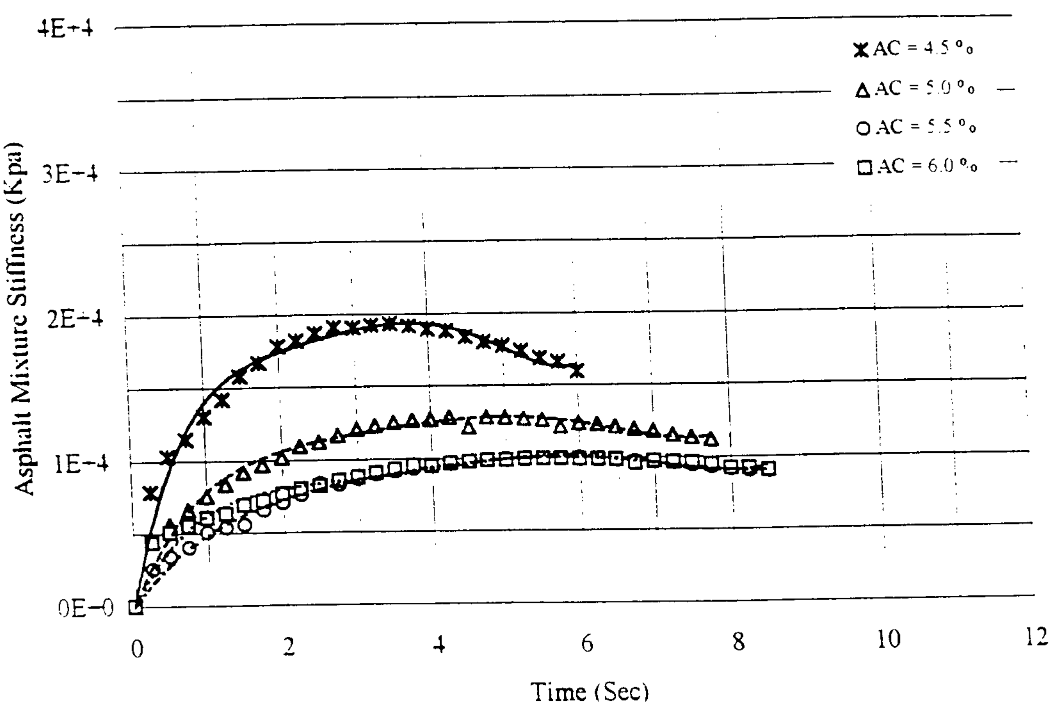


Figure 5.3 Stiffness versus Time for Rubber-Filled Mixtures (PlusRideII)

Table 5.1 Average Maximum values of Stiffness for [$S_{mix}(t) = \sigma/\varepsilon$]

Binder Content (%)	Asphalt Mixture Stiffness (Kpa)		
	Conventional	Wet Process	PlusRide II
4.0	2.697E+4	-	-
4.5	2.928E+4	-	1.360E+4
5.0	1.922E+4	1.600E+4	9.944E+3
5.5	1.789E+4	1.442E+4	6.832E+3
6.0	1.758E+4	1.809E+4	7.416E+3
6.5	-	1.356E+4	-

Table 5.2 Average Maximum values of Stiffness for [$S_{mix} = S/2F_{(0.5S)}$]

Binder Content (%)	Asphalt Mixture Stiffness (N/mm)		
	Conventional	Wet Process	PlusRide II
4.0	1754	-	-
4.5	2283	-	1530
5.0	1231	1129	1216
5.5	1270	939	769
6.0	1080	1256	805
6.5	-	879	-

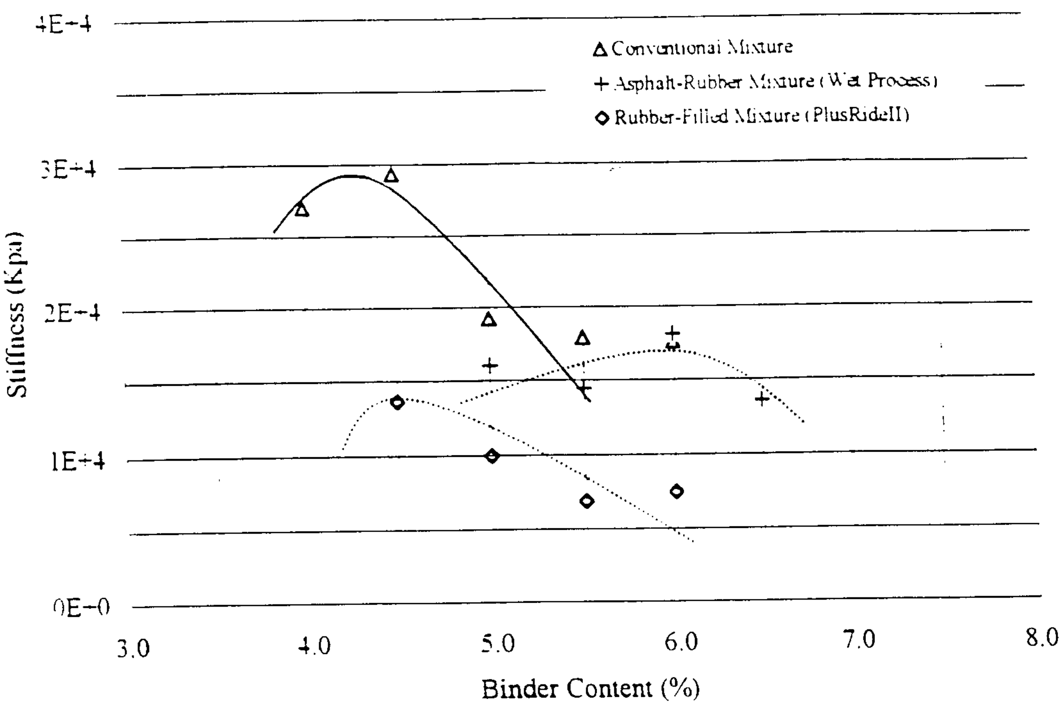


Figure 5.4 Average Maximum Values of Stiffnesses Versus Binder Content for [S_{mix} (t) = σ/ε]

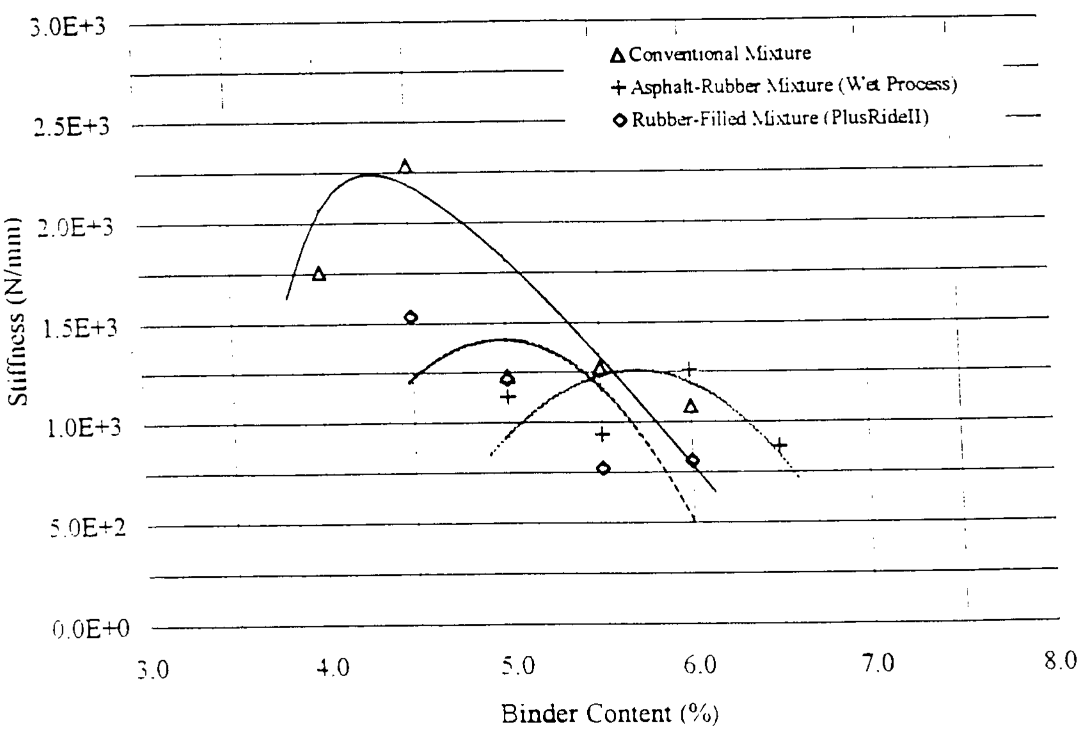


Figure 5.5 Average Maximum Values of Stiffness Versus Binder Content for [S_{mix} = $S/2F_{(0.5\ S)}$]

MIXTURE TOUGHNESS

Toughness of mixtures was also considered in evaluating mixture behavior. Toughness may be defined as the amount of force required to fail a unit volume of mixture. The force, measured in Kpa mm/mm, can be obtained by the area under the stress-strain relationship. The binder content producing the maximum Toughness could eventually be selected as one of the parameters for selecting the optimum binder design value. The relationships between load over the contact area of the Marshall breaking head sample, and the flow over the specimen diameter, for both conventional and asphalt-rubber mixtures, was plotted for the different binder contents as shown in Figures 5.6 through 5.8.

The average toughness values for both conventional and asphalt-rubber mixtures at different binder levels are provided in Table 5.3. As it can be seen from Table 5.3, mixture toughness increases with binder content up to a certain value, after which a decline is observed. It appears that for both conventional and asphalt-rubber mixtures the higher toughness values are observed in the proximity of the Marshall optimum binder content. This becomes evident from Figure 5.9, where the maximum toughness of the conventional mixture, 20 Kpa, is found at an asphalt content of 5.1%, while for asphalt-rubber mixtures, wet process and PlusRide II, the maximum toughness of 22.4 Kpa and 41.2 Kpa were observed at a binder content of 6.1% and 5.22%, respectively. Around the optimum binder content of the conventional and asphalt rubber mixtures, the behavior of the mixtures is similar at low levels of load, while at higher levels both wet process and PlusRide II mixtures resist higher deformation before failure. Thus, even thought the modified mixtures (wet process and PlusRide II) have higher toughness are able to resist higher deformation possibly due to higher elasticity of asphalt rubber binder and rubber aggregate for both wet process and PlusRide II mixtures, respectively. Thus it is expected that asphalt rubber mixtures might be able to absorb higher energy before failure than the conventional one.

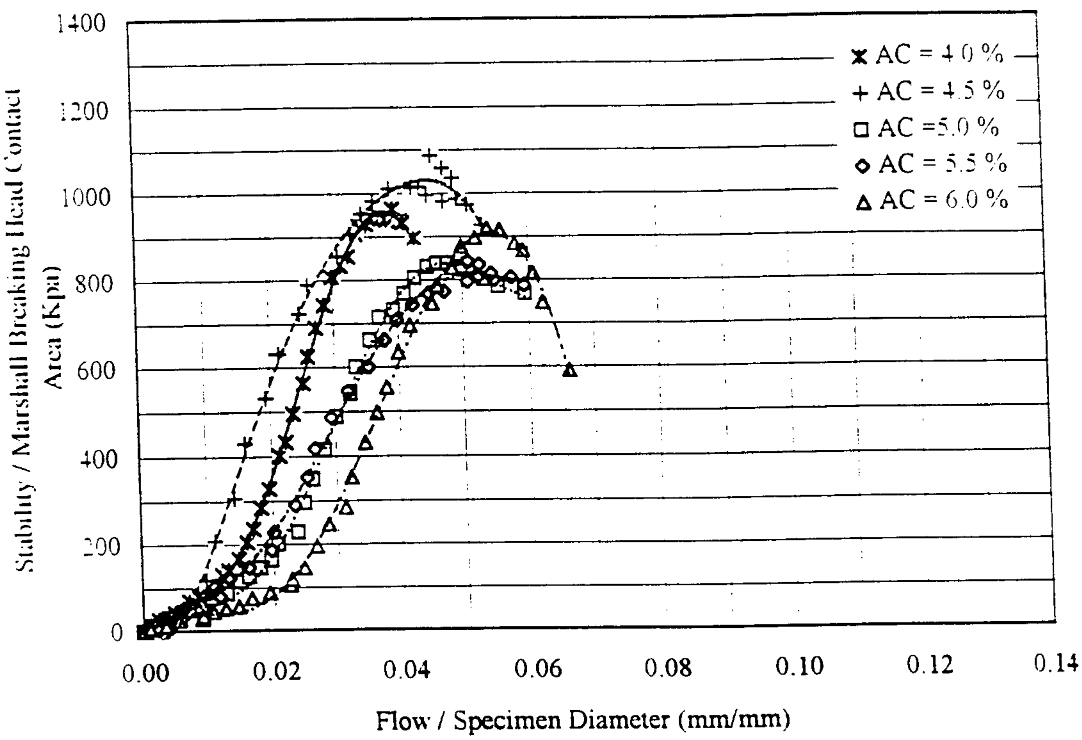


Figure 5.6 Load/Contact Area versus Flow/Diameter for Conventional Asphalt Mixtures

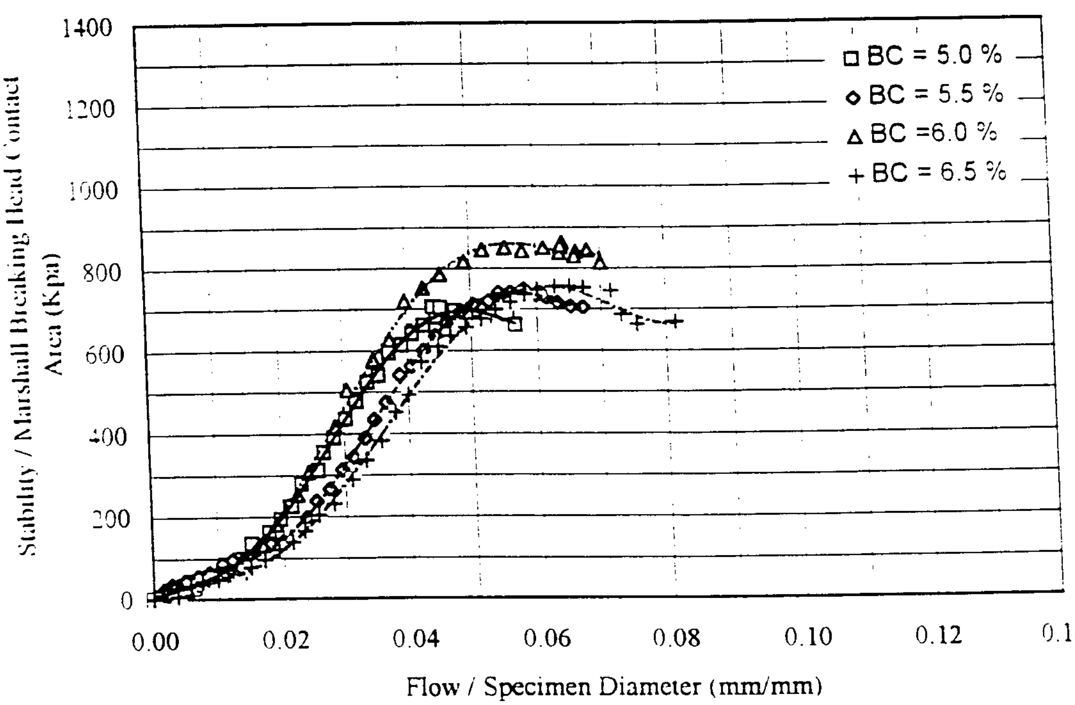


Figure 5.7 Load/Contact Area versus Flow/Diameter for Asphalt-Rubber Mixtures (Wet Process)

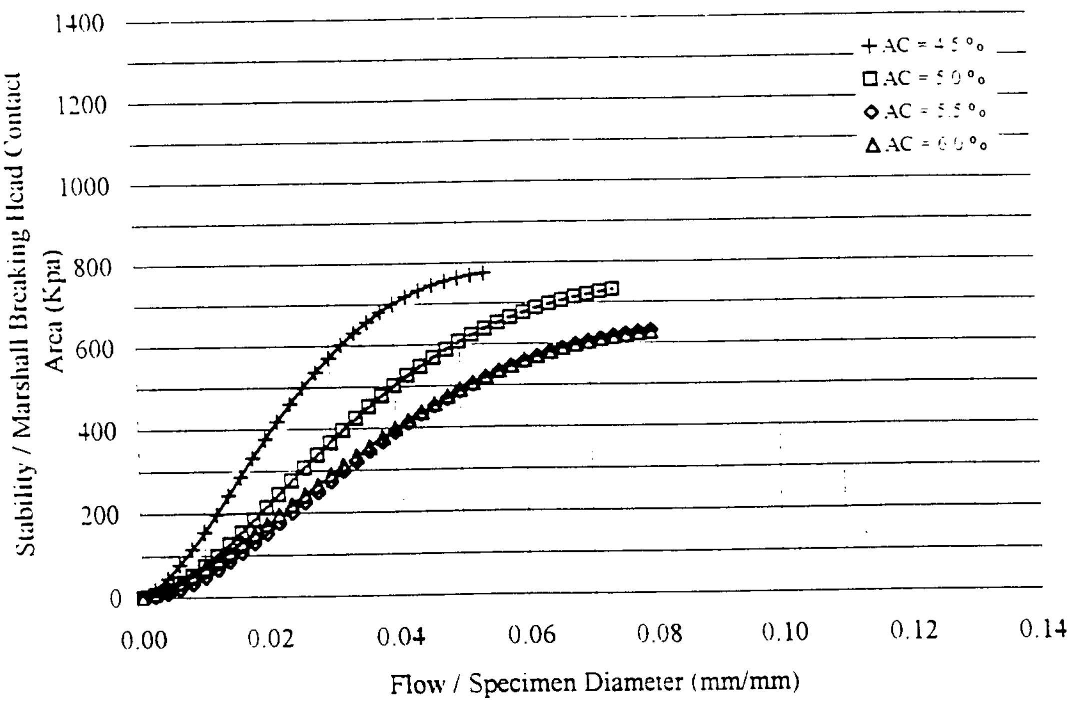


Figure 5.8 Load/Contact Area versus Flow/Diameter for Rubber-Filled Mixtures (PlusRide II)

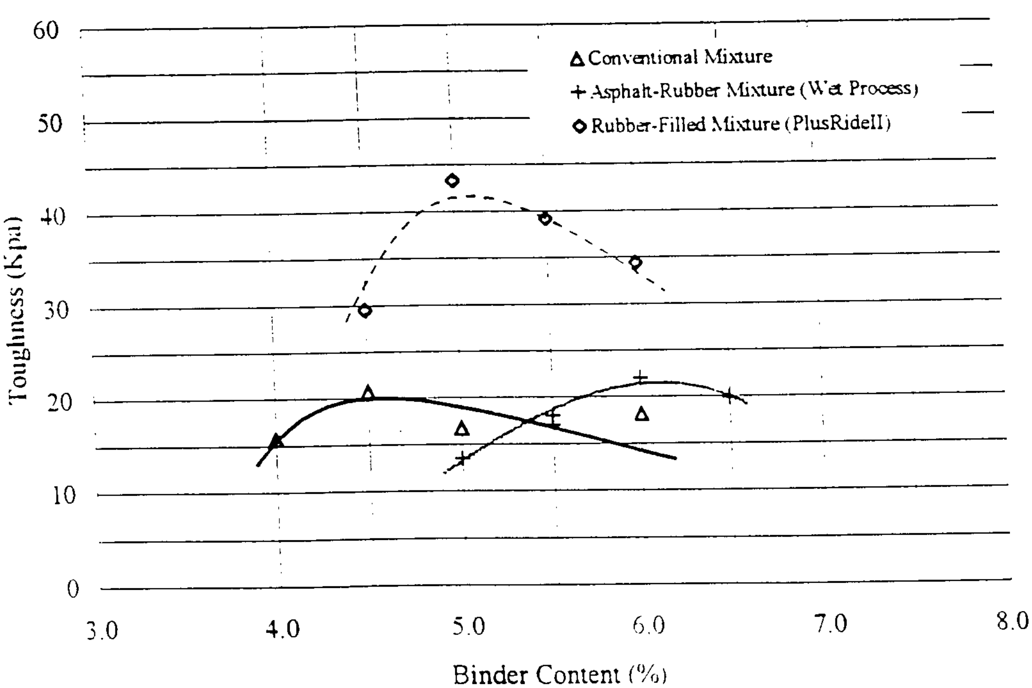


Figure 5.9 Mixture Toughness Versus Binder Content

Table 5.3 Average Toughness Values

Binder Content (%)	Asphalt Mixture Toughness (Kpa)		
	Conventional	Wet Process	PlusRide II
4.0	16	-	-
4.5	21	-	29
5.0	17	13	43
5.5	18	18	39
6.0	18	22	34
6.5	-	20	-

CUMULATIVE ENERGY ABSORBED

The absorbed energy of a mixture may be represented by the cumulative values of toughness during loading. The relation between loading time and absorbed energy for both conventional and modified mixtures are displayed in Figures 5.10, 5.11, and 5.12 where the relationships between cumulated toughness and Marshall loading time at different binder content are shown. Higher the slope of this relationship higher the rate of absorbed energy (cumulated toughness). As it can be seen from these Figures, for the conventional mixtures the rate of absorbed energy increases with increasing binder content up to 4.5 %, after which the rate of absorbed energy decreases. Similarly, the rate of absorbed energy for the asphalt rubber mixtures increases up to a binder content of 6.0% and 4.5% for the wet process and the PlusRide II, respectively, after which a decrease is observed. As it can be seen from these Figures, at Marshall optimum binder contents, the rate of absorbed energy of asphalt rubber mixtures is higher than that of the conventional, indicating that asphalt rubber mixtures have the ability to resist higher loading rates and longer loading duration due to the increased mixture flexibility.

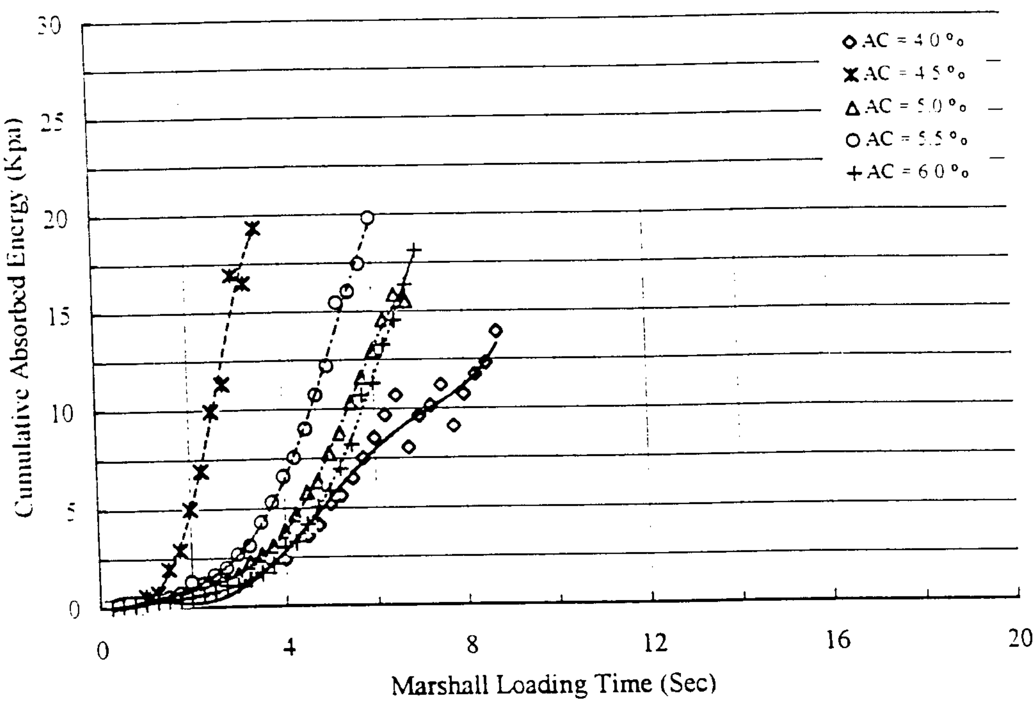


Figure 5.10 Cumulative Absorbed Energy Versus Loading Time of Conventional Mixtures

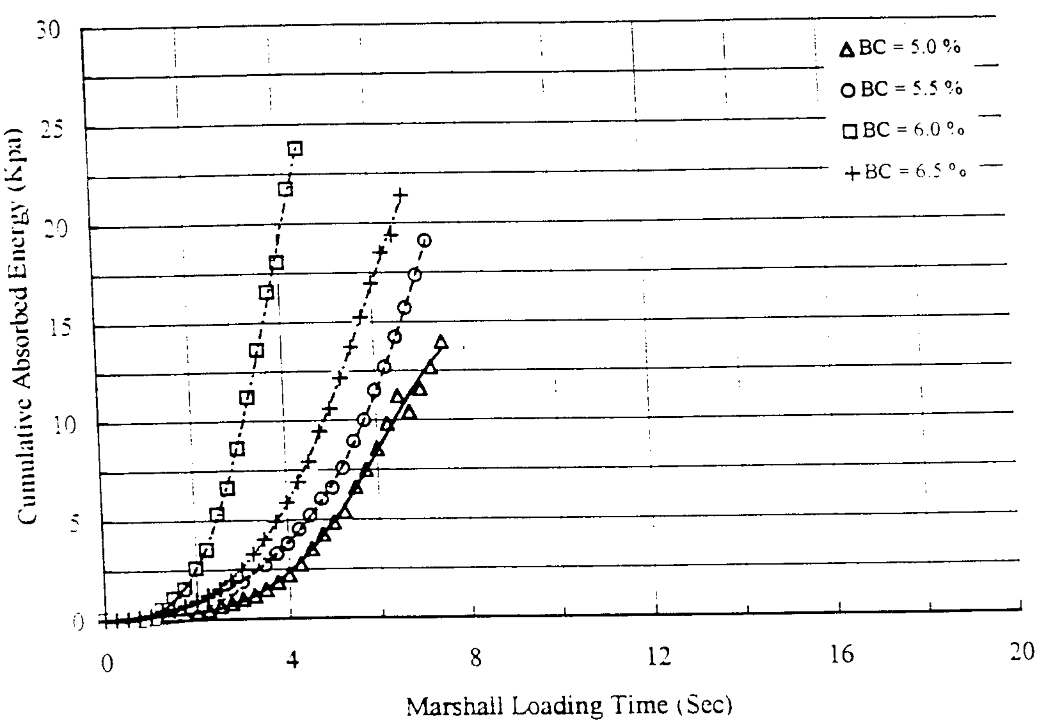


Figure 5.11 Cumulative Absorbed Energy Versus Loading Time of Asphalt-Rubber Mixtures(Wet Process)

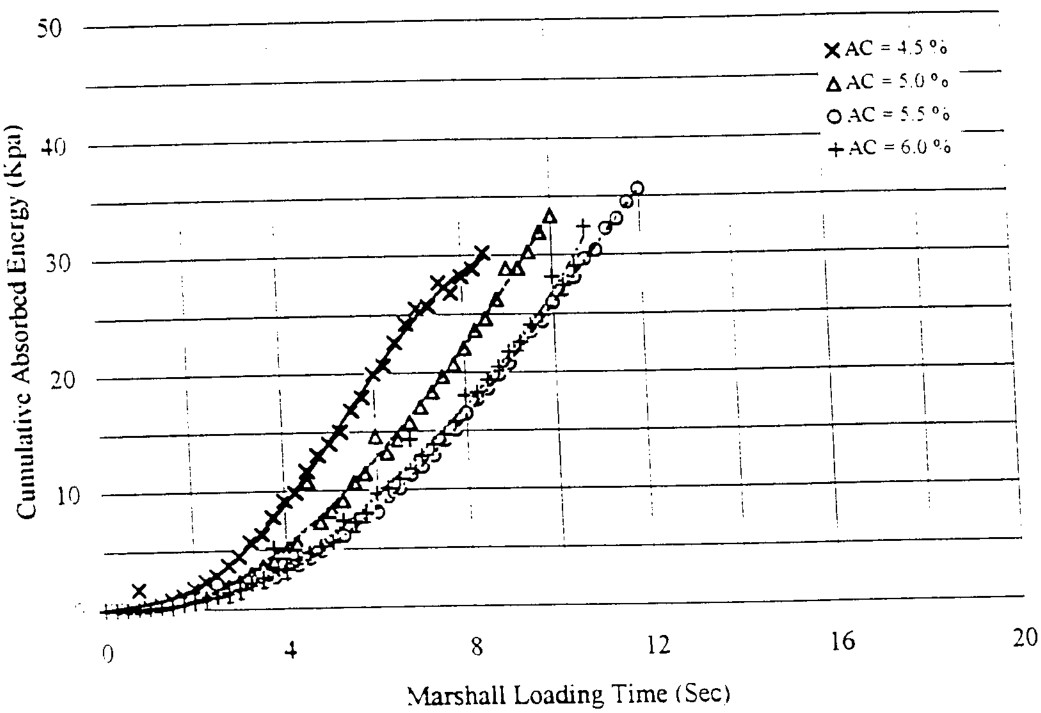


Figure 5.12 Cumulative Absorbed Energy Versus Loading Time of Rubber-Filled Mixtures (PlusRideII)

POTENTIAL ENHANCEMENT OF MARSHALL WITH MIXTURE BEHAVIOR

With the standard Marshall method of mix design, the optimum binder content is calculated from the numerical average of the asphalt contents yielding maximum stability, maximum density, and a typically chosen 4 % air void content. The results from the Marshall mix design method could be contemplated by considering toughness and stiffness evaluation of the mixtures. The binder contents corresponding to the maximum toughness and stiffness of both conventional and asphalt-rubber mixtures are provided in Table 5.4 along with the optimum from the Marshall method. As it can be seen from the Table, the Marshall optimum binder contents, 5.03%, 6.13%, and 5.00% for the conventional, and the asphalt rubber mixtures; wet process and PlusRide II, respectively, are very close to the binder content corresponding to the maximum toughness, (5.08%, 6.13%, and 5.10%, respectively). However, the binder contents corresponding to the maximum stiffness were found to be 0.23%, 0.40%, and 0.18% lower than the Marshall optimum binder content for these mixtures. Thus, if mixture behavior is to be included into the design process criteria, the optimum binder content of the traditional Marshall method should be adjusted so as to obtain a mixture providing: high stability resisting to the desired level of load applications; high density for improved durability; the desired air void content for accounting densification from the expected traffic; high toughness and absorbed energy potential for improved resistance to high loads and load duration, and resisting to high deformations before failure; and appropriate level of stiffness for improved resistance to cracking and rutting.

Table 5.4 Binder Content at Optimum Marshall and Maximum Stiffness and Toughness

Technique	Binder Content (% by Total Mixture Weight)		
	Conventional	Wet Process	PlusRide II
Optimum with Marshall Method	5.03	6.13	5.00
Maximum Stiffness	4.80	5.73	4.82
Maximum Toughness	5.08	6.12	5.10

Based on the results of this study, Table 5.5 provides the physical characteristics corresponding at the binder contents of maximum stiffness, maximum toughness, and Marshall optimum for both conventional and asphalt-rubber mixtures. The air void of the conventional mixture corresponding to the maximum stiffness is 3.85% and still within the specification limits, 3.0-5.0%. In the case of asphalt-rubber mixture the voids content passed the upper specification limits, 5.84%. However, the modified Marshall optimum binder content (potentially the average of the values that yield maximum stability, maximum stiffness and/or toughness, maximum density, and 4 percent air void) may be used in identifying the appropriate binder content. This approach needs to be further developed/validated with additional mixtures and field data.

Table 5.5 Asphalt Mixtures Characteristics at Binder Contents of Marshall Optimum, Maximum Stiffness and Toughness

Mix	BC (%) for	Stability (N)	Density (Kg/m^3)	AV (%)	Toughness (Kpa)	Stiffness $S_{mix}(t) = \sigma/\varepsilon$ (Kpa)	Stiffness $S_{mix} = S/2F_{(0.5S)}$ (N/mm)
Conventional Mixture	5.03 (Marshall Optimum)	8573	2373	3.29	20.67	27010	1985
	4.08 (Maximum Stiffness)	8628	2366	3.85	20.27	27818	2042
	5.08 (Maximum Toughness)	8538	2375	3.16	20.67	26692	1960
Asphalt-Rubber Mixture (Wet Process)	6.13 (Marshall Optimum)	6902	2305	4.56	22.22	16407	1134
	5.73 (Maximum Stiffness)	6980	2290	5.84	20.23	17610	1256
	6.12 (Maximum Toughness)	6902	2305	4.56	22.22	16463	1139
Rubber-Filled Mixture (PlusRide II)	5.00 (Marshall Optimum)	6850	2280	4.00	44.20	10000	1216
	4.82 (Maximum Stiffness)	6710	2285	4.10	37.50	10840	1285
	5.10 (Maximum Toughness)	6770	2270	3.90	42.80	9810	1110

CHAPTER 6. ASPHALT RUBBER MIXTURES EVALUATION WITH INDIRECT TENSILE TEST

INTRODUCTION

The Marshall method of mixture design, representing the most common method used by DOTs, based on empirical criteria, measures the stability and the flow of the asphalt mixture during testing. As discussed in chapter 2, the Marshall test outputs (i.e., stability and flow), have poor correlation with material and testing variables. Thus, mixture performance may not be appropriately evaluated using the Marshall test results. In addition, the validity of Marshall mix design is being questioned since mixtures designed with this method did not always perform satisfactorily. The static and repeated load indirect tensile tests have been extensively used in the characterization of asphalt mixtures, and the evaluation of mixture performance for creep, fatigue, and low temperature cracking. Thus, the indirect tensile test may be used in the development of a mixture design method. In such a case it is desirable to select a suitable evaluation test that from one side will minimize testing and on the other hand will be able to appropriately characterize the structural properties of conventional and asphalt rubber mixtures. As it was presented in Chapter 2, the indirect tensile test is being successful in evaluating the structural properties of asphalt mixture, and overcome the deficiencies of the Marshall mix design method. The repeated-load indirect tensile test has been successfully used and recommended for several of the past studies in asphalt mixture characterization due to the several advantages presented in Chapter 2 and its high repeatability.

STATIC INDIRECT TENSILE TEST RESULTS

An MTS, closed loop servo-hydraulic system was used for conducting the static indirect tensile test. An Indirect tensile test frame was locally fabricated and the results were acquired with a data acquisition system with sampling rate of 25 Hz. The loading rate used was 50.8 mm per minute and the tests was conducted at 25°C (77°F). The Indirect tensile strength, Poisson's ratio, and static resilient modulus were calculated based on the following equations:

$$\mu = (3.58791 - 0.269895\,D_R) / (0.062745 + D_R)$$

$$M_R = P\,(3.58791 - 0.062745\,\mu) / (L\,D_V)$$

$$INTS = 0.156241\,P / L$$

where μ is the Poisson's ratio; D_R is the deformation ratio (D_V/D_H); D_V is the vertical total deformation along the vertical diameter of the specimen, in; D_H is the horizontal total deformation along the horizontal diameter of the specimen, in; M_R is the resilient modulus, psi; L is the sample thickness, in; P is the magnitude of the applied load, Ib; and INTS is the indirect tensile strength, psi.

The static indirect tensile test results for the asphalt rubber mixtures (wet process and PlusRide II) are shown in Table 6.1. The relationships between the binder content and indirect tensile strength and resilient modulus are shown in Figures 6.1 and 6.2. As expected, the indirect tensile strength and resilient modulus increase with binder content up to a maximum, after which a decrease in their values is observed, Figures 6.1 and 6.2. The maximum values for the indirect tensile strength, equal to 604.00 Kpa and 510.00 Kpa for the wet process and PlusRide II asphalt-rubber mixtures respectively, were obtained at about 6.09 % and 5.03% binder contents. The static resilient modulus results, based on the static ITT testing, provided high values at 5.6 % and 5.1% binder content for the wet process and PlusRide II, respectively. The Maximum static resilient modulus values are 1.70E+5 Kpa and 9.50E+4 Kpa, respectively. The static resilient modulus of PlusRide II is lower than that of the wet process. The larger size of the crumb rubber used with PlusRide II make the asphalt mixture more flexible creating higher deformation under loads.

Table 6.1 Static Indirect Tensile Test Results

Binder Content (%)	Asphalt-Rubber Mixture (Wet Process)			Asphalt-Rubber Mixture (PlusRideII)		
	Poisson's Ratio	INTS (Kpa)	M_R (Kpa)	Poisson's Ratio	INTS (Kpa)	M_R (Kpa)
4.0				0.530	301	5.457E+4
4.5	1.029	376.0	1.801E+5	0.580	396	7.352E+4
5.0	0.783	424.5	1.514E+5	0.603	463	7.470E+4
5.5	0.868	491.0	1.580E+5	0.619	501	8.375E+4
6.0	0.816	620.0	1.690E+5	0.622	483	7.203E+4
6.5	0.831	499.1	1.146E+5			

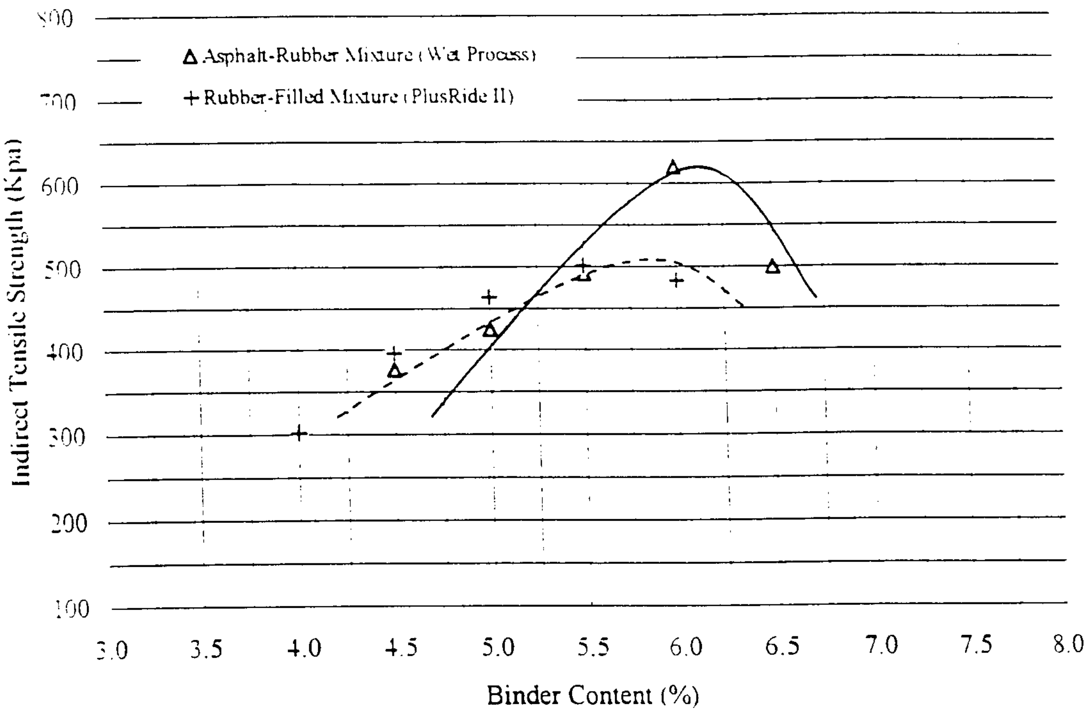


Figure 6.1 Indirect Tensile Strength Versus Binder Content

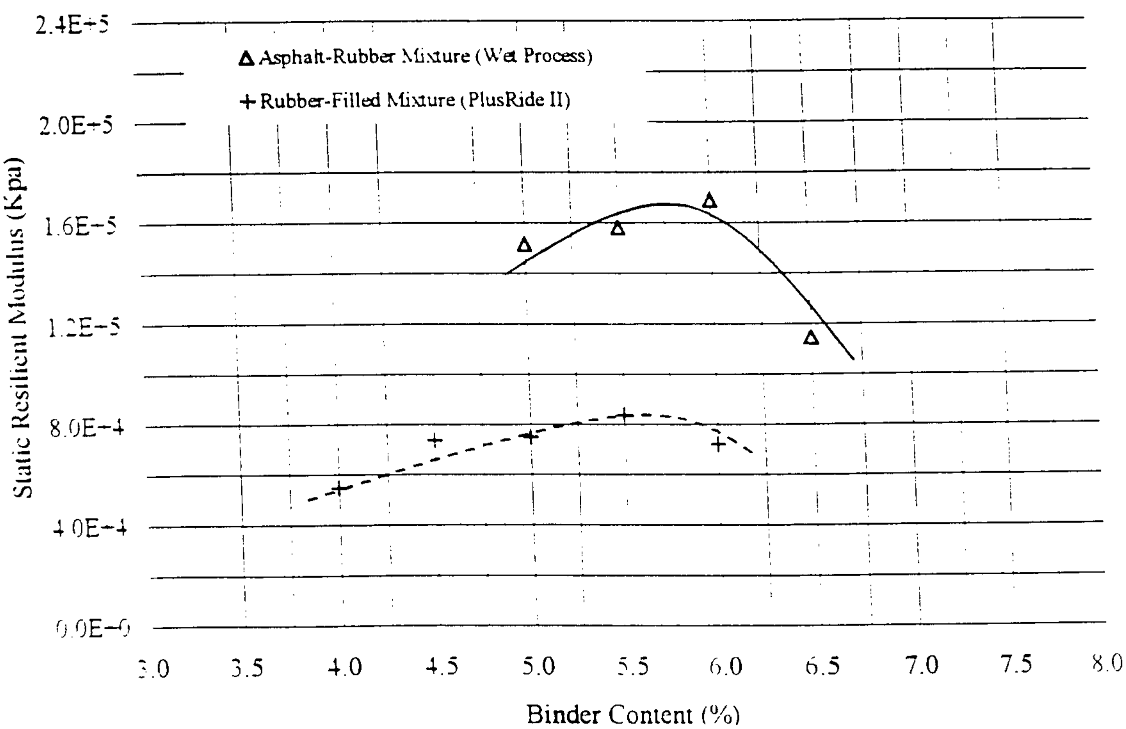


Figure 6.2 Resilient Modulus Versus Binder Content

REPEATED-LOAD INDIRECT TENSILE TEST RESULTS

The repeated-load indirect tensile test was conducted with a closed loop servo-hydraulic MTS, and the same static indirect tensile test setup as for the static case. A Rapid data acquisition system was used in this case for sampling at a higher rate, 500 Hz. sampling rate. A haversine shape repeated load was applied during testing, at 1 Hz frequency and with 0.1 second load duration. The two testing temperatures were 5 °C and 25 °C. According to the SHRP recommendations for resilient modulus testing a static load equal to 3% and 1.5 % of the indirect tensile strengths was used for keeping specimens in place during the 5 °C and 25 °C testing, respectively. Before any readings were acquired for resilient modulus evaluation, 100 and 75 load repetitions were applied to the specimens for the two testing temperatures, respectively. Each specimen was tested at two axis, 90 ° apart. These two axis were marked on the samples and remained the same during all testing period. After the initial load repetitions, five cycles were acquired with the data acquisition system. An example from one testing cycle acquired from the LVDTs and the load output voltages are shown in Figure 6.3. The acquired data were analyzed for evaluating the instantaneous and the total values of Poisson's ratio and resilient modulus using the following equations:

$$E_{RI} = P\,(\mu_{RI} + 0.27) / t\Delta H_I$$

$$E_{RT} = P\,(\mu_{RT} + 0.27) / t\Delta H_T$$

$$\mu_{RI} = 3.59\,\Delta H_I / \Delta V_I - 0.27$$

$$\mu_{RT} = 3.59\,\Delta H_T / \Delta V_T - 0.27$$

where: E_{RI} is the instantaneous resilient modulus, Mpa; E_{RT} is the total resilient modulus, Mpa; μ_{RI} is the instantaneous resilient Poisson's ratio; μ_{RT} is the total resilient Poisson's ratio; t represents the specimen thickness, mm; P is the repeated load, N; ΔH_I is the instantaneous recoverable horizontal deformation, mm; ΔH_T is the total recoverable horizontal deformation, mm; ΔV_I is the instantaneous recoverable vertical deformation, mm; and ΔV_T is the total recoverable vertical deformation, mm.

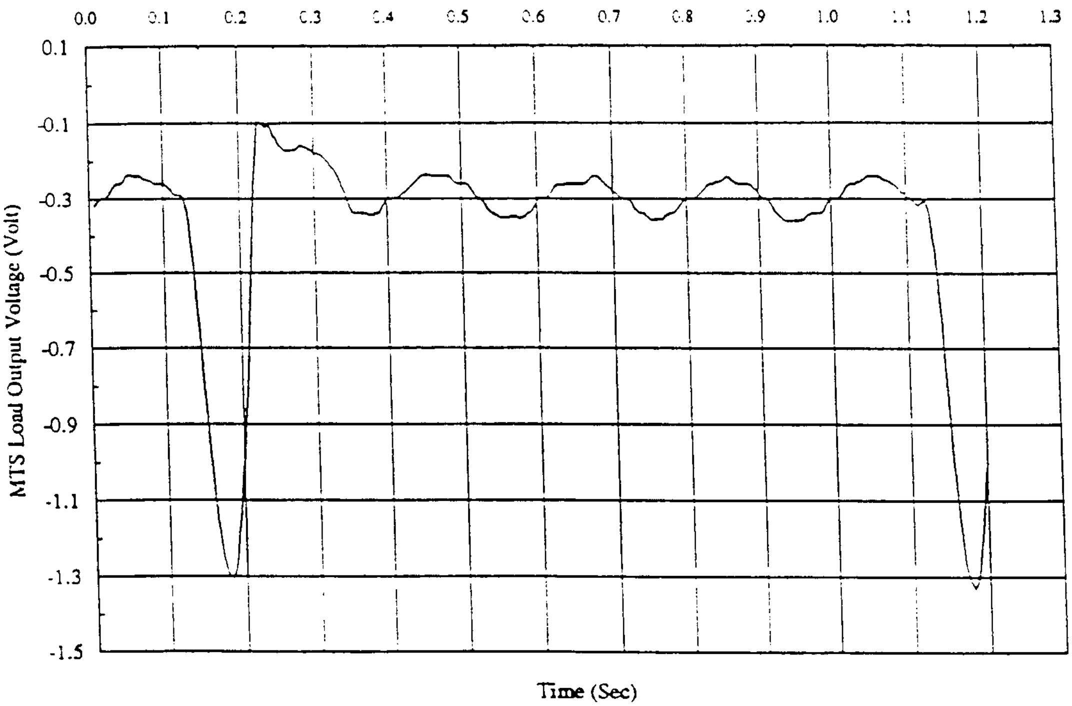
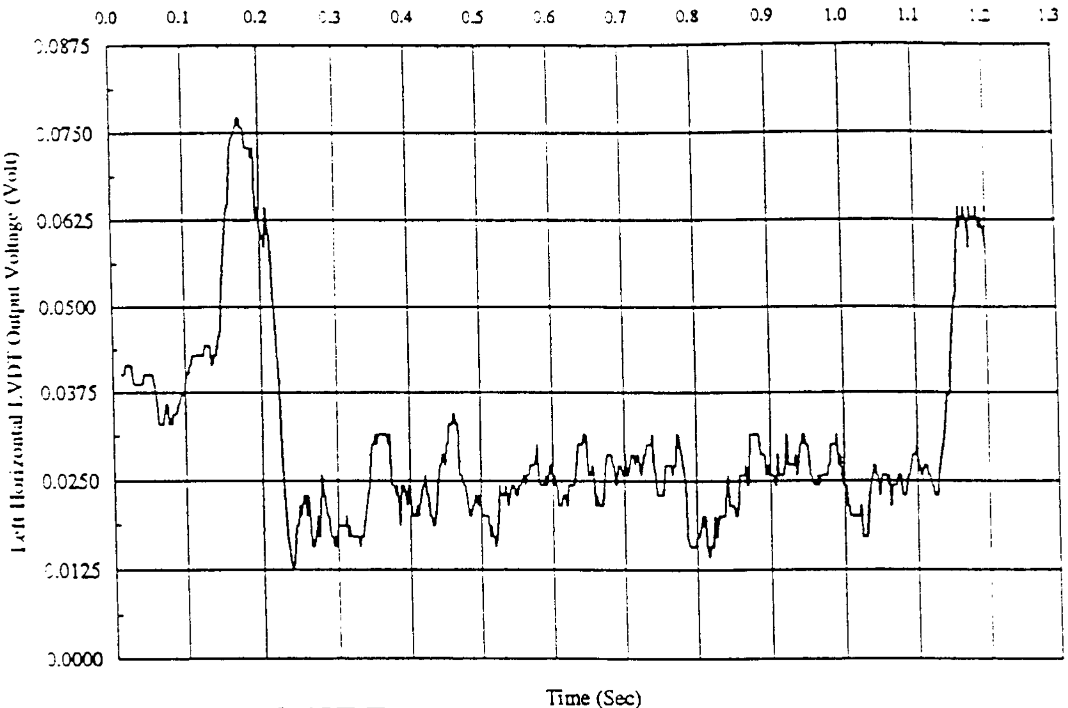
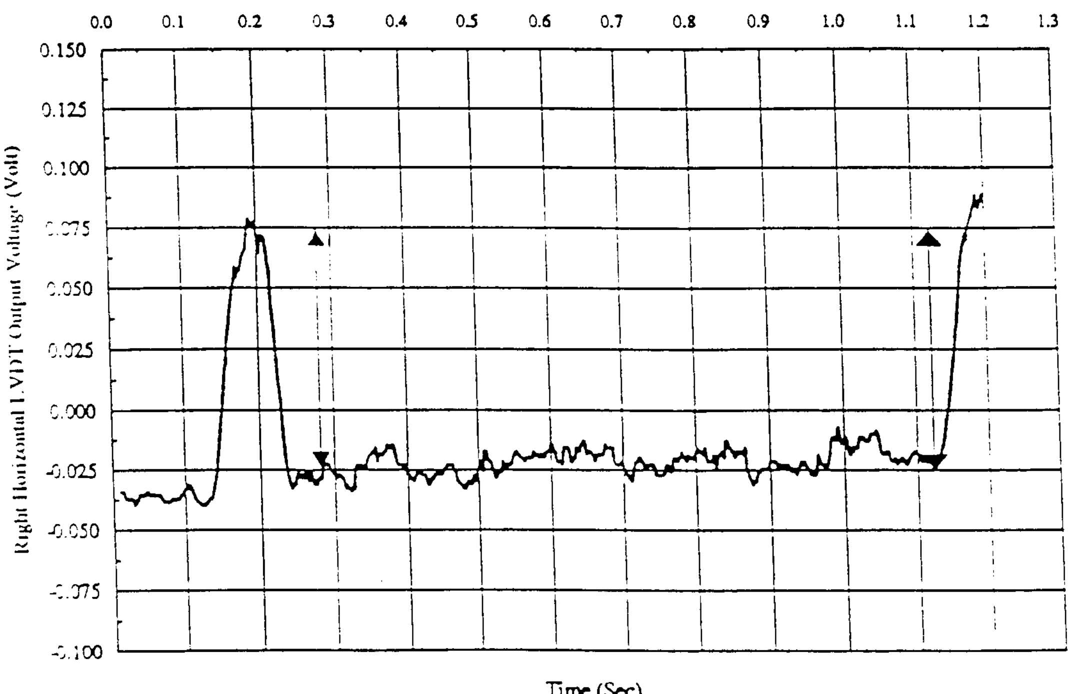


Figure 6.3 Example of Output Voltage versus Time for Mr Testing

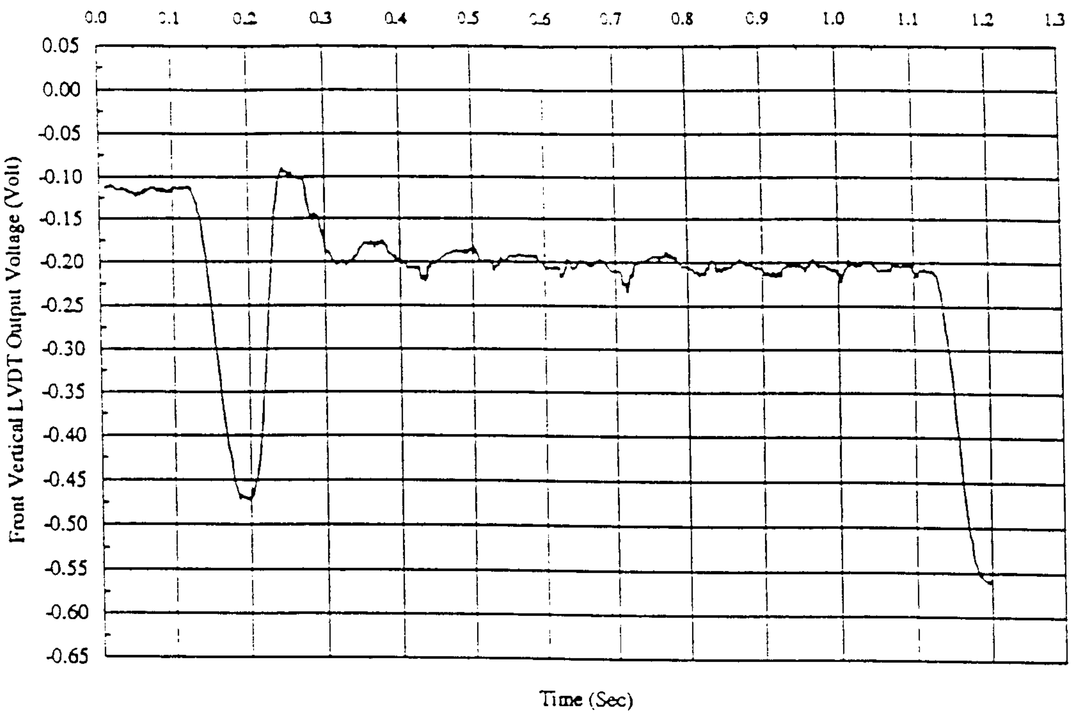


a. Left Horizontal LVDT

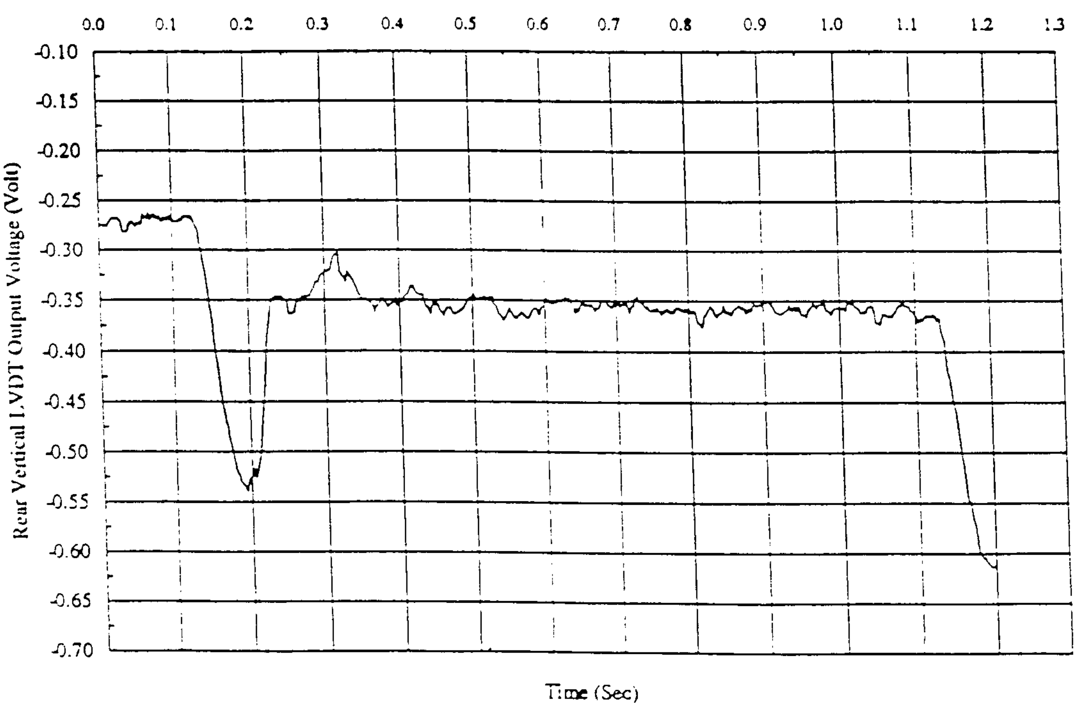


b. Right Horizontal LVDT

Figure 6.3 Example of Output Voltage versus Time for Mr Testing (Continue)



a. Front Vertical LVDT



b. Rear Vertical LVDT

Figure 6.3 Example of Output Voltage versus Time for Mr Testing (Continue)

The average values, n=3, of the instantaneous and total resilient modulus and Poisson's ratio for both axis, 0° and 90°, for asphalt-rubber mixtures (wet process) are shown in Tables 6.2 and 6.3 respectively. Also, Tables 6.4 and 6.5 present the average results for asphalt-rubber mixtures (PlusRide II). In order to examine the effect of axis on the resulted values the F-values in one-way ANOVA were computed. Past studies indicated that in certain instances the Mr-values at a 0-degree specimen position were larger than those at 90-degrees. A decrease in Mr-values was attributed in the internal damage to the specimen during testing in the initial position. Based on the results of this study, see Table 6.6 and 6.7 for wet process and PlusRide II asphalt-rubber mixtures respectively, it was found that test axis had no significant effect on resilient modulus and Poisson's ratio, at $\alpha = 0.05$. Thus no significant plane to plane variability was observed for Mr testing. The average values of 0° and 90° resilient modulus and Poisson's ratio are presented in Tables 6.8 and 6.9 respectively for the asphalt-rubber mixtures (wet process). Also, the average values for PlusRide II asphalt-rubber mixtures are shown in Tables 6.10 and 6.11.

As expected, testing temperature had a significant effect on testing results, with resilient modulus values decreasing with an increase in testing temperature. The load magnitude had a significant effect on the measured parameters at 25 °C for both wet process and PlusRide II asphalt-rubber mixtures. However, at the low level of testing temperature, 5 °C, load magnitude had no effect, see Tables 6.12 and 6.13. Thus, at 5 °C testing temperature any change with the applied load magnitude inside the elastic range, 10 to 30 % of the indirect tensile strength, as recommended by SHRP and previous studies, will not affect the mean of the measured parameters.

The effect of Poisson's ratio on Mr values has been reported in past studies. In certain instances it was concluded that when using an assumed value of 0.35 for Poisson's ratio the resilient modulus was about 1.5 to 2 times higher when the Poisson's ratio was evaluated from monitoring the horizontal and vertical deformations. In this study

Table 6.2 Average Values of Resilient Modulus of Asphalt-Rubber Mixtures (Wet Process)

a. Axis 0°

Binder Content (% of Total Mix Weight)	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Resilient Modulus (Kpa)							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
5.0	3.70E+6	3.05E+6	3.55E+6	2.98E+6	3.46E+6	2.81E+6	2.94E+6	2.38E+6
5.5	4.26E+6	3.36E+6	3.83E+6	3.20E+6	4.19E+6	3.32E+6	3.24E+6	2.38E+6
6.0	3.62E+6	2.97E+6	3.99E+6	3.26E+6	3.27E+6	2.69E+6	2.84E+6	2.29E+6
6.5	3.71E+6	3.02E+6	3.58E+6	3.00E+6	3.29E+6	2.64E+6	2.97E+6	2.45E+6

b. Axis 90°

Binder Content (% of Total Mix Weight)	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Resilient Modulus (Kpa)							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
5.0	3.80E+6	3.03E+6	3.00E+6	2.36E+6	3.50E+6	2.82E+6	2.84E+6	2.08E+6
5.5	4.18E+6	3.33E+6	3.96E+6	3.59E+6	4.88E+6	3.58E+6	3.44E+6	2.50E+6
6.0	3.62E+6	3.06E+6	4.17E+6	3.28E+6	3.14E+6	2.72E+6	2.84E+6	2.28E+6
6.5	3.65E+6	2.98E+6	3.35E+6	3.05E+6	3.20E+6	2.60E+6	2.83E+6	2.13E+6

Table 6.3 Average Values of Poisson's Ratio for Asphalt-Rubber Mixtures (Wet Process)

a. Axis 0°

Binder Content (% of Total Mix Weight)	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Poisson's Ratios							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
5.0	0.097	0.156	0.155	0.190	0.518	0.532	0.592	0.606
5.5	0.337	0.340	0.213	0.236	0.656	0.601	0.885	0.736
6.0	0.260	0.267	0.343	0.327	0.729	0.696	0.608	0.561
6.5	0.092	0.127	0.210	0.231	0.565	0.583	0.719	0.753

b. Axis 90°

Binder Content (% of Total Mix Weight)	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Poisson's Ratios							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
5.0	0.213	0.150	0.289	0.370	0.607	0.618	0.701	0.601
5.5	0.354	0.351	0.155	0.233	0.750	0.619	1.002	0.882
6.0	0.258	0.285	0.296	0.304	0.669	0.683	0.451	0.545
6.5	0.199	0.187	0.067	0.206	0.512	0.540	0.711	0.668

Table 6.4 Average Values of Resilient Modulus for Rubber-Filled Mixtures (PlusRide II)

a. Axis 0°

Binder Content (% of Total Mix Weight)	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Resilient Modulus (Kpa)							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
4.0	3.02E+6	2.40E+6	2.92E+6	2.28E+6	2.19E+6	1.45E+6	2.10E+6	1.51E+6
4.5	3.14E+6	2.52E+6	3.06E+6	2.42E+6	2.36E+6	1.66E+6	2.14E+6	1.49E+6
5.0	3.01E+6	2.48E+6	2.87E+6	2.27E+6	2.20E+6	1.60E+6	2.22E+6	1.61E+6
5.5	2.64E+6	2.28E+6	2.67E+6	2.07E+6	2.03E+6	1.62E+6	2.02E+6	1.47E+6
6.0	2.23E+6	2.08E+6	2.64E+6	1.99E+6	2.02E+6	1.45E+6	2.11E+6	1.44E+6

b. Axis 90°

Binder Content (% of Total Mix Weight)	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Resilient Modulus (Kpa)							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
4.0	2.97E+6	2.31E+6	2.88E+6	2.22E+6	2.23E+6	1.55E+6	2.18E+6	1.58E+6
4.5	3.03E+6	2.39E+6	3.12E+6	2.38E+6	2.32E+6	1.67E+6	2.16E+6	1.56E+6
5.0	2.80E+6	2.29E+6	2.95E+6	2.28E+6	2.14E+6	1.59E+6	2.22E+6	1.59E+6
5.5	2.63E+6	2.24E+6	2.68E+6	2.13E+6	2.01E+6	1.52E+6	2.26E+6	1.46E+6
6.0	2.29E+6	2.12E+6	2.52E+6	2.06E+6	2.11E+6	1.50E+6	1.98E+6	1.44E+6

Table 6.5 Average Values of Poisson's Ratio for Rubber-Filled Mixtures (PlusRide II)

a. Axis 0°

Binder Content (% of Total Mix Weight)	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Poisson's Ratios							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
4.0	0.398	0.389	0.751	0.648	0.670	0.474	0.852	0.853
4.5	0.555	0.480	0.878	0.796	0.691	0.543	0.702	0.840
5.0	0.694	0.653	0.900	0.726	0.549	0.651	0.624	0.857
5.5	0.645	0.590	0.790	0.841	0.551	0.688	0.643	0.729
6.0	0.636	0.605	0.929	0.689	0.634	0.616	0.600	0.797

b. Axis 90°

Binder Content (% of Total Mix Weight)	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Poisson's Ratios							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
4.0	0.374	0.376	0.627	0.569	0.843	0.473	0.787	0.853
4.5	0.584	0.542	0.824	0.787	0.779	0.586	0.670	0.969
5.0	0.545	0.532	0.828	0.678	0.616	0.582	0.710	0.963
5.5	0.585	0.552	0.727	0.652	0.496	0.597	0.722	0.700
6.0	0.663	0.656	0.804	0.793	0.529	0.620	0.694	0.751

Table 6.6 F-Values of the Analysis of Variance for Testing Axis Effects for Asphalt-Rubber Mixture, Wet Process. (0° and 90° Axis)

Mixture Property	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Resilient Modulus (Kpa)							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
Resilient Modulus	0.00	0.00	1.12	1.04	0.15	0.16	0.01	2.61
Poisson's Ratio	1.50	0.16	0.33	0.45	0.06	0.05	0.03	0.03
Critical F-Value	4.3							

Table 6.7 F-Values of the Analysis of Variance for Testing Axis Effects for Rubber-Filled Mixture, PlusRide II, (0° and 90° Axis)

Mixture Property	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Resilient Modulus (Kpa)							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
Resilient Modulus	0.18	1.99	0.02	0.02	0.28	0.00	1.52	0.24
Poisson's Ratio	0.76	0.13	1.36	1.10	0.26	1.17	0.28	0.31
Critical F-Value	4.3							

Table 6.8 Average Mr Values of Both Testing Axis for Asphalt-Rubber Mixture, Wet Process. (0° and 90° Axis)

Binder Content (% of Total Mix Weight)	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Resilient Modulus (Kpa)							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
5.0	3.75E+6	3.04E+6	3.27E+6	2.67E+6	3.48E+6	2.82E+6	2.89E+6	2.23E+6
5.5	4.22E+6	3.35E+6	3.89E+6	3.39E+6	4.53E+6	3.45E+6	3.34E+6	2.44E+6
6.0	3.62E+6	3.01E+6	4.08E+6	3.27E+6	3.21E+6	2.70E+6	2.84E+6	2.28E+6
6.5	3.68E+6	3.00E+6	3.46E+6	3.03E+6	3.24E+6	2.62E+6	2.90E+6	2.29E+6

Table 6.9 Average Poisson's Ratio Values of Both Testing Axis for Asphalt-Rubber Mixture, Wet Process, (0° and 90° Axis)

Binder Content (% of Total Mix Weight)	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Poisson's Ratios							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
5.0	0.155	0.153	0.222	0.280	0.563	0.575	0.646	0.604
5.5	0.346	0.346	0.184	0.234	0.703	0.610	0.944	0.809
6.0	0.259	0.276	0.319	0.316	0.699	0.689	0.529	0.553
6.5	0.146	0.157	0.139	0.219	0.539	0.561	0.715	0.711

Table 6.10 Average Mr Values of Both Testing Axis for Rubber-Filled Mixture, PlusRide II. (0 ° and 90 ° Axis)

Binder Content (% of Total Mix Weight)	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Resilient Modulus (Kpa)							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
4.0	2.99E+6	2.36E+6	2.90E+6	2.25E+6	2.21E+6	1.50E+6	2.14E+6	1.54E+6
4.5	3.09E+6	2.45E+6	3.09E+6	2.40E+6	2.34E+6	1.67E+6	2.15E+6	1.52E+6
5.0	2.91E+6	2.38E+6	2.91E+6	2.27E+6	2.17E+6	1.60E+6	2.22E+6	1.60E+6
5.5	2.64E+6	2.26E+6	2.67E+6	2.10E+6	2.02E+6	1.57E+6	2.14E+6	1.47E+6
6.0	2.26E+6	2.10E+6	2.58E+6	2.03E+6	2.06E+6	1.48E+6	2.05E+6	1.44E+6

Table 6.11 Average Poisson's Ratio Values of Both Testing Axis for Rubber-Filled Mixture, PlusRide II, (0 ° and 90 ° Axis)

Binder Content (% of Total Mix Weight)	Test Temperature (°C)							
	5				25			
	Applied Load (% of Indirect Tensile Strength)							
	10		30		10		30	
	Poisson's Ratios							
	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total	Instantaneous	Total
4.0	0.386	0.383	0.689	0.608	0.757	0.473	0.819	0.853
4.5	0.569	0.511	0.851	0.791	0.735	0.565	0.686	0.905
5.0	0.619	0.592	0.864	0.702	0.583	0.617	0.667	0.910
5.5	0.615	0.571	0.758	0.747	0.524	0.643	0.682	0.715
6.0	0.650	0.631	0.866	0.741	0.582	0.618	0.647	0.774

Table 6.12 F-Values of the Analysis of Variance for Load Magnitude Effects, 10 and 30% of INTS, for Asphalt-Rubber Mixtures (Wet Process)

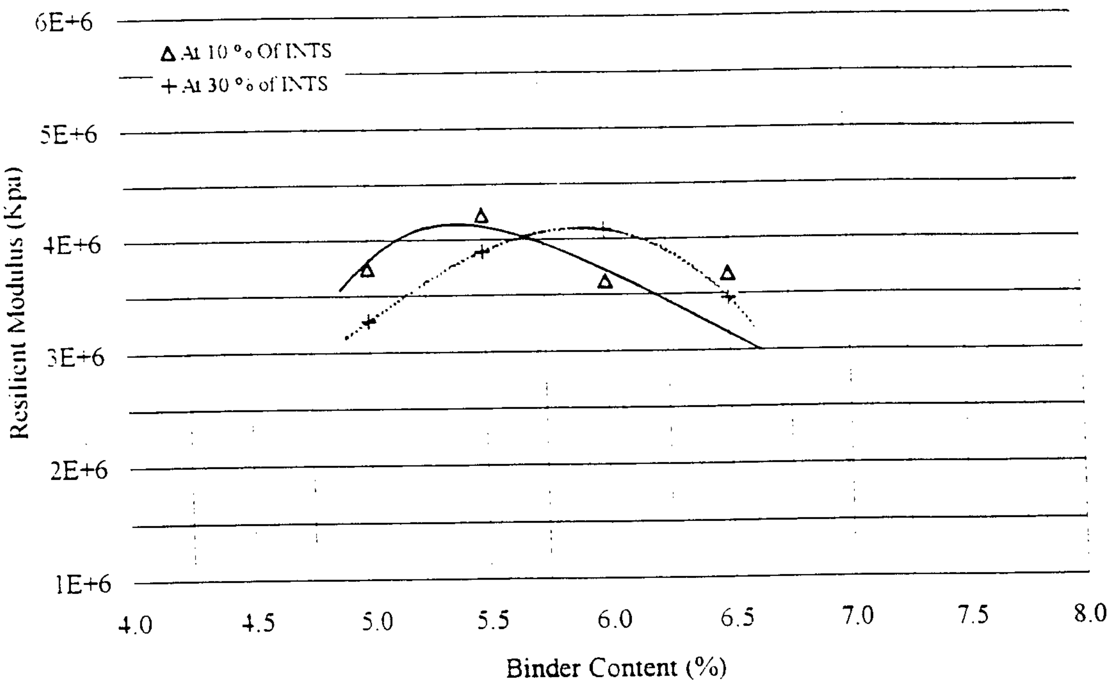
Mixture Property	Test Temperature (°C)			
	5		25	
	Computed F-Value			
	Instantaneous	Total	Instantaneous	Total
Resilient Modulus	0.91	1.13	7.77	22.67
Poisson's Ratio	0.06	0.51	1.35	1.43
Critical F-Value	4.30			

Table 6.13 F-Values of the Analysis of Variance for Load Magnitude Effects, 10 and 30% of INTS, for Rubber-Filled Mixtures (PlusRide II)

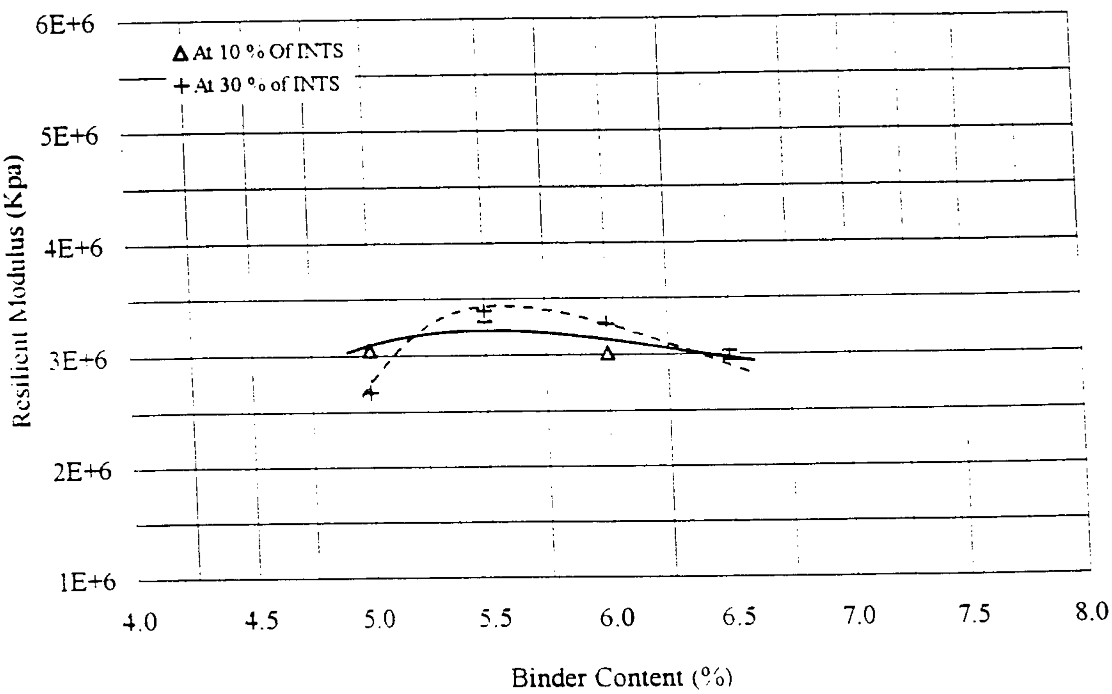
Mixture Property	Test Temperature (°C)			
	5		25	
	Computed F-Value			
	Instantaneous	Total	Instantaneous	Total
Resilient Modulus	0.30	3.23	18.47	19.84
Poisson's Ratio	0.00	2.46	1.63	0.00
Critical F-Value	4.30			

the Poisson's ratio was calculated and used in evaluating Mr values. As it can be seen from Tables 6.9 and 6.11, for wet process and PlusRide II respectively, testing temperature had a significant effect on Poisson's ratio. Higher Poisson's ratio for asphalt rubber mixtures evaluated at higher temperature is expected since the mixture becomes softer, producing higher lateral deformation. Considering the different binder contents, the average values of calculated Poisson's ratios are 0.25, and 0.65 at 5 °C and 25 °C testing temperatures, respectively, for the wet process asphalt-rubber mixtures. Also, the average values for PlusRide II are 0.66 and 0.69 at 5 °C and 25 °C, respectively. The Poisson's values at 5 °C for the asphalt rubber and the conventional mixtures are at about the same level, equal to 0.2. At 25 °C the asphalt rubber mixtures have significantly higher values, ranging from 0.5 to 0.7 and from 0.3 to 0.9 for the wet process and the PlusRide II respectively, while the conventional one had a value of about 0.35. The asphalt rubber mixtures at high temperatures have higher flexibility and deformation and thus a higher value of Poisson's ration is expected.

The instantaneous and total resilient modulus versus binder content at 5 °C and 25 °C testing temperatures are shown in Figures 6.4 and 6.5 for the wet process mixtures and in Figures 6.6 and 6.7 for the PlusRide II. The maximum values of resilient modulus are within the range of 5.5% to 6.0% and 4.2% to 5.0% binder content for the wet process and the PlusRide II. It is evident from these Figures, that at 5 °C there is no big difference between the maximum resilient modulus values obtained from the different load level. At the opposite, significantly higher maximum resilient modulus values are obtained for the load magnitude of 10 % of the indirect tensile strength for the 25 °C testing.

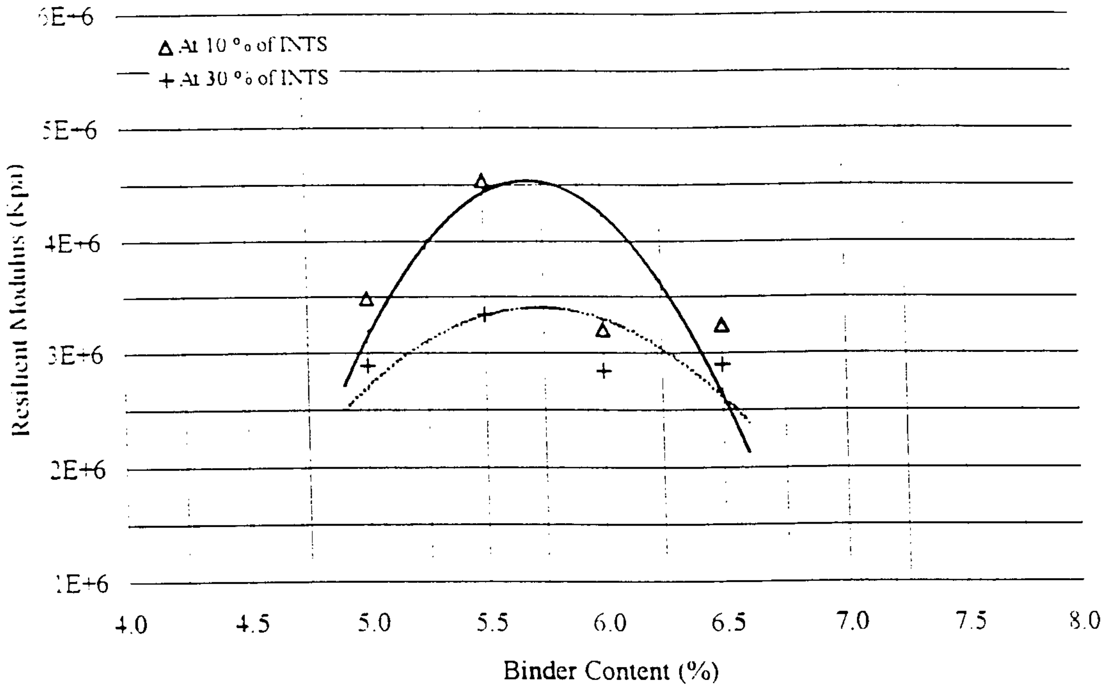


a. Instantaneou Resilient Modulus

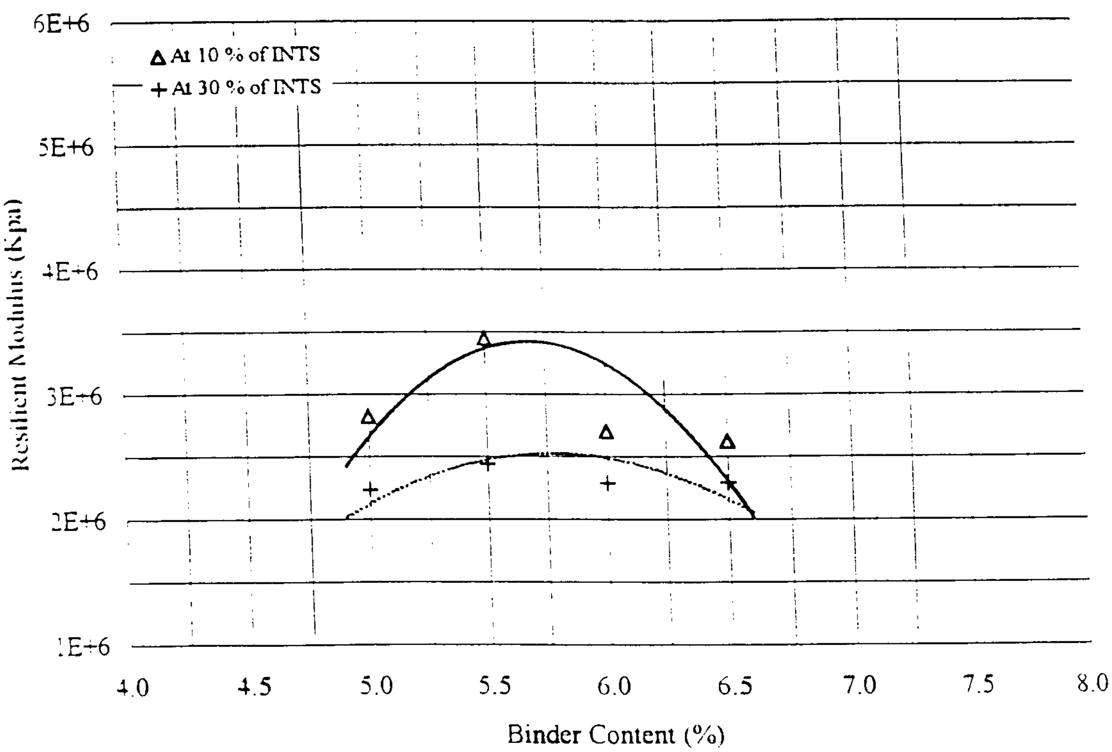


b. Total Resilient Modulus

Figure 6.4 Resilient Modulus versus Binder Content at $5\ ^{o}C$ for Asphalt-Rubber Mixture (Wet Process)

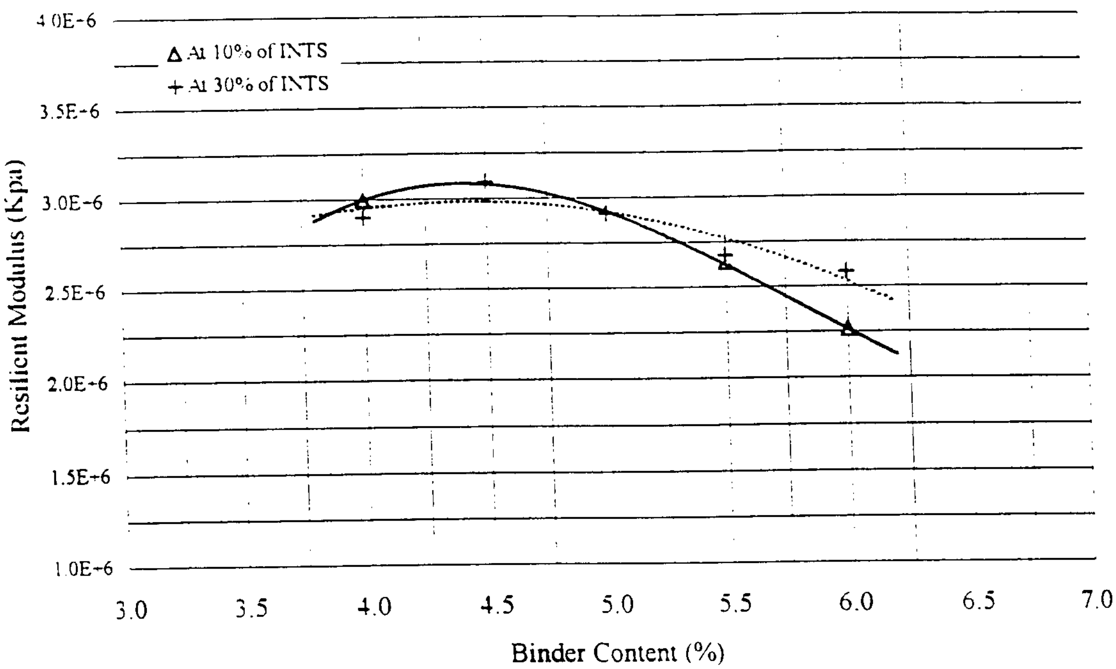


a. Instantaneou Resilient Modulus

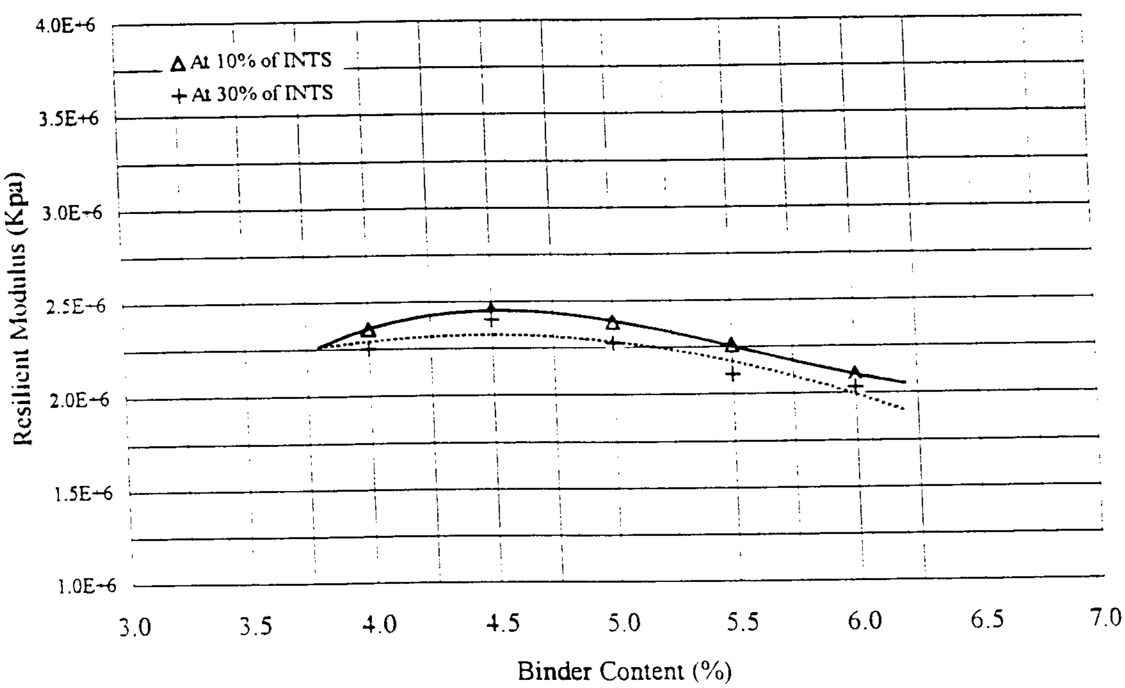


b. Total Resilient Modulus

Figure 6.5 Resilient Modulus versus Binder Content at 25 °C Asphalt-Rubber Mixture (Wet Process)

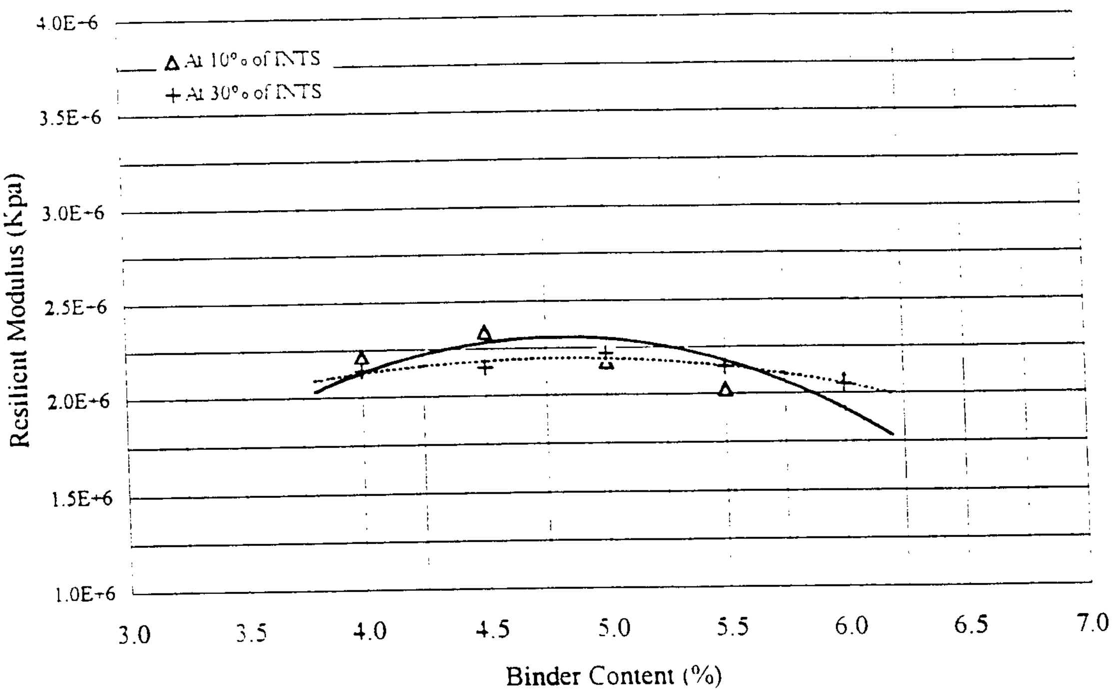


a. Instantaneous Resilient Modulus

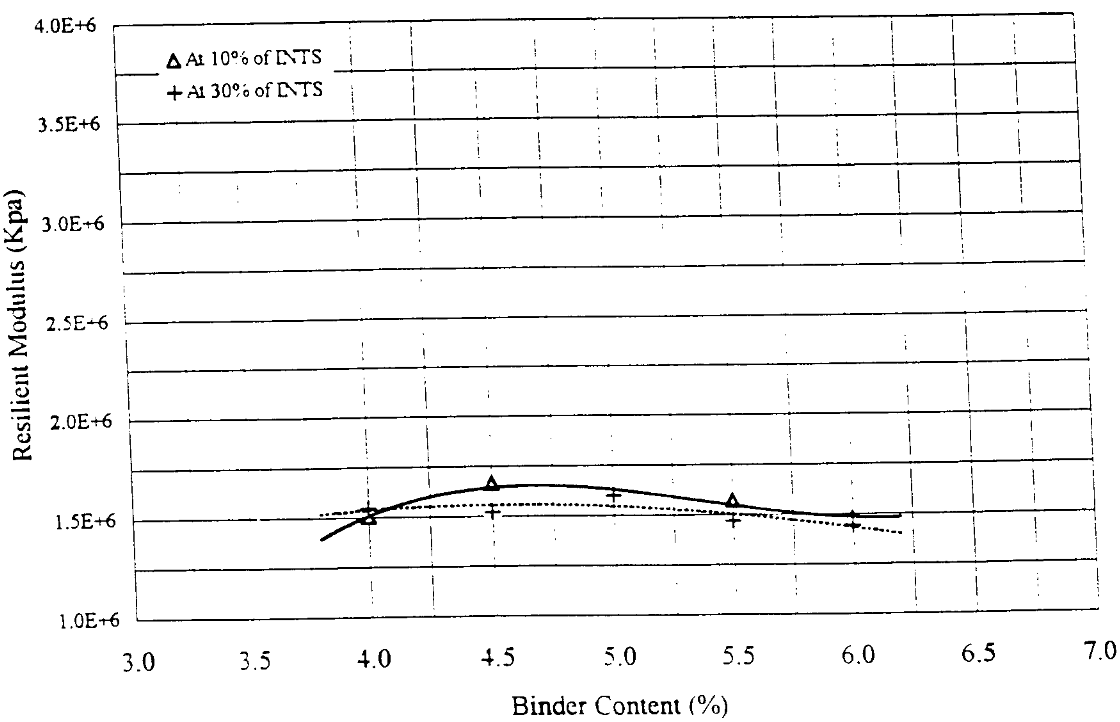


b. Total Resilient Modulus

Figure 6.6 Resilient Modulus versus Binder Content at 5 °C for Rubber-Filled Mixture (PlusRideII)



a. Instantaneou Resilient Modulus



b. Total Resilient Modulus

Figure 6.7 Resilient Modulus versus Binder Content at 25 °C for Rubber-Filled Mixture (PlusRide II)

CHAPTER 7. FATIGUE AND CREEP CHARACTERIZATION OF ASPHALT-RUBBER MIXTURES

INTRODUCTION

The design of flexible pavements has rapidly evolved from empirical and semi-empirical procedures to design methods based on elastic and/or viscoelastic principles. Today, many highway agencies, including NJDOT, use such a method in one form or another to design new, reconstructed, and/or overlaid asphalt pavements. Performance based design methods require a through knowledge of the basic structural properties and mixture performance (resilient modulus, fatigue life, and plastic deformation). Existing asphalt mix design procedures (Marshall and Hveem) are examining parameters that are not necessarily relating to the mixture performance.

Fatigue cracking and rutting are among the major distresses occurring in asphalt concrete pavements. Fatigue is defined as "the phenomenon of fracture occurring under repeated stresses with values even lower than the tensile strength of the material". In recent years considerable attention has been paid to model fatigue and the crack formation in asphalt pavements (Raad 1992).

Similarly, the problem of rutting is gaining widespread attention in many parts of the world (Kim 1988). Rutting is the formation of twin longitudinal depressions under the wheel paths from a progressive accumulation of permanent deformation in one or more locations of the pavement layers. In recent years the trend towards heavier trucks, higher tire pressure, and the substantial increase in the number of load repetitions has resulted in a significant increase in the extent and severity of rutting. Many highways in United States are experiencing extensive levels of rutting even when made with materials that, in the past, showed little propensity to rutting. Several studies have shown that most significant portion of the rutting occurs in the asphalt-bound layers. This brings into question the ability of current pavement and mixture design methods to adequately address permanent deformation under increasing demands of traffic.

Several of the past studies have successfully used and recommended the repeated-load indirect tensile diametrical fatigue test and the repeated-load unconfined triaxial creep test for evaluating asphalt mixtures' fatigue and rutting characteristics, see chapter 2. Thus in this study the repeated load unconfined triaxial test was used for evaluating the creep characteristics of asphalt rubber mixtures.

REPEATED-LOAD UNCONFINED TRIAXIAL CREEP TEST RESULTS

An MTS, closed loop servo-hydraulic system was used for creep testing. Sample ends were well greased prior to the seating of the loading platens in order to reduce friction between the sample and the platen. An environmental chamber was used to maintain the target testing temperature. Samples were instrumented so as to monitor deformations (a) over the entire height of the samples, and (b) over the middle third of the samples. The MTS vertical displacement LVDT's output voltage was used to monitor the axial deformation over the total height of the sample. The deformation over the middle third was measured with three LVDTs placed 120° apart around the sample. The axial deformation of the middle third is the average value of these three LVDT's readings. The creep test was conducted at two temperatures, 25 °C (77 °F) and 40 °C (104 °F), with two stress levels, 345 Kpa (50 psi) and 138 Kpa (20 psi) respectively. A haversine shape repeated-load was used during testing at 1 Hz frequency and 0.1 second of load duration. All samples were preconditioned, with the same testing conditions, for 60 seconds before testing. Testing started immediately at the end of this preconditioning and continued for 60 minutes. Vertical deformations were monitored with a data acquisition system with sampling rate of 5 Hz. A sample seating load of 44.5 N (10 Lb.)

was used during testing. The data were used to calculate the axial compressive strains over the instrumented length, as shown:

a. Over the total height

$$\varepsilon_{(t)} = \Delta_{(t)} / H_o$$

Where:

$\varepsilon_{(t)}$ = Axial compressive strain over the total sample height at time (t), mm/mm

$\Delta_{(t)}$ = Axial deformation over the total sample height at time (t), mm

H_o = Original height of the sample, mm

a. Over the Middle Third

$$\varepsilon_{m(t)} = \Delta_{m(t)} / H_{m\,o}$$

Where:

$\varepsilon_{m(t)}$ = Axial compressive strain over the middle third at time (t), mm/mm

$\Delta_{m(t)}$ = Axial deformation over the middle third at time (t), mm

$H_{m\,o}$ = Original height of the middle third, mm

The creep test results for both wet process and PlusRide II asphalt-rubber mixtures are presented in Table 7.1. The axial compressive strains over the total height are in most of the cases higher than those over the middle third in agreement with earlier studies (Krutz 1992).

The relationship between the axial compressive strain and the loading time for the wet process asphalt-rubber mixtures, is illustrated in Figure 7.1. At 25 °C testing temperature and with a 345 Kpa axial stress, the rate of axial compressive strain did dramatically increased after 2800 load repetitions, and for mixtures with binder contents of 5.5 and 6.13%. This might be probably due to the fact that the samples started to crack before test completion. Overall the rate of axial strain increases with binder content at this temperature. The higher binder content makes the sample softer and faster to deform and fail. The thicker coating film around the aggregate particles makes the asphalt mixture more ductile and lowers the aggregates interlock, decreasing thus mixture stiffness.

The relationship between the axial compressive strain and the loading time for the PlusRide II rubber-filled mixtures is shown in Figure 7.2. The rate of axial strain increases with binder content for both testing conditions (T= 25 °C, Stress = 345 Kpa and T = 40 °C, stress = 138 Kpa). Compared with the wet process, PlusRide II asphalt-rubber mixtures have higher axial compressive strain under loading. This is probably associated with the aggregate gradation (gap-graded for PlusRide II against dense-graded for wet process) and the presence of the coarse rubber aggregate. These coarse rubber particles make the PlusRide II more flexible. Also, the higher air voids (gap-graded gradation) create more axial compressive strain before reaching failure.

Table 7.1 Creep Test Results of Asphalt Rubber Mixtures

a. Wet Process

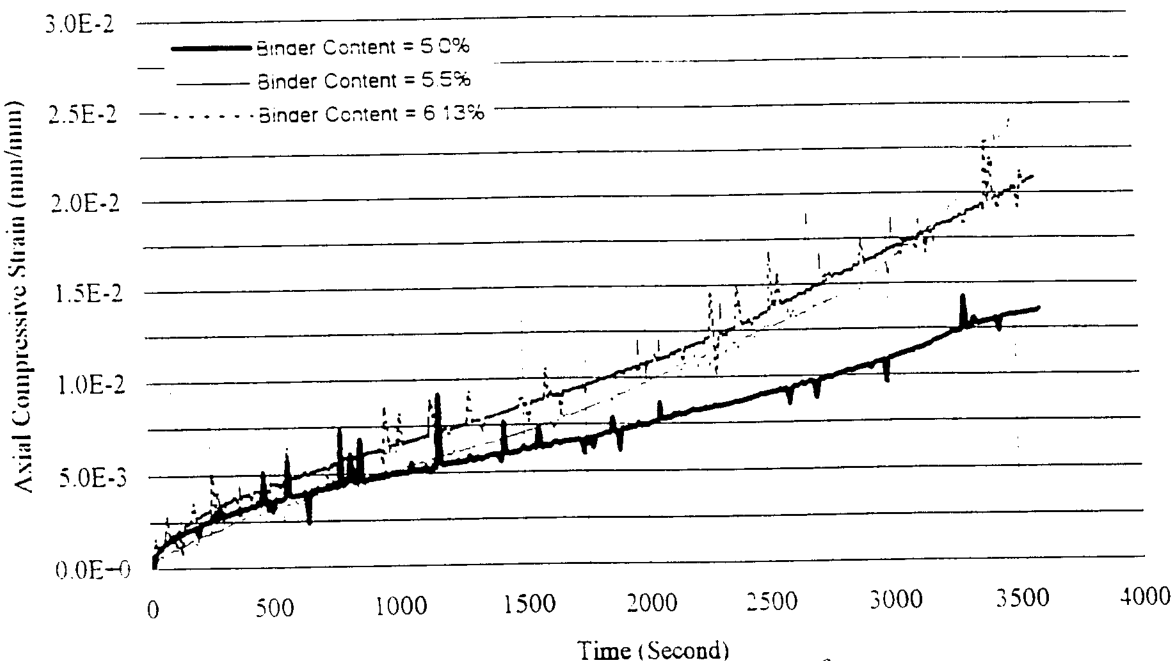
Testing Temperature (°C)	Binder Content (%)	Axial Compressive Strain (mm/mm)	
		Middle Third	Total Height
25	5.00	0.013107	0.016770
	5.50	0.024074	0.024863
	6.13	0.021543	0.022193
40	5.00	0.011657	0.012487
	5.50	0.020507	0.025213
	6.13	0.023053	0.026043

Note: three replicates n=3

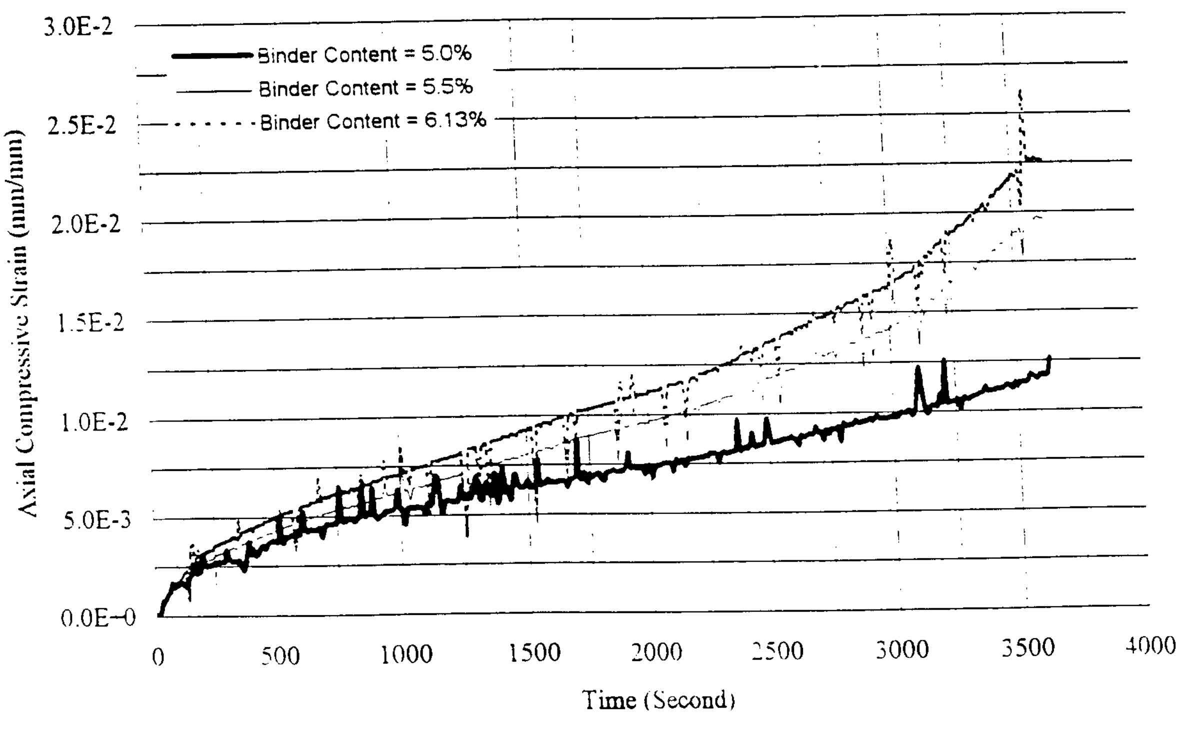
b. PlusRide II

Testing Temperature (°C)	Binder Content (%)	Axial Compressive Strain (mm/mm)	
		Middle Third	Total Height
25	4.5	0.016167	0.024000
	5.0	0.019333	0.022750
	5.5	0.021917	0.027500
40	4.5	0.008468	0.015458
	5.0	0.011046	0.015275
	5.5	0.011833	0.017000

Note: three replicates n=3

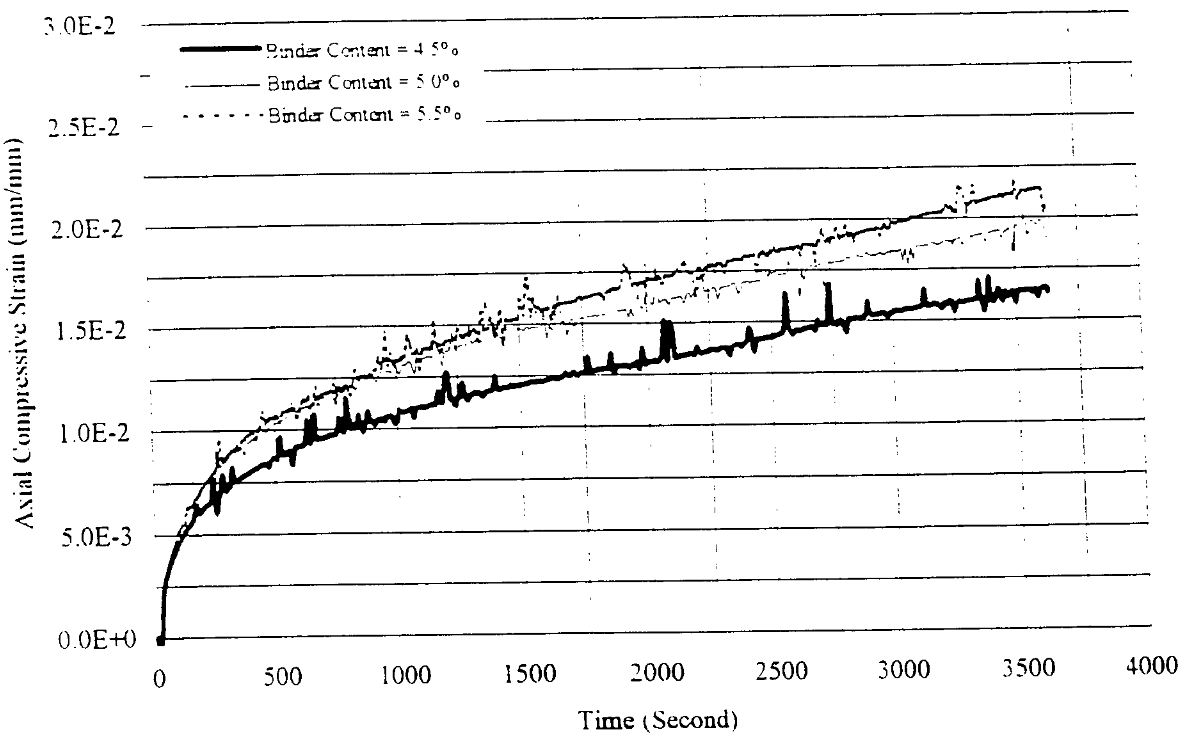


a. Test Temperature = 25 °C

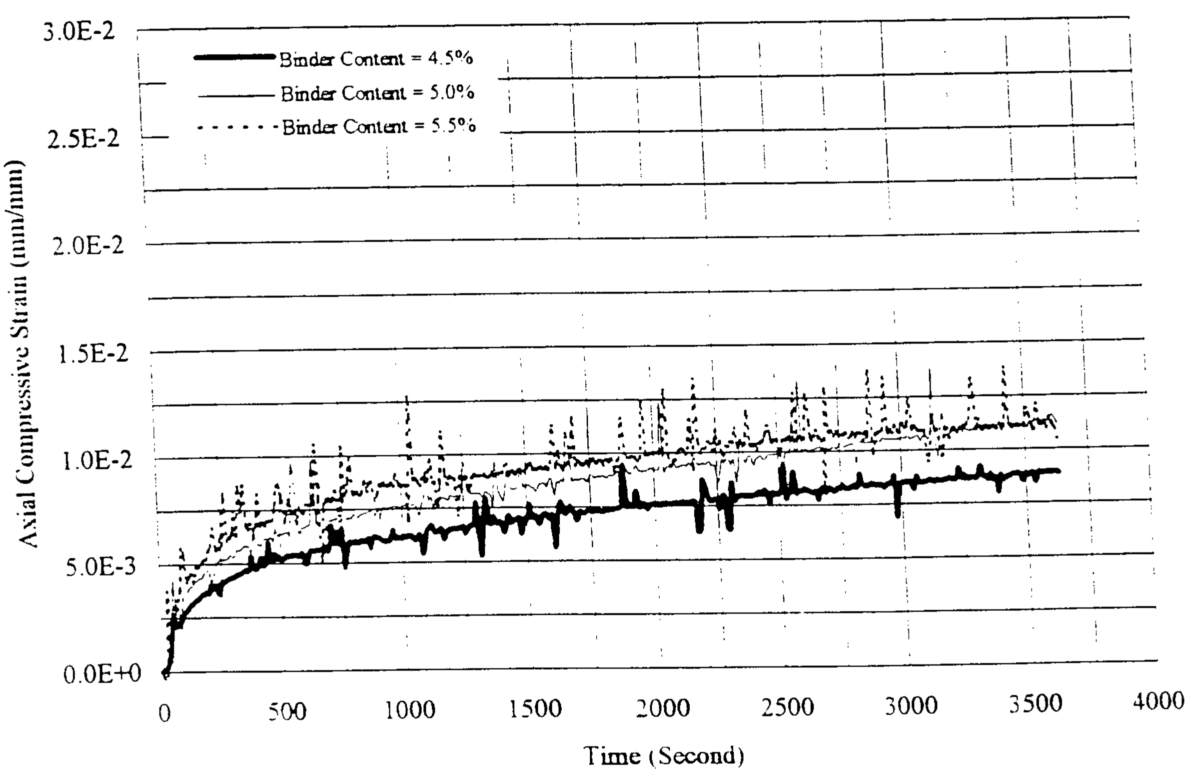


b. Test Temperature = 40 °C

Figure 7.1 Creep Test Results of Asphalt-Rubber Mixture, Wet Process



a. Test Temperature = 25 °C



b. Test Temperature = 40 oC

Figure 7.2 Creep Test Results of PlusRide II

REPEATED-LOAD INDIRECT TENSILE FATIGUE TEST RESULTS

The MTS closed-loop system with the indirect tensile test frame was used in fatigue testing. The testing temperature, 25 °C, was kept constant by using an environmental chamber. A haversine repeated-load was used equal to about 17 % of INTS (1113 N, 250 Lb., for the wet process and 890 N, 200 Lb., for the PlusRide II asphalt-Rubber Mixture) was used with 1 Hz load frequency and 0.1 second load duration. Seating load of 45 N (10 Lb.) was used during testing for both the wet process and the PlusRide II samples. Two LVDTs were used to monitor the deformation along the horizontal diameter of the tested samples. The results were acquired using a data acquisition system with 5 Hz. sampling rate. The data were used to calculate the fatigue life of the tested samples. Fatigue life is defined as the number of load repetitions that creates a total permanent horizontal deformation of 0.1 in (2.5mm).

Fatigue life was evaluated for both mixtures, the wet process and the PlusRide II, at 25 °C since the effect of binder content on fatigue is more pronounced at higher temperatures (Geotzee 1990). This can be explained by crack initiation and crack propagation concepts. It is believed that the growth in horizontal deformation at low temperature is almost negligible until a certain point. Then the crack growth becomes suddenly chaotic that result in a complete failure. Meanwhile, the growth in horizontal deformation at higher temperature is more gradual. Based on this, it may be concluded that the crack initiation is the major process at low temperature and the crack propagation becomes more important as test temperature increases. With this concept as a background, it can be concluded that the effect of air void content (which is a function of the binder content) on fatigue life, at high temperature is more pronounced than at low temperature (because at the higher temperature the failure is governed by crack propagation through the voids). But the difference in the air void content may not influence the fatigue resistance at low test temperature because the failure is mainly governed by the crack initiation process. That is why fatigue testing has been conducted at elevated temperature, 25 °C.

The horizontal deformation during this test was plotted against the number of cycles, Figure 7.3. The horizontal deformation increased drastically after the failure criteria limit, (Kim 1991), 2.5 mm., 0.1 inch, of horizontal deformation. Increasing the asphalt content improves fatigue life (to a certain point corresponding to the optimum binder), fatigue life then decreases. In other words, decreasing the air voids, by increasing the binder content, will provide resistance to the crack propagation up to a certain point after which any decrease in the air voids (at higher binder contents) implies a softer asphalt mixture with higher deformation. The PlusRide II asphalt-rubber mixtures proved to have higher fatigue lives since the presence of coarse rubber particles makes the mixture more flexible under loading.

The relationship between the number of load repetitions to failure and the binder content for both wet process and PlusRide II asphalt-rubber mixtures is illustrated in Figure 7.4. Increasing the binder content has improved the fatigue life to certain limit (optimum) after which fatigue life is reduced. Maximum fatigue lives are 3.75E+3 and 1.21E+4 for the wet process and the PlusRide II, respectively. The binder contents for the maximum fatigue life are 5.6% and 5.2% for the wet process and the PlusRide II, respectively.

Table 7.2 Fatigue Test Results of Asphalt Rubber Mixtures

a. Wet Process (Temperature = 25 °C and Load = 1113 N)

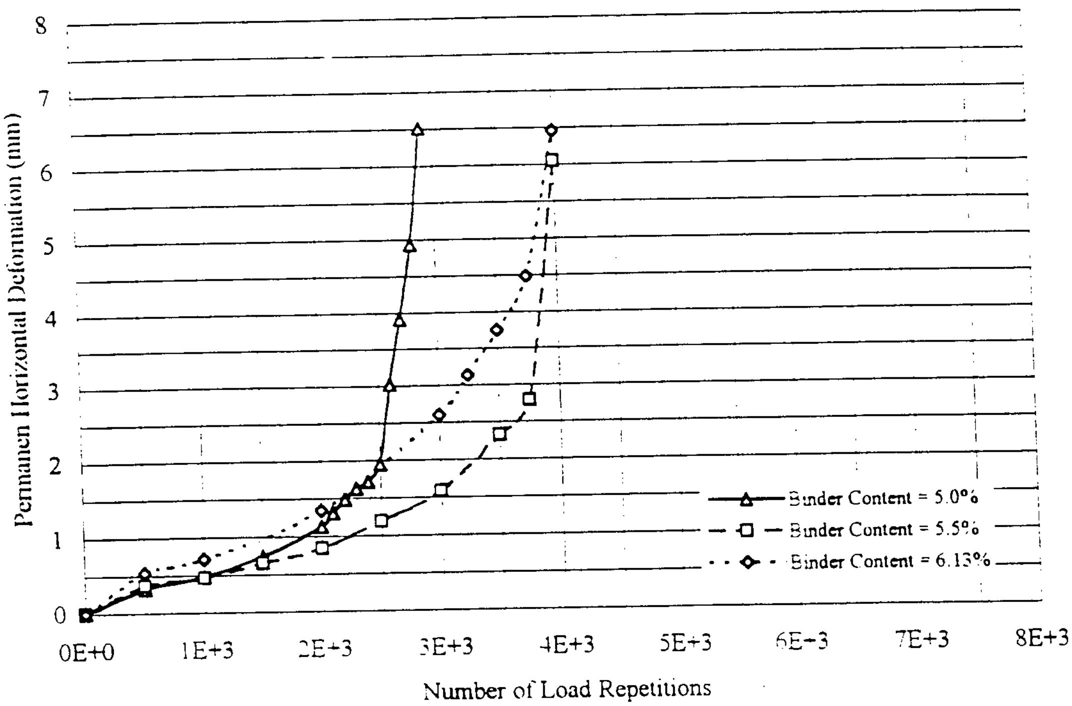
Binder Content (% by Total Mix Weight)	Number of Repetitions to Failure (N_f)
5.0	2.700E+3
5.5	3.714E+3
6.13	3.098E+3

Note: three replicates n = 3

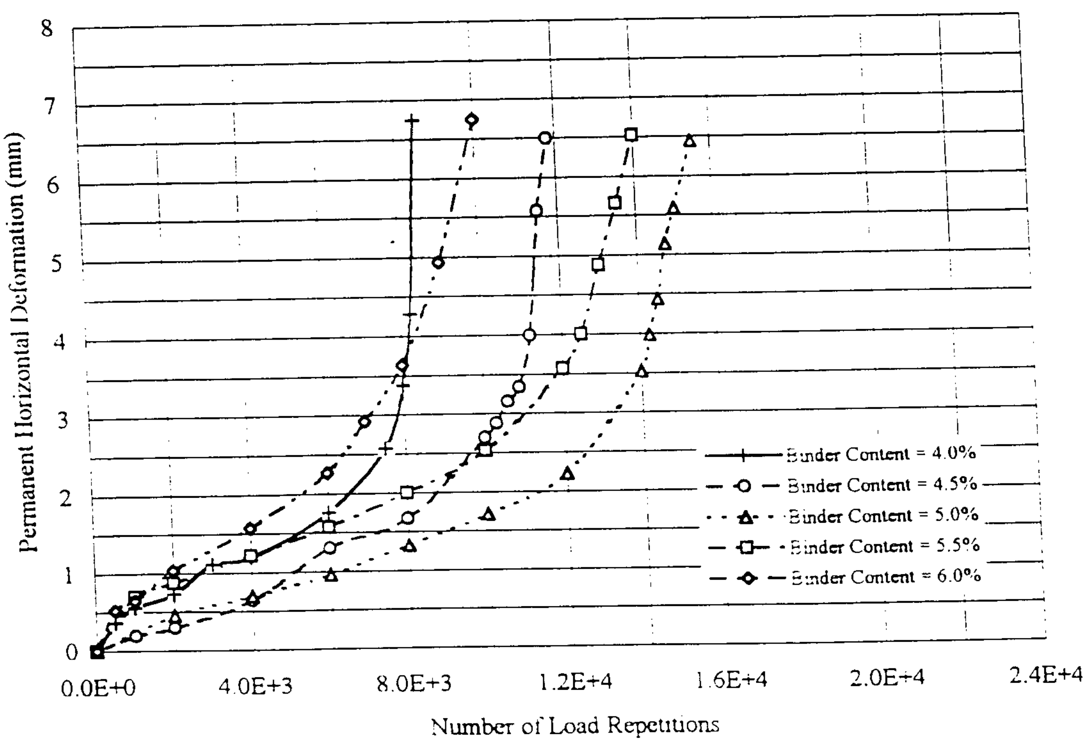
b. PlusRide II (Temperature = 25 °C and Load = 890 N)

Binder Content (% by Total Mix Weight)	Number of Repetitions to Failure (N_f)
4.0	7.368E+3
4.5	9.799E+3
5.0	1.274E+4
5.5	1.069E+4
6.0	6.659E+3

Note: three replicates n = 3

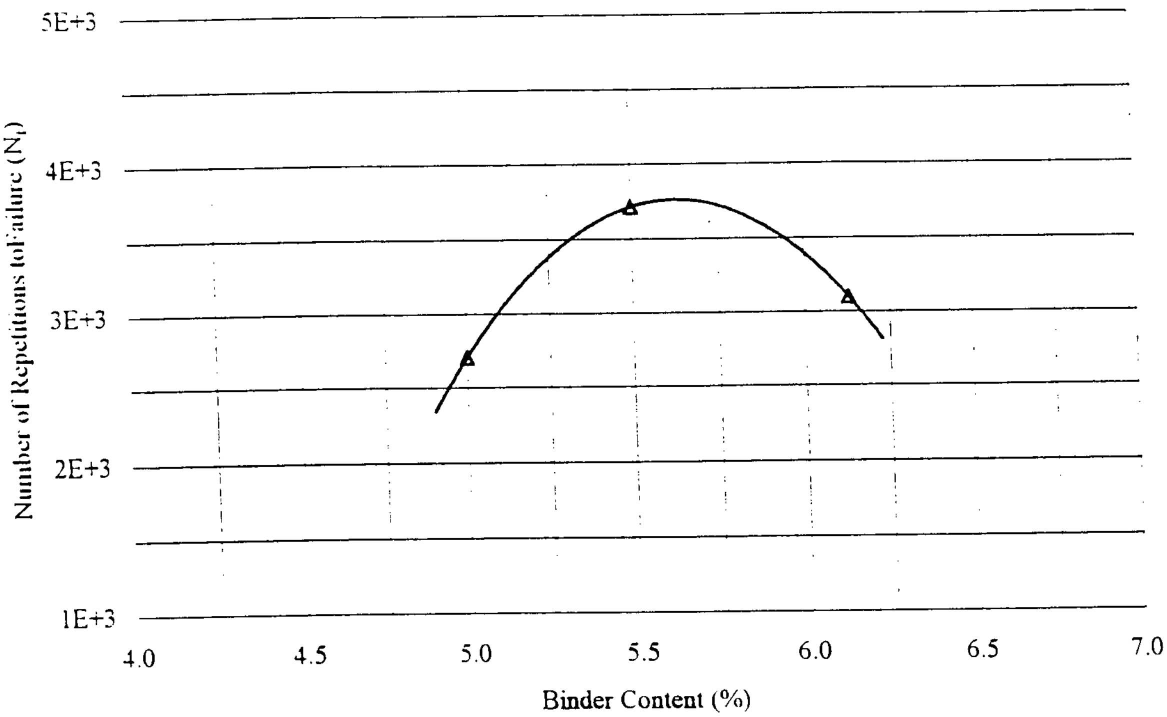


a. Wet Process (T = 25 °C and Load = 1113 N)

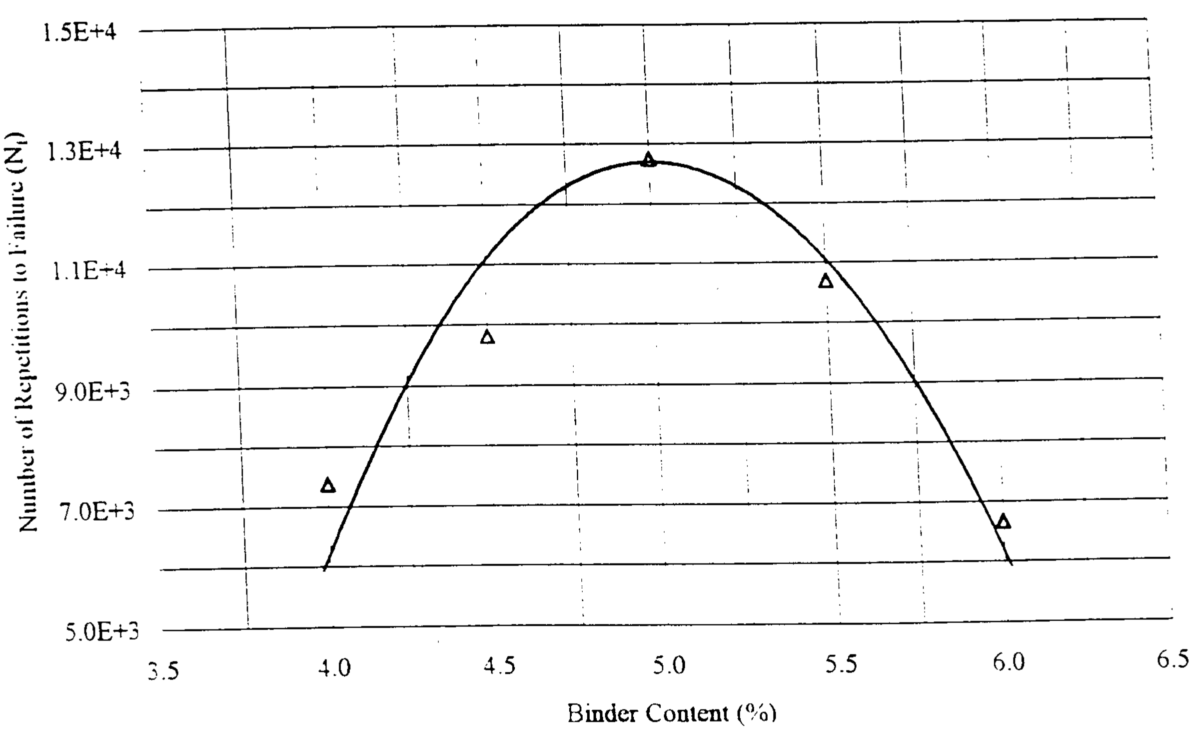


b. PlusRide II (T = 25 °C and Load = 890 N)

Figure 7.3 Permanent Horizontal Deformation versus Number of Load Repetitions



a. Wet Process (T = 25 °C and Load = 1113 N)



b. PlusRide II (T = 25 °C and Load = 890 N)

Figure 7.4 Fatigue Life versus Binder Content Relationship

CHAPTER 8. MIXTURE PERFORMANCE AND DESIGN METHODOLOGY

INTRODUCTION

Mixture design establishes the proportion of binder to aggregate that produces a paving mixture that will serve the longest time without serious pavement distress. Mixture design is often a compromise between high stiffness, which ensures strength and resistance to permanent deformation, and flexibility, which aids fatigue and fracture resistance. Several methods have been developed over the years to design asphalt mixtures, including Marshall, Hveem, Hubbard-Field, and other unconfined compression test methods. The Marshall and Hveem methods become the most popular. The Marshall method was developed by the Corps of Engineers in an effort initiated at the US Army Engineer Tulsa District in the period 1941 to 1942. It defines, as described in ASTM D-1559, the optimum asphalt content as the numerical average of three asphalt contents that yield the maximum unit weight, the maximum stability, and the selected level of air voids (3 to 5 percent). However, the Marshall criteria, (in terms of compaction energy, maximum specific gravity determination, stability and flow acceptable limits), for determining the optimum binder content are not adequate for the design of asphalt mixtures under existing loading conditions and environments as concluded from the field performance of the pavement. Because field mixture performance is not directly related to properties measured with the Marshall method, efforts continue on developing performance based mix designs. A brief description of the problems of the Marshall method of mix design follows (Brown 1989).

PROBLEMS RELATED TO THE CURRENT ASPHALT MIX DESIGN METHOD

- When the Marshall test was developed hand compactors were used in samples compaction. Mechanical compactors were subsequently developed. It has been reported that hand operated hammers yield higher specimen density compared with the mechanical hammers. At the present time, 23 States are using mechanical hammer; 1 State is using hand operated hammer; and 14 States are using both. It is obvious that the density of the prepared samples will be affected by the compaction hammer type (mechanical or manual), and this will provide different optimum binder content, for the same materials (aggregate and binder). Density is one of the mixture design parameters in optimum binder determination.

- Today, the modern traffic levels and tire pressures have resulted in higher stresses imposed in pavements, which has caused increased rutting, fatigue, and cracking. During Marshall specimens preparation, ASTM D-1559 recommend 35, 50, and 75 blows per each specimen face for mixtures to be used in light, medium and heavy traffic, respectively. However a wide disparity exists in the definition of what constitutes a heavy, medium or light traffic. For example, average daily traffic (ADT) for heavy traffic ranges from 1000 to 10000 vpd. Some States use the number of equivalent 80.1 KN (18 kips) single axle load for such categories. This concludes that the specified range in ASTM D-1559 for specimen compaction does not reflect the actual loading conditions in the filed. For example, a research project on the problems of Marshall mix design in the Middle East resulted in the use of an alternative method of specimen preparation. The asphalt mixture was designed according to the Marshall method, ASTM D-1559. Few months after construction, the asphaltic concrete rutted seriously after the highway has been opened to traffic. Investigation showed that good-quality aggregate had been used, the binder content and gradations were close to that evaluated and specified by ASTM D-1559 and project specifications, and the asphalt cement had a penetration of less than 60 (0.1 mm). Cores were taken in the wheel paths and tested. The air voids were found to be virtually zero in many cases. This concluded that compaction energy specified by Marshall does not represent the actual traffic loading conditions in the field. It is obvious that the density of Marshall specimens is one of the accounted parameters in optimum asphalt content determination.

- ASTM D-1559 specifies the use of 101.6 mm (4 inches) diameter specimen molds for mixes containing aggregate up to 25.4 mm (1 inch) maximum size. There is no provision for binder or base course mixes which might contain aggregate of 38.1 mm (1.5 inch) or 50.8 mm (2 inches) maximum size.

- The theoretical maximum specific gravity of the mix is always used to analyze the void parameters, such as air voids, voids in the mineral aggregate, and voids filled with asphalt in compacted specimens. The test method used to determine this property varies between the states. 30 States are using the Rice method (ASTM D-204), 6 states are using the calculation method (using aggregate proportions and specific gravities), solvent immersion method is being used by one state, and 1 state is using the Michigan method. Therefore, different void parameters will be obtained for the same mixtures. Since the void content is one of the parameters used in optimum binder evaluation by the Marshall method, different designs may be obtained for the same materials.

- Stability and flow are the two measured asphalt mixture parameters used in Marshall. Stability is the maximum load carried by the specimen till failure. The Asphalt Institute recommends a minimum stability of 2.25 KN (500 Ib) for Marshall specimen compacted with 50 blows for medium traffic. Most of the states specify significantly higher minimum values since 2.25 KN (500 Ib) is very low stability for today's pavement loading conditions. Also, Marshall stability is based on a constant stain test with 50.8 mm per minutes loading rate, which does not simulate the actual loading conditions in the field, (repeated load). Thus the actual loading of pavement mixture in the field is not directly related to the laboratory measured parameters by Marshall.

- The Asphalt Institute recommends that the optimum asphalt content is defined by the numerical average of the three asphalt contents that yield the maximum unit weight, the maximum stability, and the selected air void (3 to 5 percent). Only 8 of the 38 States using the Marshall method follow the Asphalt Institute recommendations. Actually the number of parameters used by the states in selecting the optimum asphalt content varies from one to five. The most commonly used parameters are: median air voids (28 States), maximum stability (20 States), maximum unit weight (18 states), voids in the mineral aggregate (8 States), median flow (2 states), and 2 percent air voids, film thickness and stability/flow ratio (1 state). Uncommon bases in optimum binder content evaluation reflect that neither one nor different combinations of Marshall parameters (stability, flow, density, air voids, and voids in mineral aggregate) are adequate to design asphalt mixes with satisfactory field performance.

In summary major shortfall of this empirical method, Marshall, is that its design criteria are not based on the field loading and environmental conditions. Besides, the mixture performance in the field is not directly related to the properties measured in the laboratory. Effort of this study is to develop a procedure that integrates into mix design the structural performance of the pavements. The philosophy of this mix design procedure is to design a hot mix asphalt-rubber mixture based on the expected structural performance of the pavement considering different distresses such as rutting, fatigue, creep, thermal cracking, moisture damage, and subgrade deformation.

IMPROVED ASPHALT MIXTURE DESIGN TRENDS

With improvements in technology the concept of mix design evolved (Mahboub 1988). Mixtures should be designed based on performance criteria. An improved mix design procedure shall design an HMA that will provide a stiffness to protect the layers underneath, and an adequate level of mix flexibility so as to resist the effects of repeated-loads. Once the stiffness and flexibility properties are determined to be acceptable, the permanent deformation potential of the mixture can be examined. Finally, the low-temperature fracture potential and resistance to moisture damage will be evaluated based on the basis of the mix's stiffness and tensile strength. Although all distresses could be considered, the following five distresses, resulting from load or environmental conditions, are believed to be the most important with respect to reduction in serviceability and in asphalt pavement performance:

- Subgrade excessive deformation
- Fatigue cracking
- Rutting
- Thermal cracking
- Moisture damage

Figure 8.1 shows the steps of the mixture design method. Previous chapters discussed the Marshall and performance testing results. Incorporation of pavement distress models in asphalt mixes design is explained next.

PAVEMENT DESIGN

Asphalt mixes shall be designed to satisfy the application needs. Traffic volume is one of the most significant factors affecting pavement distresses. Three different traffic levels (low, medium, and high) are assumed in this analysis for examining the impact of different design inputs in critical stresses and strains of the pavement structure. Based on this a thin, medium, and thick pavement are designed, respectively. The total Equivalent Single Axle Load (ESAL) numbers for these three traffic levels are shown in Table 8.1.

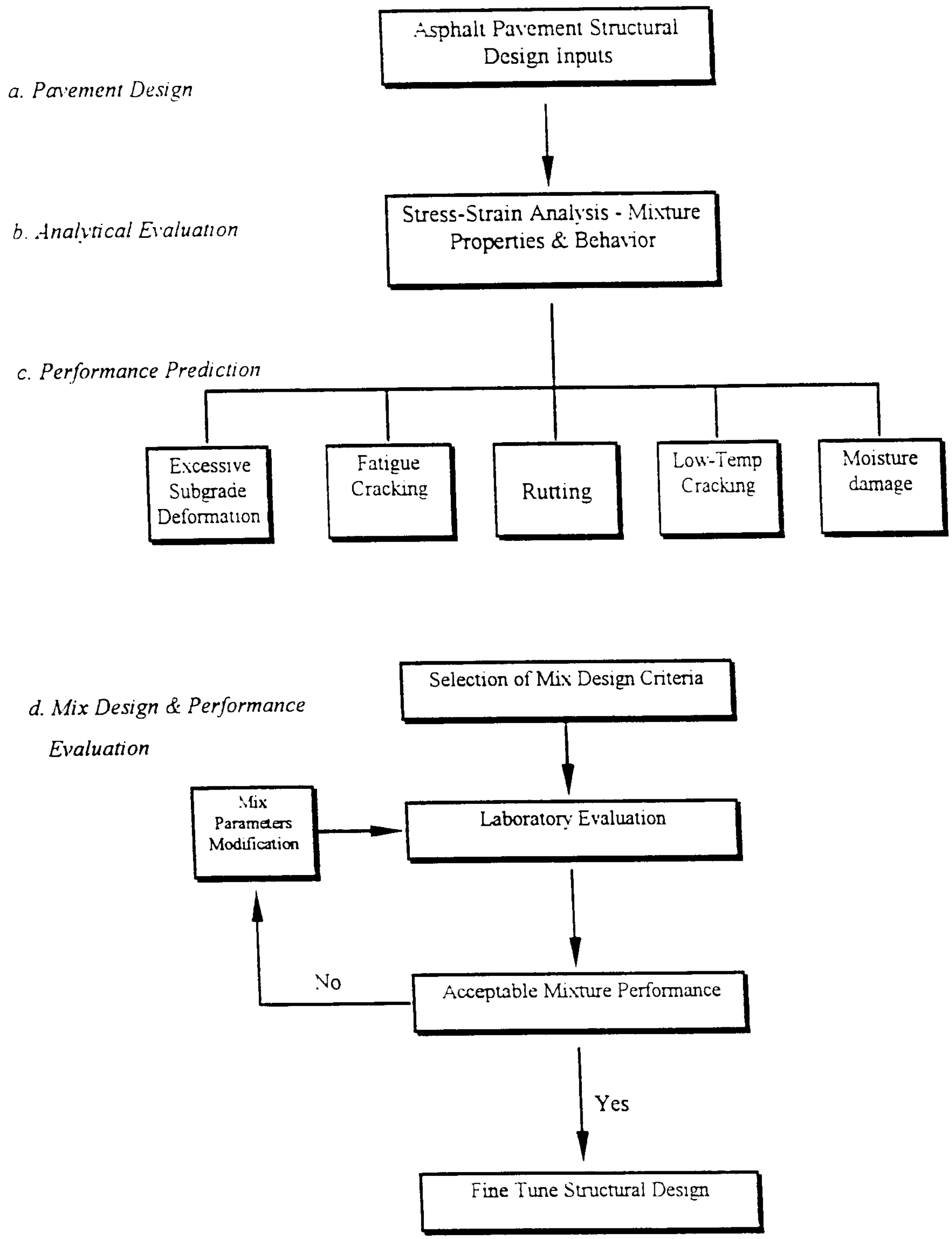


Figure 8.1 Flow Chart For Mix Design Procedure

Table 8.1 Assumed ESAL in 20-years Design Period

Traffic Level	ESAL
Low	$7x10^6$
Medium	$1.0x10^7$
High	$1.4x10^7$

Other parameters affecting the structural design were kept constant. Standard deviation and reliability levels were assumed 0.42 and 99% respectively. Initial and terminal serviceability indices were 4.5 and 2.5, respectively. Pavement drainage conditions for both base and subbase layers were assumed excellent ($m_2 = m_3 = 1$). Average values for resilient modulus and Possion's ratio for each layer were assumed as shown in Table 8.2. The resilient modulus for the asphalt surface layer was selected to represent an average value of asphalt mixtures.

Table 8.2 Layers Strength and Possion's Ratio

Layer	Resilient Modulus Kpa (Psi)	Possion's Ratio
Asphalt Surface	2.76×10^6 (4×10^5)	0.35
Base	4.83×10^5 (7×10^4)	0.40
Subbase	2.07×10^5 (3×10^4)	0.43
Subgrade	4.14×10^4 (6×10^3)	0.46

The designed thickness, using the AASHTO method, are shown in Table 8.3. Also, Figure 8.2 illustrates with schematic drawings the thin, medium, and thick pavement structures.

Asphalt Surface Course (10.2 cm)	$E = 2.76 \times 10^6$ Kpa $\mu = 0.35$
Base Course (33.0 cm)	$E = 4.83 \times 10^5$ Kpa $\mu = 0.40$
Subase Course (53.3 cm)	$E = 2.07 \times 10^5$ Kpa $\mu = 0.43$
Subgrade	$E = 4.14 \times 10^4$ Kpa $\mu = 0.46$

a. Thin Pavement

Asphalt Surface Course (11.4 cm)	$E = 2.76 \times 10^6$ Kpa $\mu = 0.35$
Base Course (33.0 cm)	$E = 4.83 \times 10^5$ Kpa $\mu = 0.40$
Subase Course (58.4 cm)	$E = 2.07 \times 10^5$ Kpa $\mu = 0.43$
Subgrade	$E = 4.14 \times 10^4$ Kpa $\mu = 0.46$

b Medium Pavement

Asphalt Surface Course (12.7 cm)	$E = 2.76 \times 10^6$ Kpa $\mu = 0.35$
Base Course (35.6 cm)	$E = 4.83 \times 10^5$ Kpa $\mu = 0.40$
Subase Course (61.0 cm)	$E = 2.07 \times 10^5$ Kpa $\mu = 0.43$
Subgrade	$E = 4.14 \times 10^4$ Kpa $\mu = 0.46$

c. Thick Pavement

Note: E = Modulus of Elasticity
μ = Possion's Ratio

Figure 8.2 Schematic Drawing and Thicknesses of Thin, Medium, and Thick Pavements

Table 8.3 Pavement Thicknesses

Layer	Thickness, cm (inch)		
	Thin Pavement	Medium Pavement	Thick Pavement
Asphalt Surface	10.2 (4.0)	11.4 (4.5)	12.7 (5.5)
Base	33.0 (13.0)	33.0 (13.0)	35.6 (14.0)
Subbase	53.3 (21.0)	58.4 (23.0)	61.0 (24.0)

ANALYTICAL EVALUATION

There are many models currently available that can be used to predict pavement performance. Major inputs for these models are stresses and strains in pavement layers due to wheel loadings. The ELSYM5 software was used in the stress and strain analysis. A single tire load of 40.05 KN (9000 Lb.) with tire pressure of 552 Kpa (80 psi) were employed in this analysis. Base and subbase layers strength characteristics were kept constant during the analysis for thin, medium, and thick pavements, Table 8.2. To investigate the effect of asphalt layer strength on pavement performance, five different resilient modulus levels were used ($6.90x10^5$, $1.38x10^6$, $2.07x10^6$, $2.76x10^6$, and $3.45x10^6$ Kpa). Also, the effect of subgrade strength on pavement performance was taken into account during this analysis by examining three different levels of subgrade strength, (weak, moderate, and strong), with resilient modulus values of $2.41x10^4$, $4.83x10^4$, $1.03\ x10^5$ Kpa, respectively. Layers Possion's ratio are kept constant as in Table 8.2. The following parameters were calculated at different locations, Figure 8.3:

- Vertical compressive strain at the top of the subgrade
- Horizontal tensile strain at the bottom of the asphalt layer
- Vertical compressive stress at the mid-depth of the asphalt layer

Tables 8.4, 8.5, and 8.6 show the calculated parameters. The relationships between asphalt layer resilient modulus and these parameters are illustrated in Figures 8.4, 8.5, and 8.6.

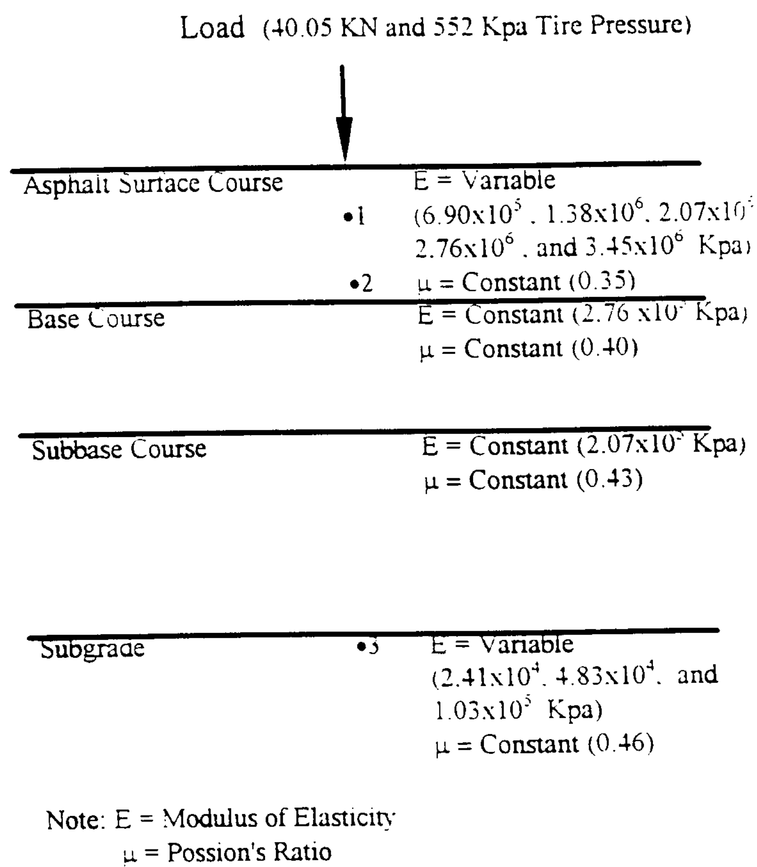
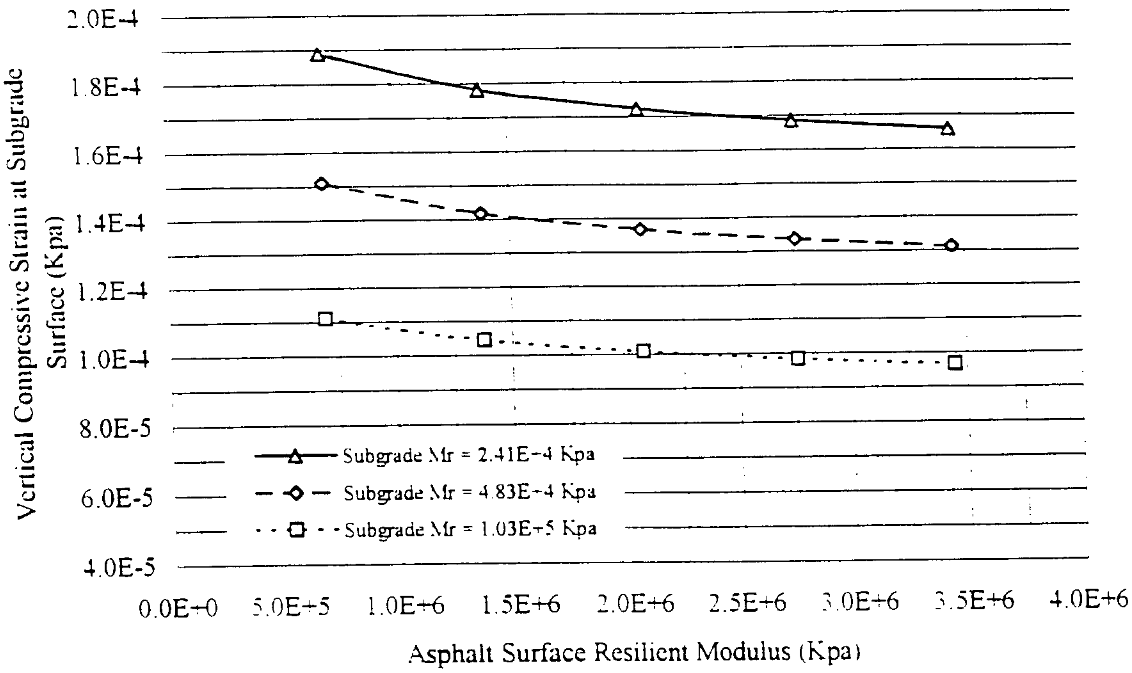


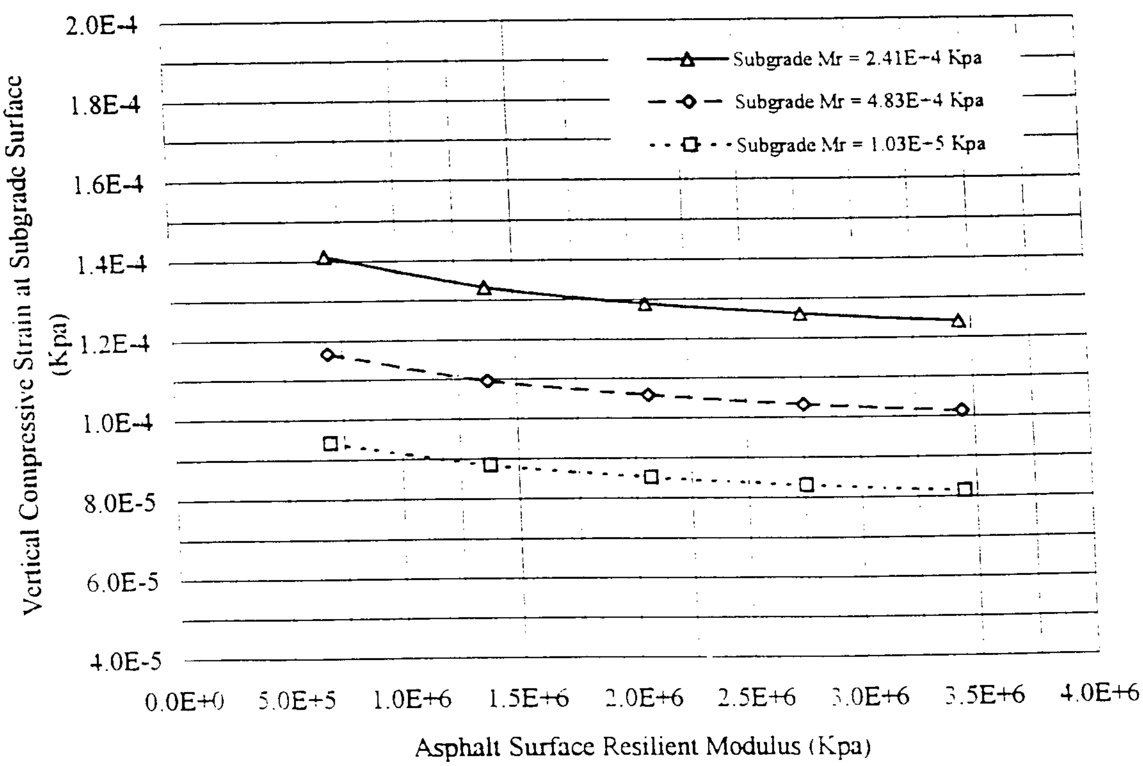
Figure 8.3 Locations for Stress & Strain Analysis

Table 8.4 Vertical Compressive Strain at The Top of the Subgrade

Subgrade Mr Kpa (Psi)	Surface Mr Kpa (Psi)	Vertical Compressive Strain at Subgrade Surface (mm/mm) Thin Pavement	Medium Pavement	Thick Pavement
2.41E+4 (3.50E+3)	6.90E+5 (1.0E+5)	1.889E-04	1.411E-04	1.511E-04
	1.38E+6 (2.0E+5)	1.780E-04	1.331E-04	1.425E-04
	2.07E+6 (3.0E+5)	1.723E-04	1.288E-04	1.380E-04
	2.76E+6 (4.0E+5)	1.685E-04	1.260E-04	1.350E-04
	3.45E+6 (5.0E+5)	1.657E-04	1.239E-04	1.327E-04
4.83E+4 (7.00E+3)	6.90E+5 (1.0E+5)	1.509E-04	1.167E-04	1.184E-04
	1.38E+6 (2.0E+5)	1.417E-04	1.096E-04	1.112E-04
	2.07E+6 (3.0E+5)	1.368E-04	1.058E-04	1.074E-04
	2.76E+6 (4.0E+5)	1.335E-04	1.032E-04	1.048E-04
	3.45E+6 (5.0E+5)	1.311E-04	1.013E-04	1.028E-04
1.03E+5 (1.50E+4)	6.90E+5 (1.0E+5)	1.110E-04	9.410E-05	8.636E-05
	1.38E+6 (2.0E+5)	1.045E-04	8.833E-05	8.098E-05
	2.07E+6 (3.0E+5)	1.008E-04	8.511E-05	7.800E-05
	2.76E+6 (4.0E+5)	9.835E-05	8.292E-05	7.595E-05
	3.45E+6 (5.0E+5)	9.647E-05	8.124E-05	7.437E-05

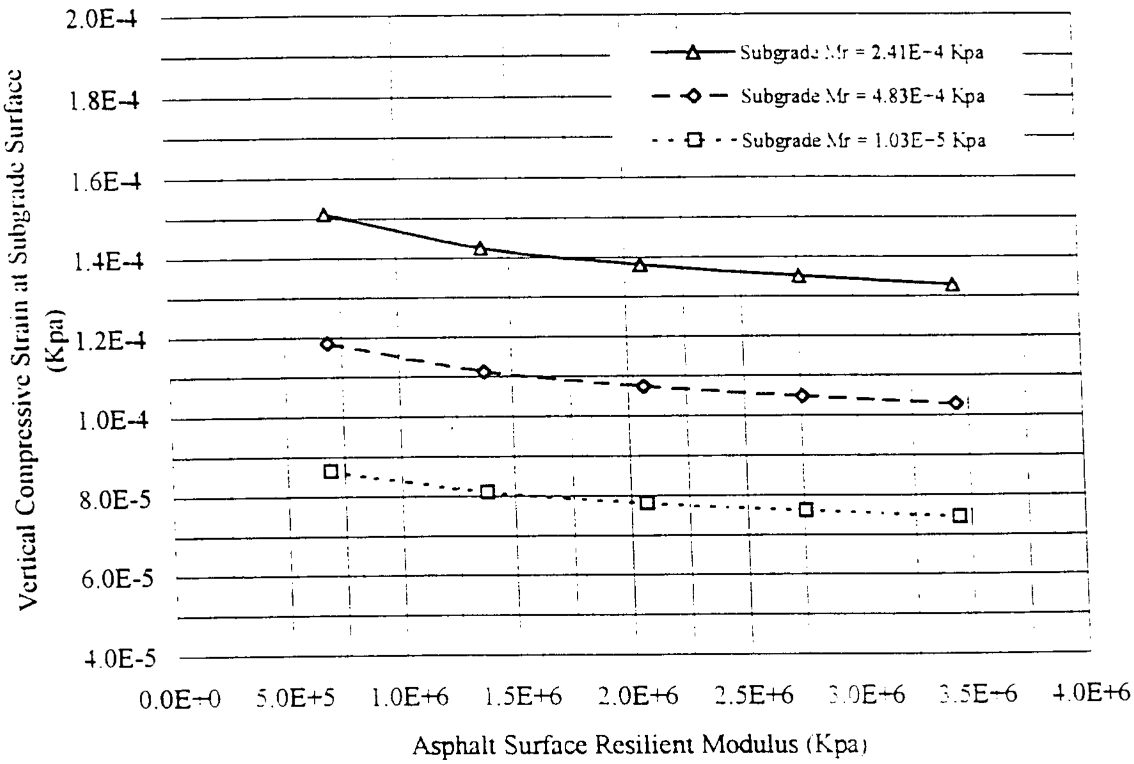


a. Thin Pavement



b. Medium Pavement

Figure 8.4 Asphalt Surface Mr & Vertical Compressive Strain at the Top of the Subgrade

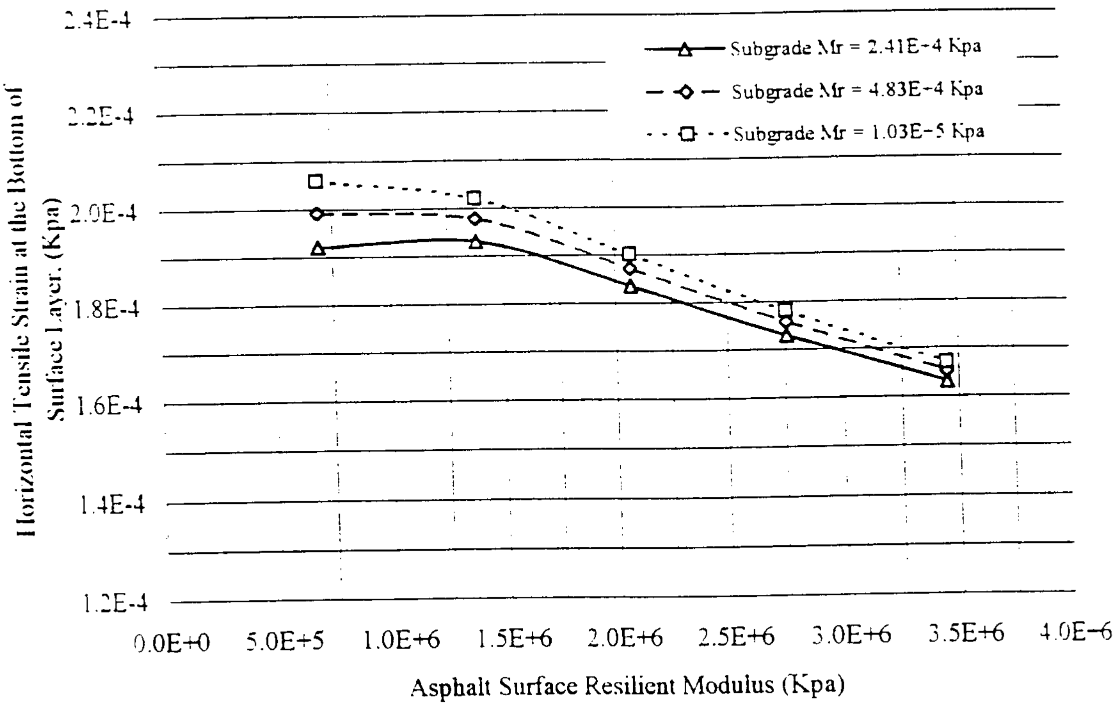


c. Thick Pavement

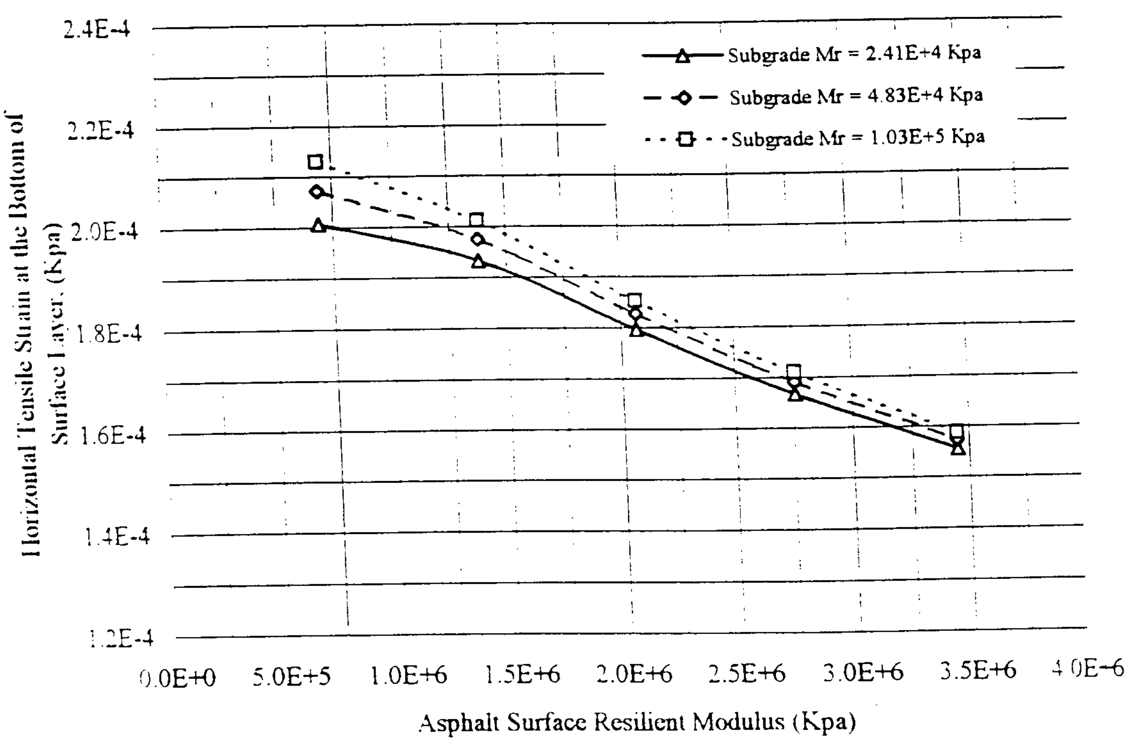
Figure 8.4 Asphalt Surface Mr & Vertical Compressive Strain at the Top of the Subgrade (Continue)

Table 8.5 Horizontal Tensile Strain at the Bottom of the Asphalt Surface Layer

Subgrade Mr Kpa (Psi)	Surface Mr Kpa (Psi)	Horizontal Tensile Strain at the Bottom of Surface Layer (mm/mm)		
		Thin Pavement	Medium Pavement	Thick Pavement
2.41E+4 (3.50E+3)	6.90E+5 (1.0E+5)	1.921E-04	2.007E-04	2.039E-04
	1.38E+6 (2.0E+5)	1.930E-04	1.932E-04	1.892E-04
	2.07E+6 (3.0E+5)	1.835E-04	1.796E-04	1.725E-04
	2.76E+6 (4.0E+5)	1.729E-04	1.667E-04	1.581E-04
	3.45E+6 (5.0E+5)	1.630E-04	1.554E-04	1.460E-04
4.83E+4 (7.00E+3)	6.90E+5 (1.0E+5)	1.992E-04	2.072E-04	2.098E-04
	1.38E+6 (2.0E+5)	1.978E-04	1.975E-04	1.930E-04
	2.07E+6 (3.0E+5)	1.871E-04	1.827E-04	1.753E-04
	2.76E+6 (4.0E+5)	1.757E-04	1.691E-04	1.602E-04
	3.45E+6 (5.0E+5)	1.652E-04	1.573E-04	1.477E-04
1.03E+5 (1.50E+4)	6.90E+5 (1.0E+5)	2.059E-04	2.130E-04	2.149E-04
	1.38E+6 (2.0E+5)	2.021E-04	2.012E-04	1.963E-04
	2.07E+6 (3.0E+5)	1.902E-04	1.853E-04	1.775E-04
	2.76E+6 (4.0E+5)	1.780E-04	1.710E-04	1.619E-04
	3.45E+6 (5.0E+5)	1.670E-04	1.587E-04	1.489E-04

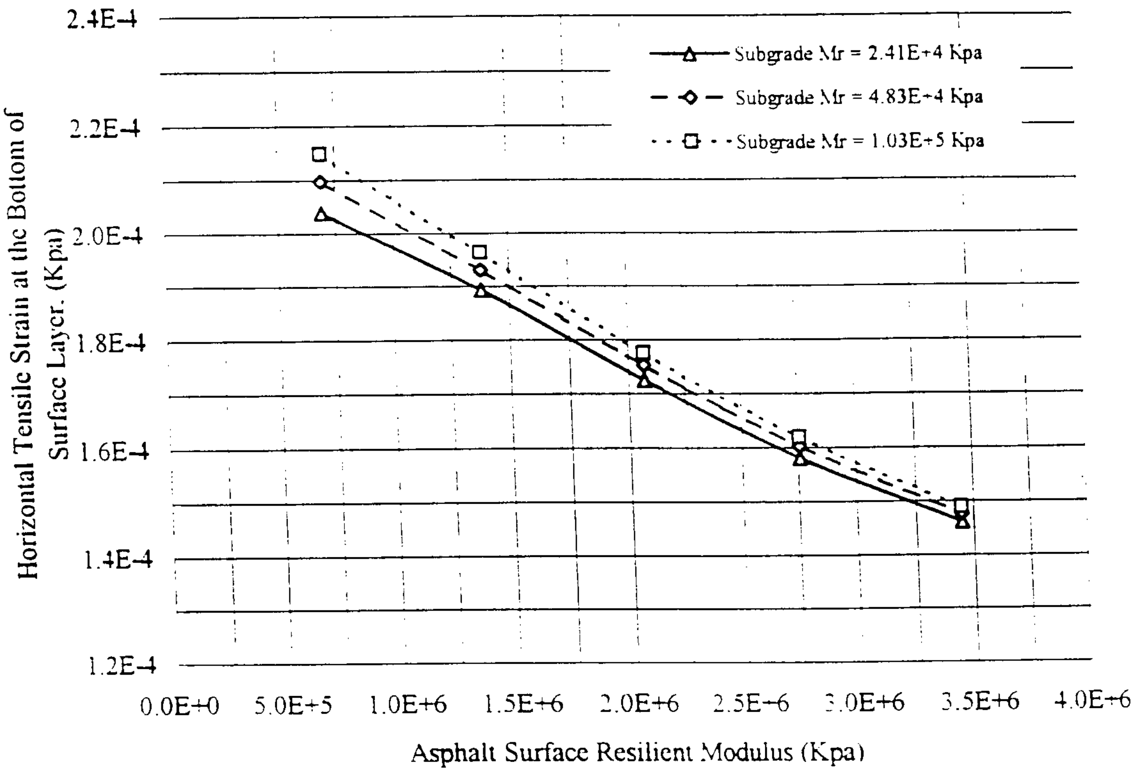


a. Thin Pavement



b. Medium Pavement

Figure 8.5 Asphalt Surface Mr & Horizontal tensile Strain at the Bottom of the Surface Layer



c. Thick Pavement

Figure 8.5 Asphalt Surface Mr & Horizontal tensile Strain at the Bottom of the Surface Layer (Continue)

Table 8.6 Vertical Compressive Stress at Mid-Depth of the Asphalt Layer

a. For different strength level

Subgrade Mr Kpa (Psi)	Surface Mr Kpa (Psi)		Vertical Compressive Stress at Asphalt Layer Mid-Depth (Kpa)		
			Thin Pavement	Medium Pavement	Thick Pavement
2.41E+4 (3.50E+3)	6.90E-5	(1.0E+5)	494.8	480.3	466.5
	1.38E-6	(2.0E+5)	469.3	452.4	437.6
	2.07E-6	(3.0E+5)	451.0	433.5	418.5
	2.76E+6	(4.0E+5)	437.0	419.4	404.7
	3.45E-6	(5.0E+5)	425.7	408.2	394.0
4.83E+4 (7.00E+3)	6.90E-5	(1.0E+5)	494.9	480.4	466.6
	1.38E-6	(2.0E+5)	469.4	452.5	437.7
	2.07E-6	(3.0E+5)	451.1	439.8	418.6
	2.76E+6	(4.0E+5)	437.1	419.5	404.8
	3.45E+6	(5.0E+5)	425.8	408.4	394.1
1.03E+5 (1.50E+4)	6.90E+5	(1.0E+5)	495.0	480.5	466.7
	1.38E+6	(2.0E+5)	469.5	452.7	437.8
	2.07E-6	(3.0E+5)	451.3	433.8	418.8
	2.76E-6	(4.0E+5)	437.3	419.7	405.0
	3.45E-6	(5.0E+5)	426.0	408.6	394.3

b. Average values for three levels of subgrade strength

Surface Mr Kpa (Psi)		Vertical Compressive Stress at Asphalt Layer Mid-Depth (Kpa)		
		Thin Pavement	Medium Pavement	Thick Pavement
6.90E+5	(1.0E+5)	494.9	480.4	466.6
1.38E+6	(2.0E+5)	469.4	452.6	437.7
2.07E-6	(3.0E+5)	451.1	435.7	418.7
2.76E+6	(4.0E+5)	437.1	419.5	404.8
3.45E+6	(5.0E+5)	425.9	408.4	394.1

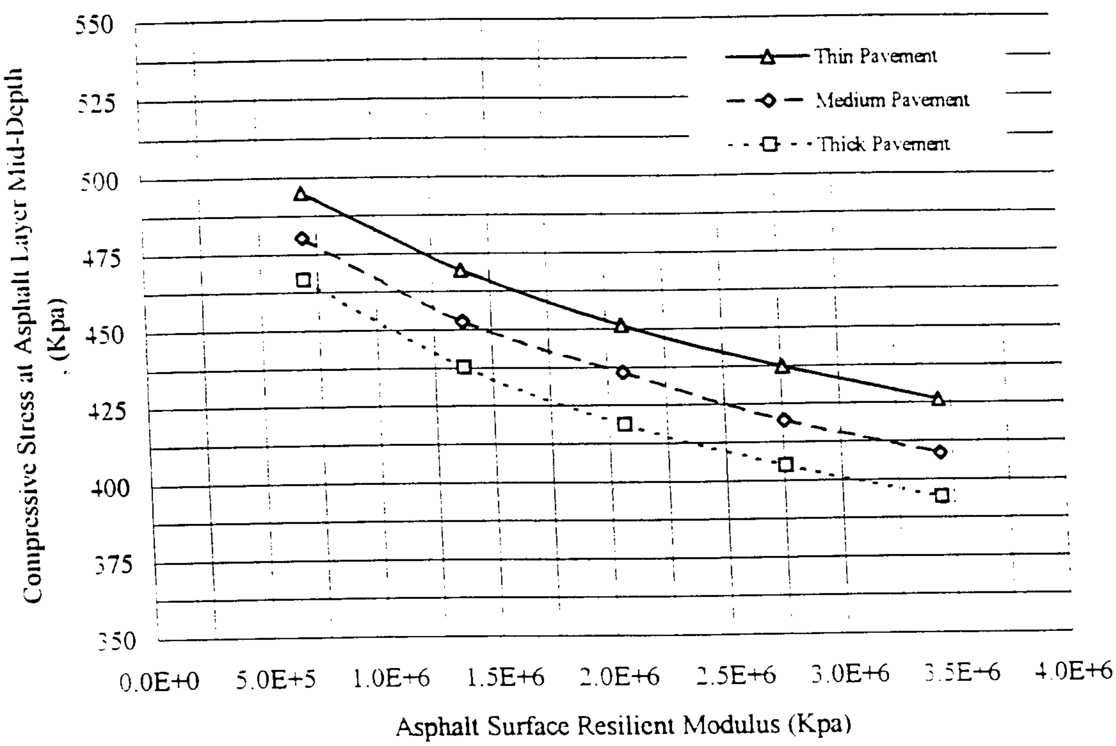


Figure 8.6 Asphalt Surface Mr & Vertical Compressive Stress at the Surface Layer Mid-Depth

PERFORMANCE PREDICTION

Over the past two decades much progress was made in measuring fundamental mix properties (such as resilient modulus, fatigue, and creep), and correlating them with mix parameters (such as aggregate gradation, asphalt cement content, and mix air voids). Objective is always to design asphalt mixtures based on expected pavement performance. Several models have been developed to correlate pavement performance, in terms of rutting, fatigue and cracking to asphalt mixture characteristics (resilient modulus, indirect tensile strength, creep, and stiffness), pavement and environmental conditions (temperature, thickness, and moisture), and traffic loading conditions (load magnitudes and repetitions). Models for pavement performance prediction are summarized next. Based on early studies' recommendations, these models were sound and reliable in performance prediction.

a. Subgrade Excessive Deformation

Natural subgrade is the weakest layer in any pavement structure. So, it should be protected from being overstressed from the above layers. A criterion should be incorporated in the asphalt mix design to insure an adequate level of subgrade protection from high vertical compressive stress and strain. Shell researchers developed an empirical model which was reported by Monismith and Finn (1977) to correlate the number of 80.1 KN (18 Kips) axle loads with the repeated vertical compressive strain at the top of the subgrade. A total subgrade vertical deformation of 19.0 mm (¾ inch) was identified as a criterion for the pavement failure. It was believed that a deformation higher than 19.0mm will seriously crack and damage the pavement layers. Also, pavement strength and riding comfortability will be decreased. This model was used in this analysis.

$$N_{18} = 6.15 \times 10^{-7}\, \varepsilon_3^{-4}$$

Where:

N_{18} = Number of 80.1 KN axle passes that will cause 19 mm rutting in the subgrade

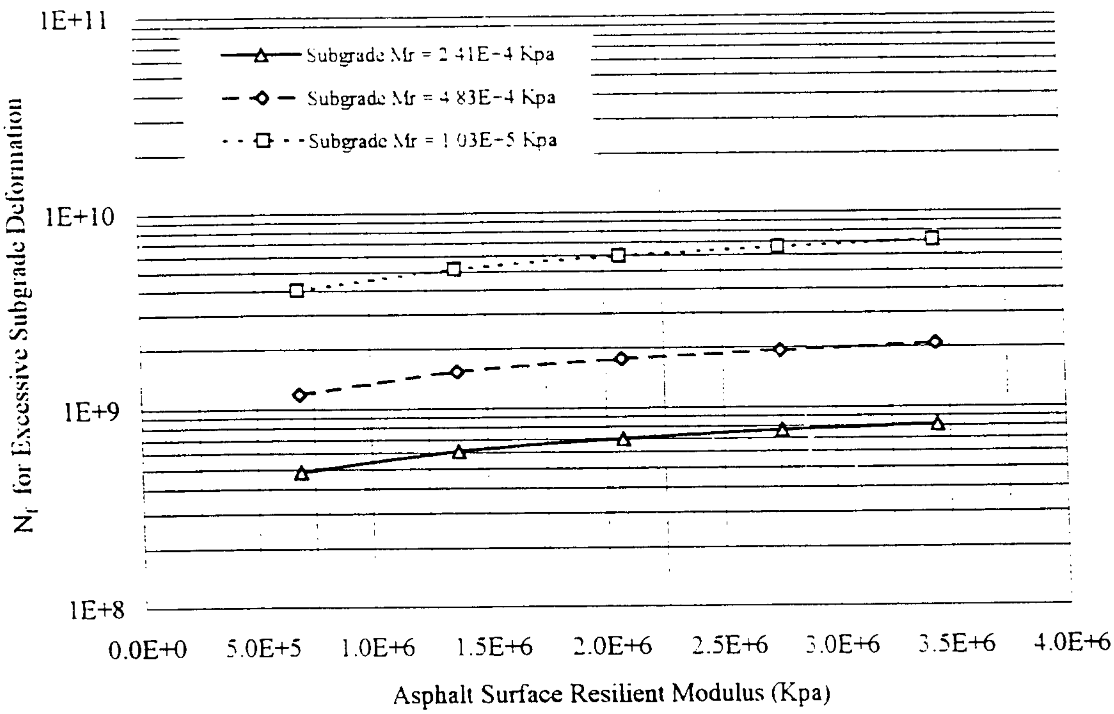
ε_3 = Repeated vertical compressive strain at the top of the subgrade

Predicted total ESAL causing excessive subgrade deformation for different pavement structure, subgrade strength, and asphalt layer resilient modulus are shown in Table 8.7. The relationship between asphalt layer resilient modulus and predicted ESAL (for excessive subgrade deformation) at different subgrade strength is illustrated in Figure 8.7.

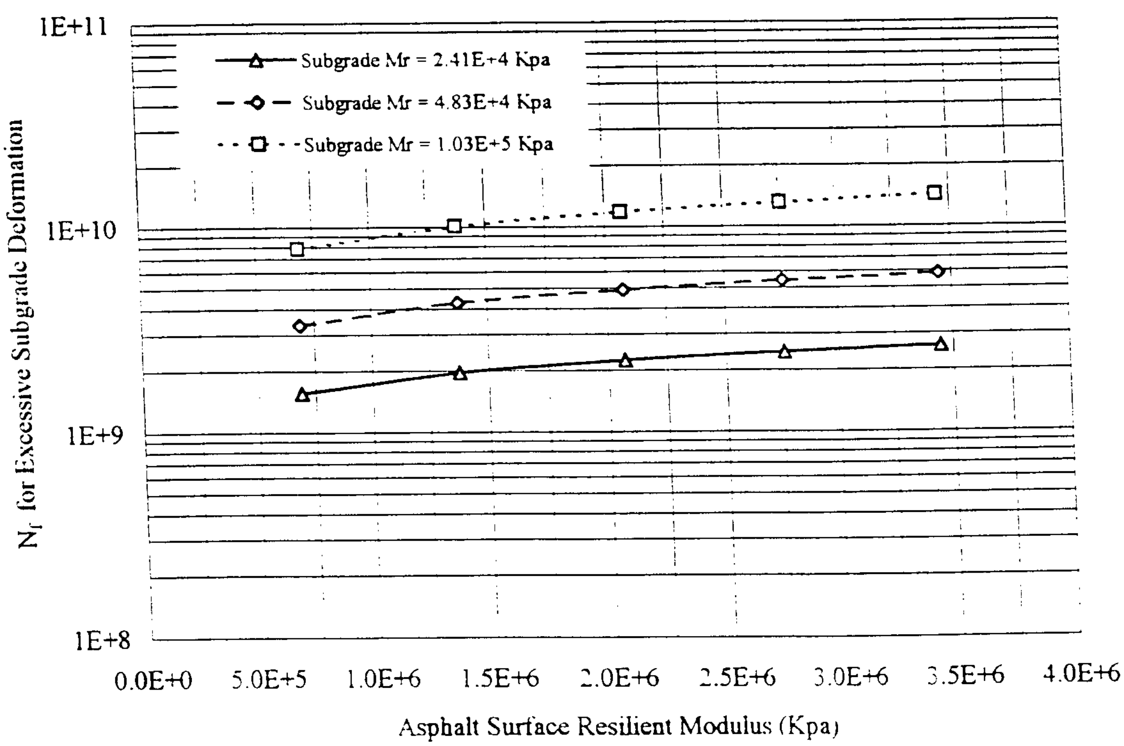
Table 8.7 Predicted ESAL for Excessive Subgrade Deformation

Subgrade Mr Kpa (Psi)	Surface Mr Kpa (Psi)	N_{18} (ESAL) for Excessive Subgrade Deformation: Thin Pavement	Medium Pavement	Thick Pavement
2.41E+4 (3.50E+3)	6.90E+5 (1.0E+5)	4.830E+08	1.552E+09	1.180E+09
	1.38E+6 (2.0E+5)	6.126E+08	1.960E+09	1.491E+09
	2.07E+6 (3.0E+5)	6.978E+08	2.235E+09	1.696E+09
	2.76E+6 (4.0E+5)	7.629E+08	2.440E+09	1.852E+09
	3.45E+6 (5.0E+5)	8.158E+08	2.610E+09	1.983E+09
4.83E+4 (7.00E+3)	6.90E+5 (1.0E+5)	1.186E+09	3.316E+09	3.129E+09
	1.38E+6 (2.0E+5)	1.525E+09	4.262E+09	4.022E+09
	2.07E+6 (3.0E+5)	1.756E+09	4.908E+09	4.622E+09
	2.76E+6 (4.0E+5)	1.936E+09	5.422E+09	5.098E+09
	3.45E+6 (5.0E+5)	2.082E+09	5.840E+09	5.507E+09
1.03E+5 (1.50E+4)	6.90E+5 (1.0E+5)	4.051E+09	7.844E+09	1.106E+10
	1.38E+6 (2.0E+5)	5.157E+09	1.010E+10	1.430E+10
	2.07E+6 (3.0E+5)	5.957E+09	1.172E+10	1.661E+10
	2.76E+6 (4.0E+5)	6.573E+09	1.301E+10	1.848E+10
	3.45E+6 (5.0E+5)	7.101E+09	1.412E+10	2.010E+10

Note: N_{18} = Total 80.1 KN Equivalent Single Axle Load

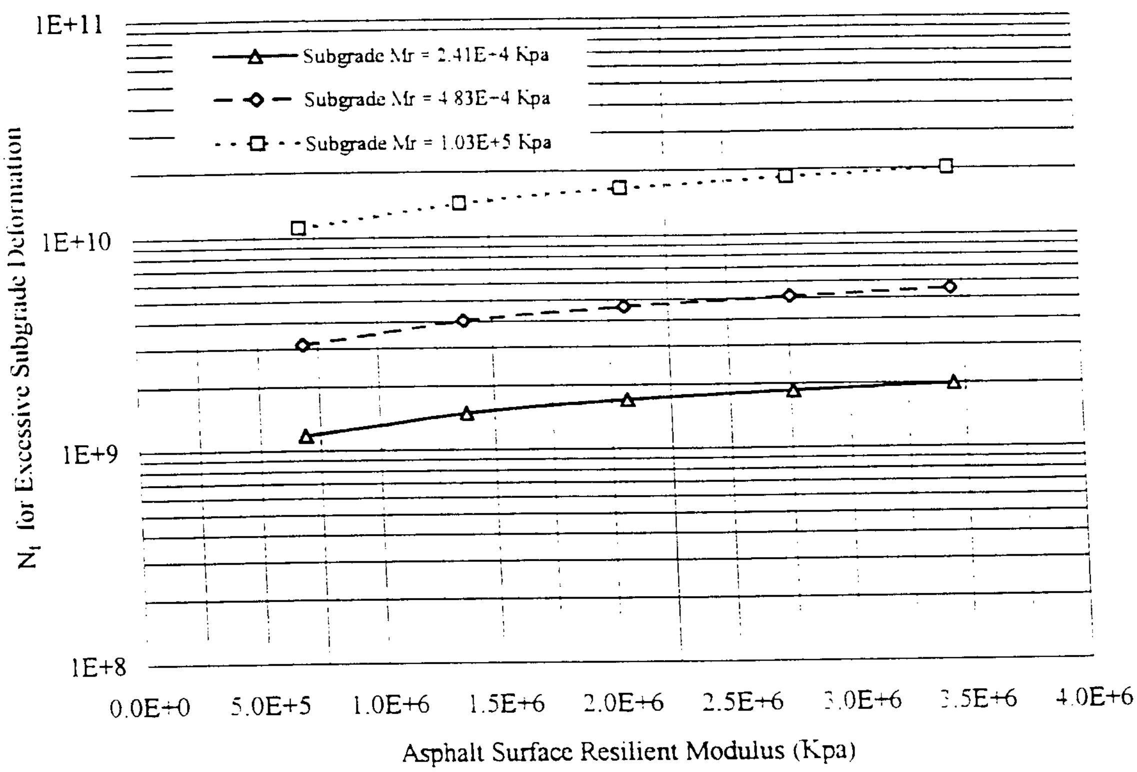


a. Thin Pavement



b. Medium Pavement

Figure 8.7 Asphalt Surface Mr & Load Repetitions to Failure (N_f) for Excessive Subgrade Deformation



c. Thick Pavement

Figure 8.7 Surface Mr & Load Repetitions to Failure (Nf) for Excessive Subgrade Deformation(Continue)

b. Fatigue Cracking

A long term distress phenomenon considered by most design and evaluation procedures is fatigue cracking. Based on laboratory and field data from the AASHTO Road Test, a model was developed to predict the fatigue life, generated by the number of 80.1 KN (18-Kips) passes that caused 10 percent fatigue cracking in the wheel path area at the Road Test (Monismith 1985). The model form is:

$$\text{Log } N_f = 15.947 - 3.291 \text{ Log } (\varepsilon_t) - 0.854 \text{ Log } (E^*/10^3)$$

Where:

N_f = Number of 80.1 KN passes to fatigue failure

ε_t = Repeated tensile strain, mm/mm x 10^6 (in/in x 10^6)

E^* = Complex modulus of asphalt mixture, psi, approximated in this study by the resilient modulus, psi.

The failure fatigue strain was calculated, using the above model, at different levels of asphalt layer resilient modulus for a range of ESALs, Table 8.8. The relationship between failure fatigue strain and asphalt layer resilient modulus at different load application levels, N_f, is illustrated in Figure 8.8.

Table 8.8 Fatigue Cracking Prediction

Surface Mr Kpa (Psi)	Failure Fatigue Strain (mm/mm) for ESAL of					
	1E+5	1E+6	1E+7	1E+8	1E+9	1E-10
6.90E-4 (1.0E+4)	1.166E-03	5.794E-04	2.878E-04	1.430E-04	7.103E-05	3.528E-05
6.90E-5 (1.0E+5)	6.417E-04	3.188E-04	1.584E-04	7.866E-05	3.908E-05	1.941E-05
1.38E-6 (2.0E+5)	5.361E-04	2.663E-04	1.323E-04	6.571E-05	3.264E-05	1.622E-05
2.07E-6 (3.0E+5)	4.826E-04	2.397E-04	1.191E-04	5.915E-05	2.938E-05	1.460E-05
2.76E-6 (4.0E+5)	4.478E-04	2.225E-04	1.105E-04	5.490E-05	2.727E-05	1.355E-05
3.45E-6 (5.0E+5)	4.226E-04	2.099E-04	1.043E-04	5.181E-05	2.574E-05	1.278E-05
5.52E-6 (8.0E+5)	3.741E-04	1.858E-04	9.232E-05	4.586E-05	2.278E-05	1.132E-05

Note: ESAL = Total 80.1 KN Equivalent Single Axle Load

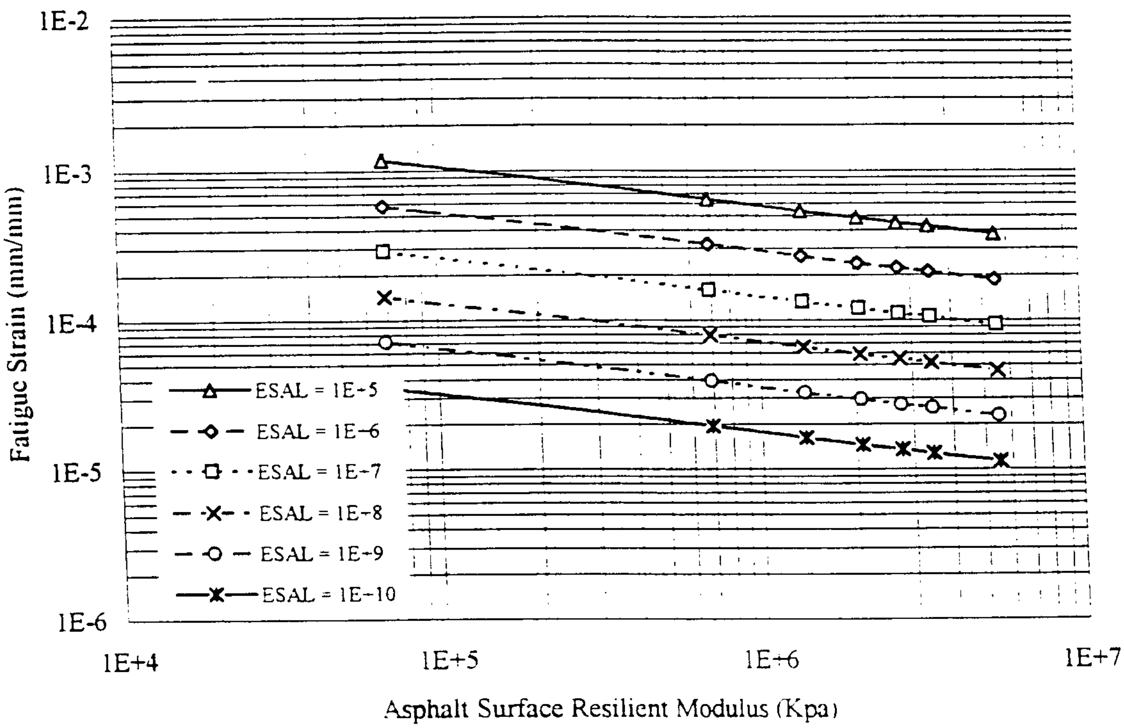


Figure 8.8 Asphalt Surface Mr & Fatigue Strain Relationship

c. *Rutting*

Kim (1991) correlated the permanent deformation in the asphalt layer (rutting) to mixture characteristics and traffic loading conditions with the following model:

$$\delta = (C_m H_o \sigma_{avg}) / S_{mix}$$

where:

δ = reduction in asphalt layer thickness (rutting), mm

C_m = correction factor for the so called dynamic effect, which takes into account the difference between static (creep) and dynamic (rutting) behavior (this factor depends on the type of mix and determined empirically). In this analysis it was assumed to be equal to 1 since results from laboratory repeated-load creep testing will be used.

H_o = Design thickness of asphalt layer under the moving wheel, mm

S_{mix} = Value of mix stiffness, Kpa

σ_{avg} = vertical compressive stress, Kpa

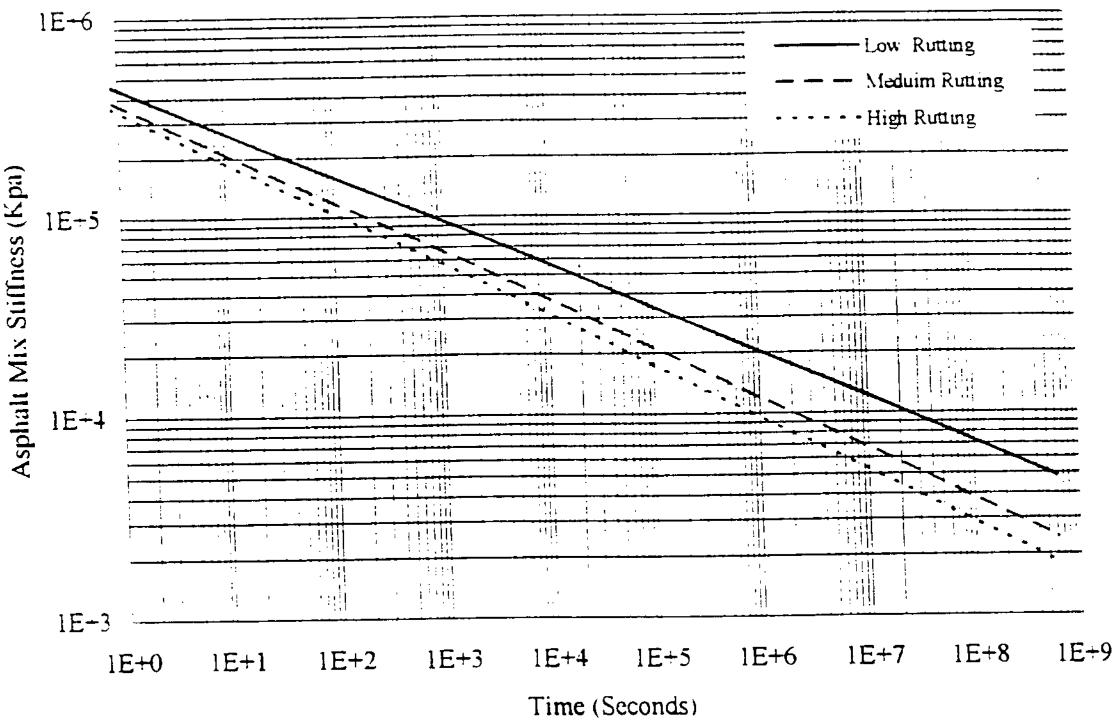
In the development of rutting criteria, limiting values or levels of severity for rutting are being specified. In this study, the limiting values for rutting were obtained from Federal Highway Administration (Monismith 1985) . Rutting has been classified in severity levels of high, medium, and low. The rutting limiting values are summarized in Table 8.9. For each rutting level, the average rut depth was used with the above rutting model to estimate the mix stiffness that will provide that level of distress, at the end of pavement design life, Table 8.10. In developing the time-stiffness relationship, Figure 8.9, an initial mix stiffness (immediately after construction, i.e. before opening the pavement to traffic) is needed. Initial mix stiffness of 2.07E+6 Kpa (3.0E+5 psi) was assumed in this analysis based on the laboratory results.

Table 8.9 Rutting Severity Classification (After Monismith 1985)

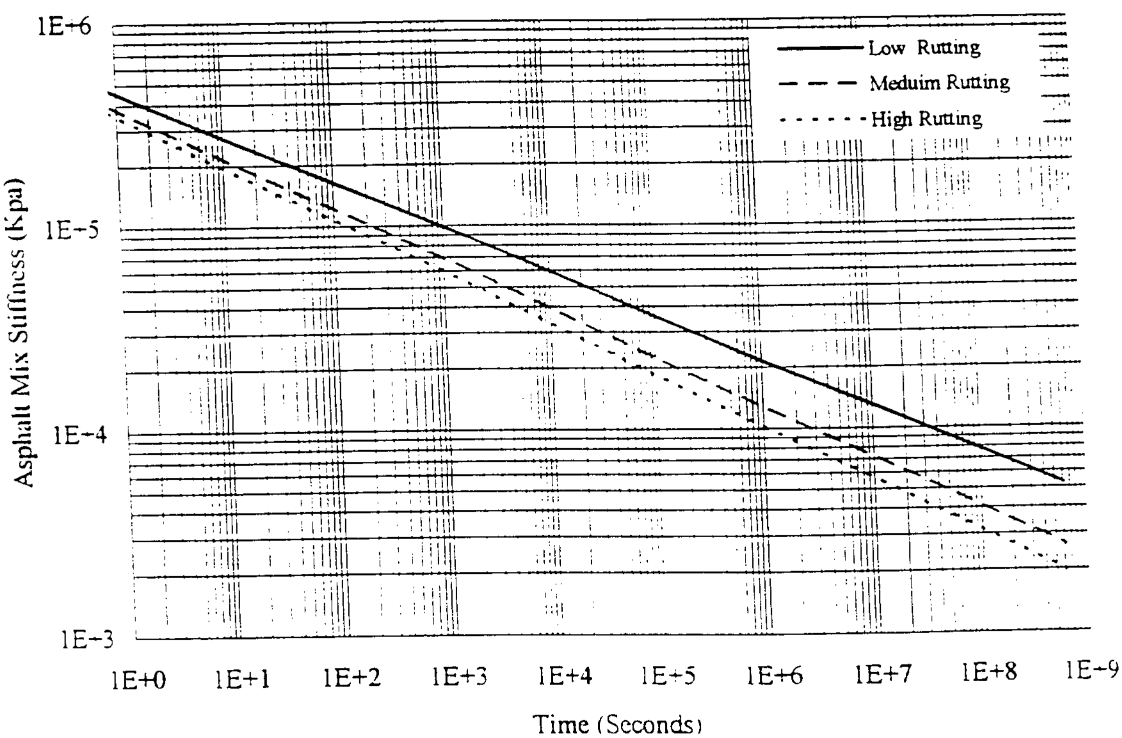
Severity	Rut Depth, mm (inch)	Average Rut Depth, mm (inch)
Low	> 6.35 - 12.7 (0.25 -0.50)	9.5 (0.38)
Medium	> 12.7 - 25.4 (0.50 - 1.00)	19.1 (0.75)
High	> 25.4 (1.00)	25.4 (1.00)

Table 8.10 Asphalt Mix Stiffness at the End of the Design Life

Pavement Type	Surface Mr		Asphalt Mix Stiffness. (Kpa)		
	Kpa	(Psi)	Low Rutting	Medium Rutting	High Rutting
Thin	2.07E+6	(3.0E+5)	4.81E+3	2.41E+3	1.80E-3
Medium	2.07E+6	(3.0E+5)	5.23E+3	2.61E+3	1.96E-3
Thick	2.07E+6	(3.0E+5)	5.58E+3	2.79E+3	2.09E-3

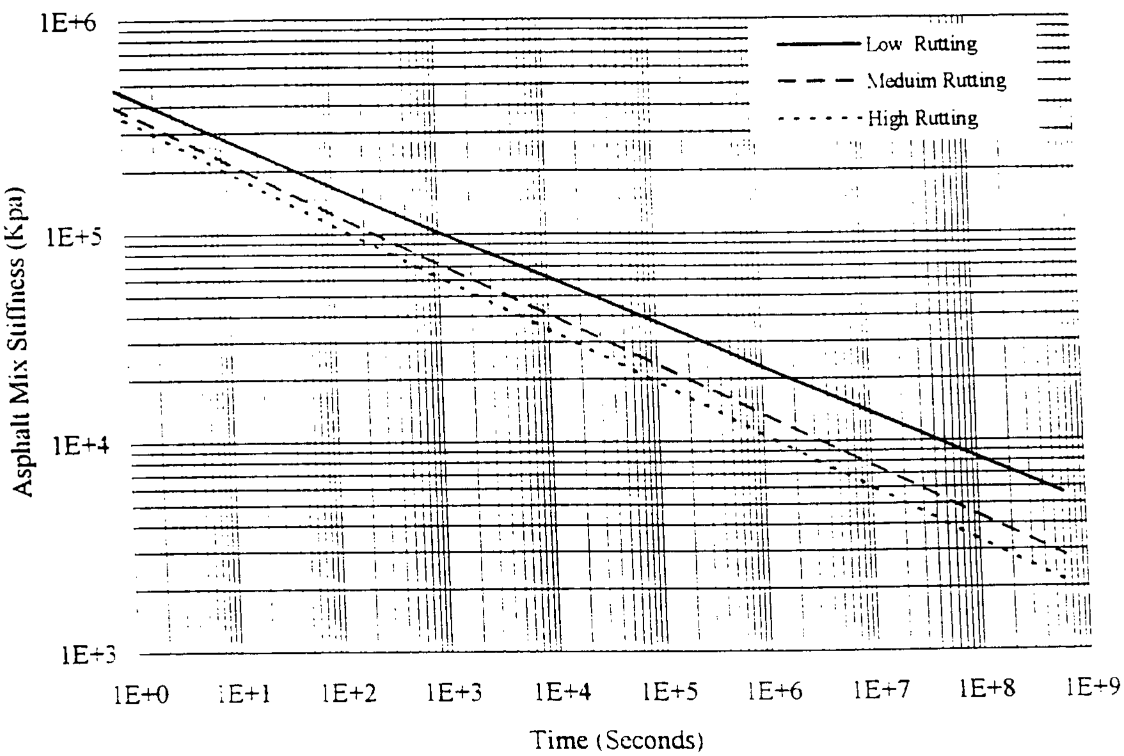


a. Thin Pavement



b. Medium Pavement

Figure 8.9 Asphalt Mix Stiffness & Time Relationship



b. Thick Pavement

Figure 8.9 Asphalt Mix Stiffness & Time Relationship (Continue) *d. Thermal Cracking Prediction*

Thermal cracking results from the stresses created by different rates of thermal contraction within the asphalt-aggregate mixture (Von Quintus 1991). As the temperature decreases, the asphalt layers will contract at a greater degree than the aggregate. This thermal induced stress condition causes microcracks to form at the pavement surface. Microcracks, they have little or no effect on pavement performance and may eventually heal. However, with repeated thermal cycles and continuous vehicle loading, many of these microcracks grow into macrocracks. Rapid deterioration results when macrocracks grow through the depth of the asphalt concrete layer, thereby allowing water to penetrate into the base layer. This type of cracking needs serious consideration when designing a mixture and it is difficult to evaluate and predict. The reason for this difficulty is related to the aging characteristics and viscoelastic properties of the asphalt. Low temperature cracking results when the tensile stresses, caused by temperature drop, exceed the mixture's fracture strength. The rate at which thermal cracks occur is dependent on rheological properties of the asphalt, the mixture properties, and environmental factors.

Thermal cracking is predictable by using models based on empirical or statistical relationships that relate cracking to asphalt and environmental parameters. To evaluate thermal cracking, certain critical mixture properties, as well as project-specific environmental conditions, must be measured. These mixture properties include indirect tensile strength, low temperature creep modulus, failure strain and the thermal coefficient of contraction.

The thermal coefficient of contraction is usually equal to 1.25×10^{-5} mm/mm for dense graded asphalt mixtures. The mixture's strength is measured using the indirect tensile strength test on aged specimens, simulating the operational and environmental effects on the asphalt pavement. Loads are applied at slow rates, 1.27 to 1.65 mm per minute in identifying models. The change in tensile stress. $\Delta\sigma$ (T_i), caused by a drop in temperature of the asphalt concrete surface layer can be calculated with the following equation (Von Quintus 1991).

$$\Delta\sigma\ (T_i) = \alpha_A\ (\Delta T_i)\ \Delta Ect$$

where α_A is the thermal coefficient of contraction of the asphalt mixture (3.0×10^{-5} to 5.4×10^{-5} mm/mm /°C); ΔT_i is the change or drop in temperature, °C; and ΔEct is the change in mixture stiffness (creep modulus) caused by a drop in temperature of ΔT_i, Kpa.

The tensile strength and stiffness of a mixture can be measured at various temperatures using slow loading rates and extended loading times, respectively. For most mixtures within a reasonable temperature range, there is a relationship between stiffness and strength that can be represented by (Von Quintus 1991):

$$\log Ect\ (T_i) = \log E_o + n_t \log S_t\ (T_i)$$

where S_t (T_i) is the indirect tensile strength measured at temperature T_i , Kpa ; E_o is a regression constant developed from the laboratory test data; n_t is the slope of the relationship between indirect tensile strength and total resilient modulus of the mixtures measured of 5, 25, and 40 °C; and Ect (T_i) is the indirect tensile creep modulus measured at temperature T_i

The stiffness and strength of the asphaltic concrete mixture vary with both temperature and loading time, as the temperature decreases. The tensile strength is constant at a particular temperature change, but the tensile stress decreases because of stress relaxation during a constant strain test. The decrease in the thermal stress due to relaxation can be approximated by (Von Quintus 1991):

$$\sigma_t (T_i) = \alpha_A (\Delta T) E_o(T_i) (t_r)^{-nc}$$

where n_c is the slope of the indirect tensile creep curve at temperature T_i ; $E_o(T_i)$ is the intercept of the indirect tensile creep curve at temperature T_i, Kpa; t_r is the relaxation time, and is assumed to be 3600 seconds for most examples; and ΔT is the critical temperature change at which cracking is expected to occur, °C. Obviously, measuring all of these properties over a range of temperatures, including values less than 0 °C, is time consuming and unpractical from a mixture design/evaluation point of view. Thus, the forgoing relationships were combined and it was assumed that the slopes, at these lower temperatures change at which cracking occurs can be estimated by the following equation.

$$\Delta T = (Ect (T_i) / E_o)^{1\ nt} \quad (t_r)^{nc} / \alpha_A E_o(T_i)$$

Laboratory testing for low-temperature cracking evaluation was not conducted in this study due to the limited time frame of this research. However, if conducted, the results of the indirect tensile test for different binder contents (at variable temperatures, loading magnitudes and rates) could be used in identifying the optimum binder content that provides indirect tensile strength higher than the expected thermal stresses .

e. Moisture Damage

Moisture damage is a serious problem, particularly on high traffic highways (Ishai 1988). Stripping (loss of adhesion) and softening (loss of cohesion) result as a consequence of the damaging action of water. Loss of adhesion and cohesion will result in increased pavement distress with corresponding loss of pavement performance. Stripping action can be initiated from either the top or bottom of the pavement. If it initiates from the top, the raveling can be spotted easily. When the stripping initiates from the bottom, it progresses rapidly and would only be noticed at the advanced stage. Internal and external factors affect the stripping of asphalt pavement. Internal (mix) factors include aggregate and asphalt characteristics, mix design, and component variations. The external factors include construction methods and environmental effects.

Several concepts were developed for explaining the stripping phenomenon. Briefly, the mechanical concept for explaining stripping assumes that the surface texture of the aggregate is the main factor responsible for the mechanical adhesion. The surface energy concept studies the wetting behavior of asphalt at the asphalt-aggregate-water-air interface. The chemical concept assumes that a chemical reaction will take place between the aggregate and the adsorbed binder in the presence of water.

Moisture damage can be evaluated using several laboratory testing methods. One of these techniques is the retained strength (such as Marshall stability, indirect tensile strength, resilient modulus, ... other) of asphalt mixture. This test involves immersing asphalt samples in water at different temperatures over a certain period of time. A schematic representation of the durability curve (plot of the retained strength versus immersion time) is illustrated in Figure 8.10.

The durability index (r) can be determined using the following equation:

$$r = \sum_{i=0}^{i=n-1} (S_i - S_{i+1}) / (t_{i+1} - t_i)$$

where i is the immersion time (in this study is 0, 1, 4, 7, and 14 days); S_{i+1} is the percent retained strength at time t_{i+1}; S_i is the percent retained strength at time t_i; t_i is the immersion time (day); and n is the total number of immersion periods (1, 2, 3,....). For strength measurements for a period of 14 days, the durability index is calculated as follows:

$$r = (S_0 - S_1)/1 + (S_1 - S_4)/3 + (S_4 - S_7)/3 + (S_7 - S_{14})/7$$

where S_0, S_1, S_4, S_7, and S_{14} are retained strengths at 0, 1, 4, 7, and 14 days, respectively. It is obvious that positive values of (r) indicate strength loss while negative values show strength gain. In terms of absolute values of the weighted loss in strength (R), the first durability index (the sum of the slopes of the consecutive sections of the durability curve) can be defined as:

$$R = r\, S_0 / 100$$

where S_0 is the absolute value of the initial strength. The units of R are the same units as S_0.

The second durability index (a) is defined in terms of the average strength loss area enclosed between the durability curve and the line S_0 = 100 percent, see Figure 8.10.

$$a = 1/t_n \times \sum_{i=1}^{i=n} (a_i) = 1/(2t_n) \times \sum_{i=1}^{i=n-1} (S_i - S_{i+1})[2t_n - (t_i + t_{i+1})]$$

From a previous study (Ishai 1988), it was concluded that the moisture damage may be evaluated by immersion of asphalt mixture samples in water for 7 days. Thus, criteria can be identified so that the durability index at a specific temperature, and after a certain immersion period (i.e. 7 days) is identified. Therefore, mixture design can be adjusted to meet such durability index criteria.

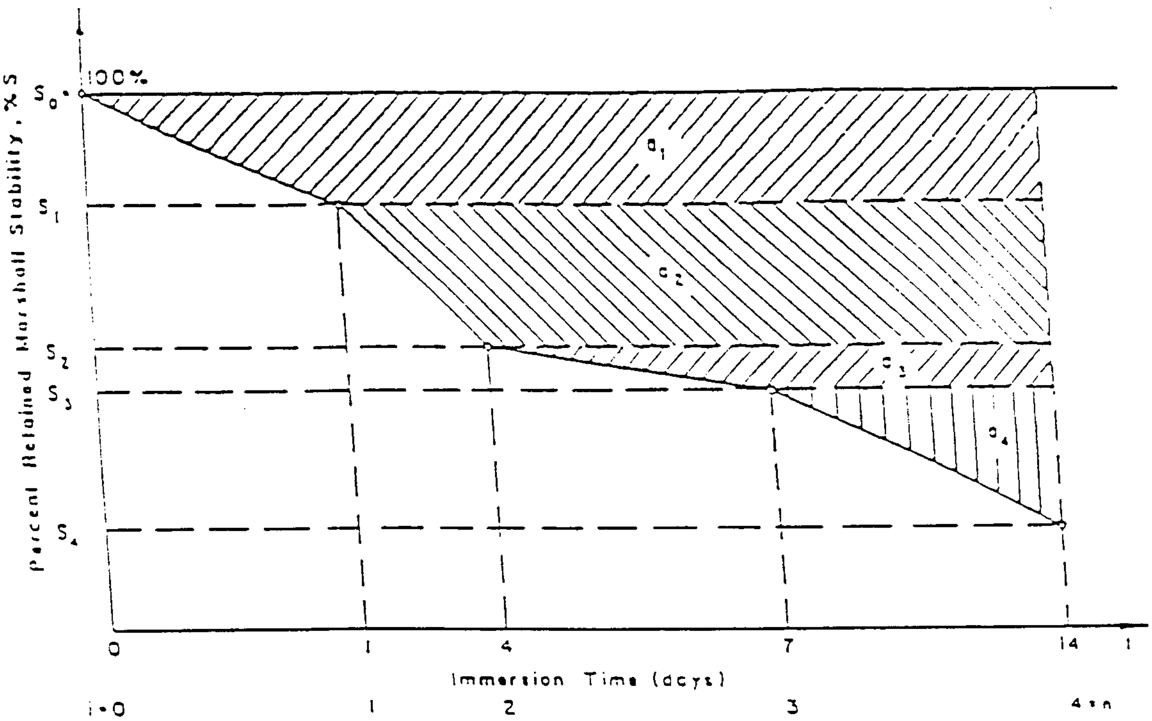


Figure 8.10 Durability Curves and Parameters Defining the Durability Index (After Ishai 1988)

MIX DESIGN & PERFORMANCE EVALUATION

The necessary analysis for mixture performance prediction was discussed in the previous section. Data from the prediction analysis may be compared against laboratory testing results to design an asphalt mixture based on performance criteria. This section explains the steps on how to satisfy the performance criteria for excessive subgrade deformation, fatigue cracking, rutting, low temperature cracking, and moisture damage, during asphalt mixture design.

a. Protection Against Excessive Subgrade Deformation

a.1 Based on the pavement analysis, evaluate the required minimum resilient modulus (Mr) for the asphalt mixture necessary to provide subgrade protection, Figure 8.7, based on pavement thickness, total ESAL, and the subgrade strength.

a.2 Based on the laboratory evaluation of the specific mixture, use the minimum Mr from step a.1 to determine the binder content range (R_s), Figures 8.11 and 8.12, for either the wet process or the PlusRide II, respectively.

a.3 Resilient modulus interpolation may be used for other in-service pavement temperatures by conducting the laboratory Mr testing over a wide range of temperatures so as to develop the relationship between Mr and the testing temperatures for different binder contents.

b. Fatigue Cracking

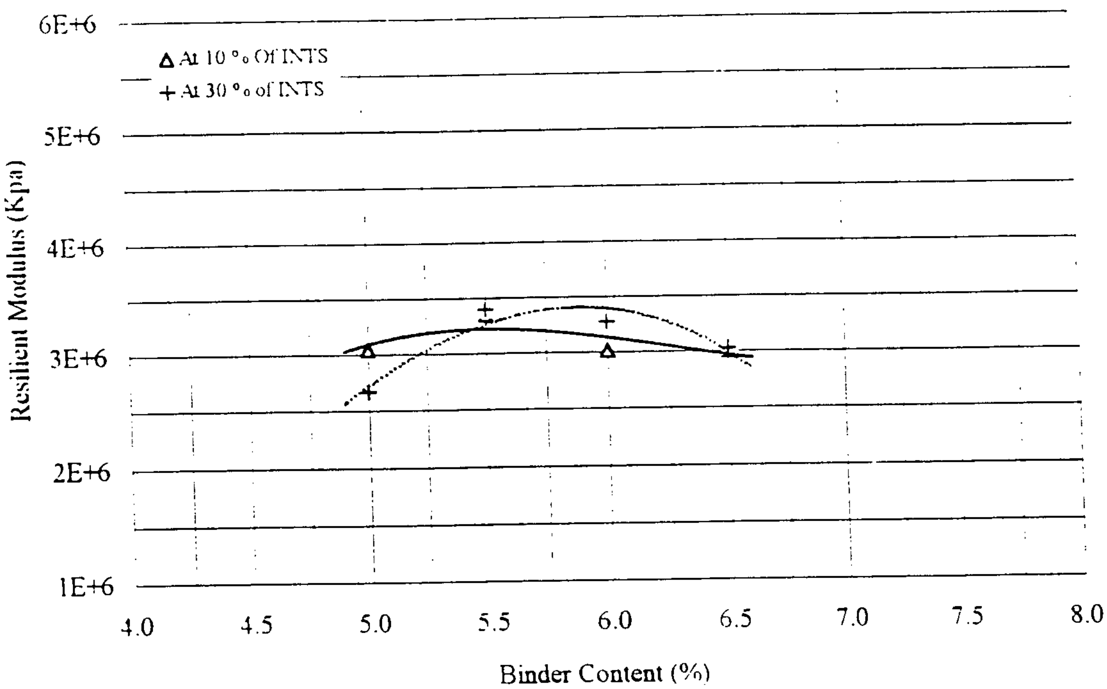
b.1 From the pavement analysis, using the Mr-value from step (a.1), determine the corresponding horizontal tensile strain generated at the bottom of the asphalt surface layer (ε_T), for the specific pavement thickness, and the subgrade strength, Figure 8.5.

b.2 Based on the fatigue prediction model for asphalt mixture and using the same Mr-value as in step (b.1) and the design ESAL, determine the fatigue failure horizontal tensile strain, Figure 8.8.

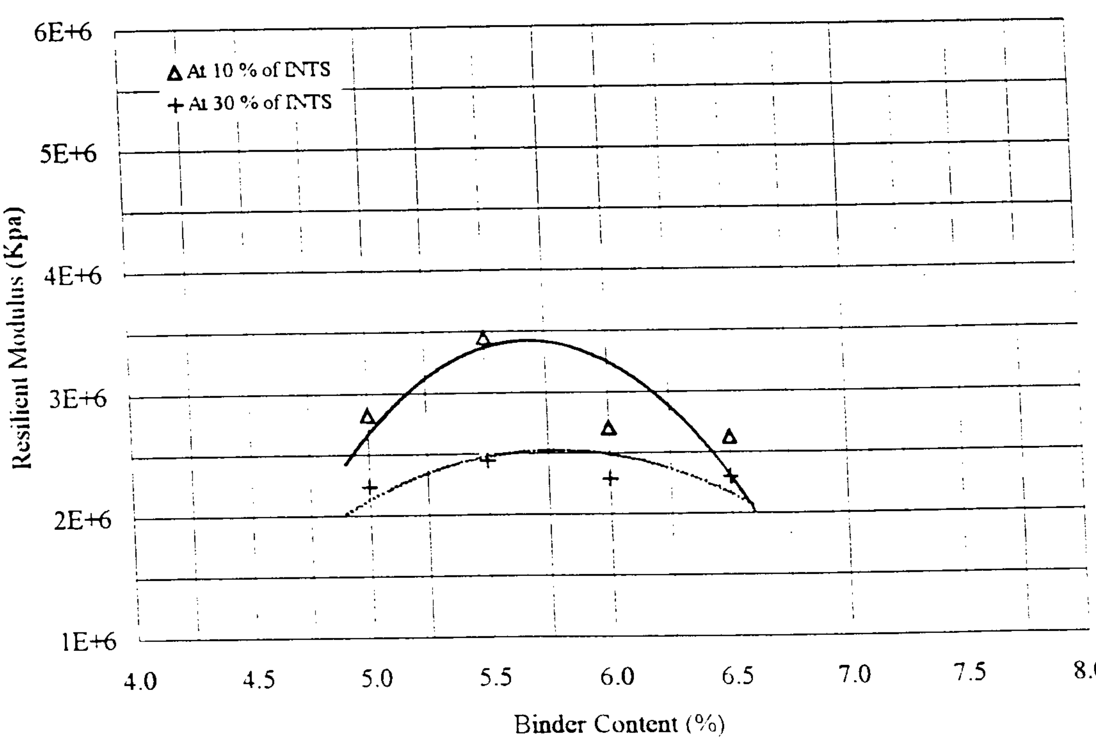
b.3 Repeat steps (b.1) and (b.2) using different Mr-values until getting the fatigue failure horizontal tensile strain (Figure 8.8) is equal to the horizontal tensile strain from the pavement analysis (Figure 8.5)

b.4 With the Mr-value from step (b.3), (where the horizontal tensile strain and the fatigue failure tensile strain values are the same), estimate the binder content range (R_f), using Figures 8.11 or 8.12, for either the wet process or the PlusRide II mixes, respectively.

b.5 Resilient modulus interpolation may be used for other in-service pavement temperatures.

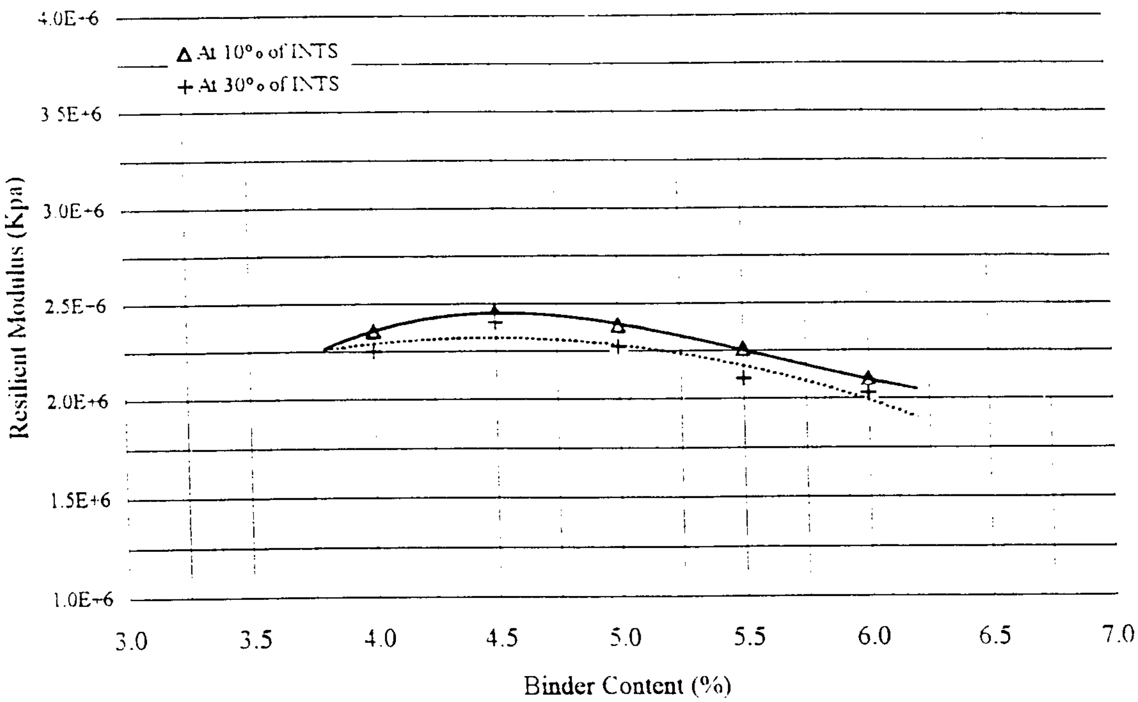


a. Testing Temperature = 5 °C

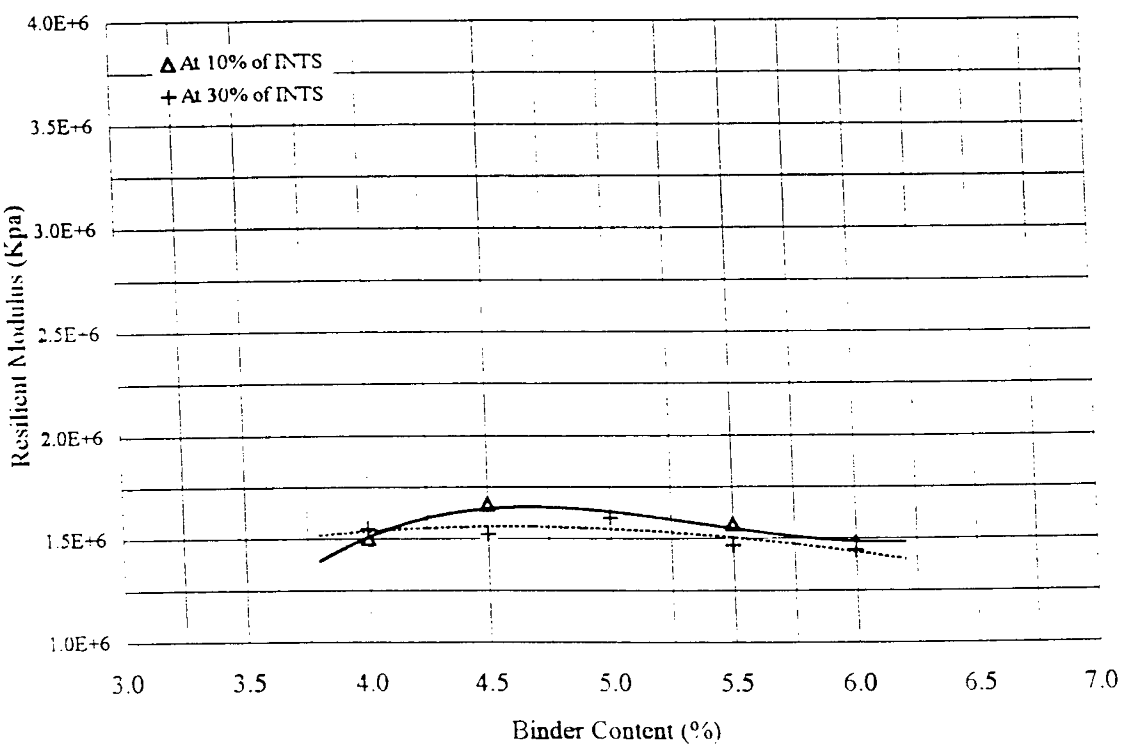


b. Testing Temperature = 25 °C

Figure 8.11 Resilient Modulus versus Binder Content for the Asphalt-Rubber Mixtures (Wet Process)



a. Testing Temperature = 5 ^{o}C



b. Testing Temperature = 25 ^{o}C

Figure 8.12 Resilient Modulus versus Binder Content for the Rubber-Filled Mixtures (PlusRide II)

c. Rutting

c.1 Based on the rutting prediction model, determine the minimum mix stiffness (S_{mix}), after the same continuation time of the laboratory creep test, necessary to prevent the identified rutting severity level (low, medium and high, Table 8.9) from Figure 8.9 based on the specific pavement thickness.

c.2 Using S_{mix} from step (c.1), and the mixture laboratory testing results, determine the binder content range (R_r), Figure 8.13, for either the wet process or the PlusRide II mixes.

c.3 Mix stiffness interpolation may be used for other in-service pavement temperature.

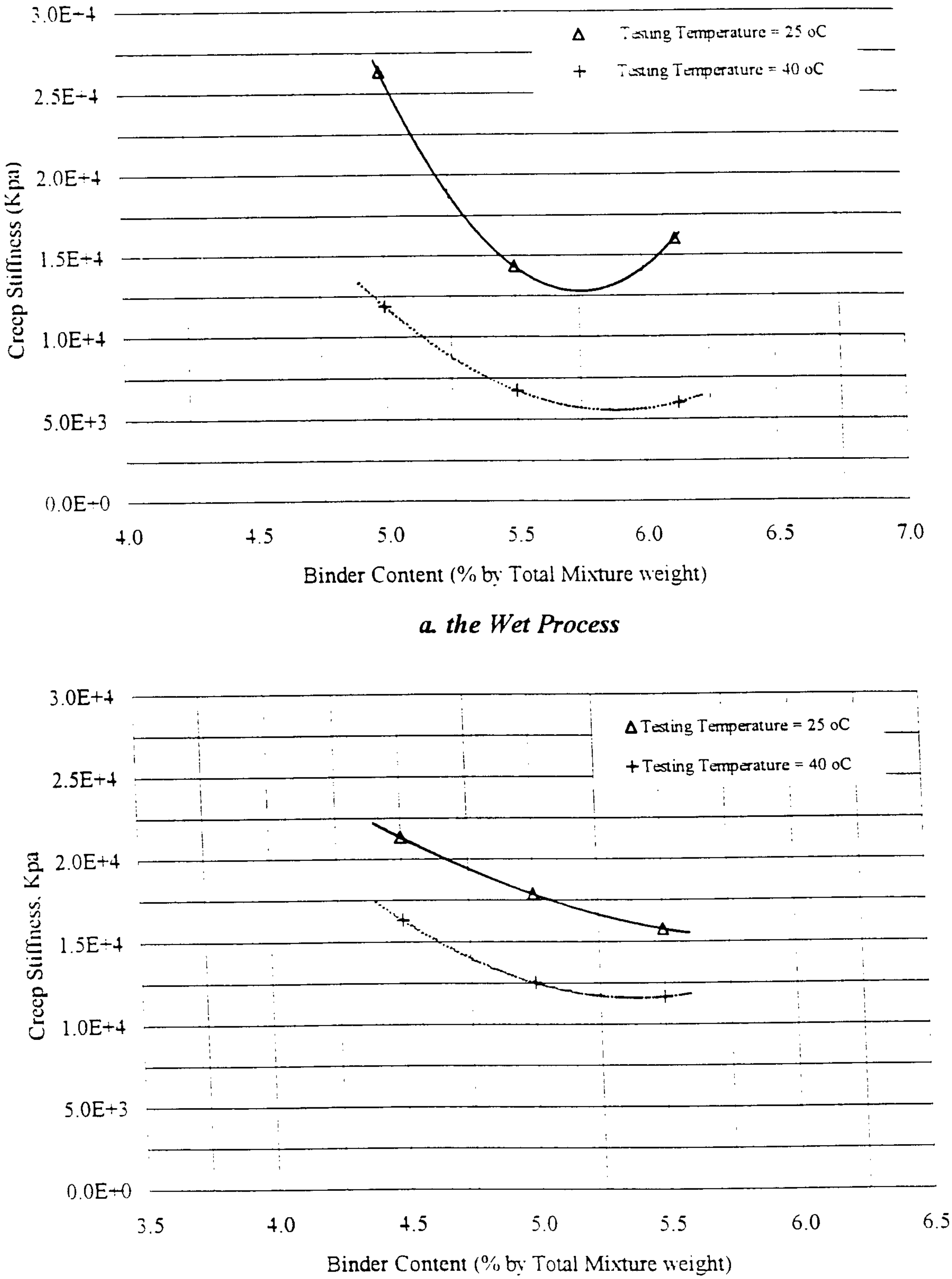
d. Low-Temperature Cracking

Although no laboratory results are available in this study to cover this mode of distress, the steps are given as guidelines for future studies.

d.1 Based on the climatic region within which the pavement is placed, the low temperature boundary limits need to be identified.

d.2 For different binder contents, determine the tensile strength using the indirect tensile test at slow rates and variable temperatures. Construct the relationships between binder content and mixture indirect tensile strength at different temperatures.

d.3 At the corresponding low temperature limits, step (d.1) define the binder content range (R_t), using the relationships established in step (d.2).



a. the Wet Process

b. the PlusRide II

Figure 8.13 Creep Stiffness versus Binder Content

e. Moisture Damage

As explained earlier, moisture damage is another significant input in asphalt mix design. This analysis involved immersing asphalt mixture samples (prepared according to the standard Marshall method, ASTM D 1559, of mix design at 75 blows per face) in water at two different temperatures, 25 °C, 60 °C (77 °F and 140 °F), and over a period of 14 days. Mr testing may be used for asphalt mixtures moisture damage evaluation. For the 14-day period, three samples were subjected to Marshall test after 0, 1, 4, 7, and 14 days. Stability, flow, and stiffness were evaluated and shown in Tables 8.11 and 8.12 for the conventional and the wet process asphalt-rubber mixtures. Durability indices were calculated and shown in Tables 8.13 and 8.14 for the conventional and the wet process mixtures, respectively. The incorporation of moisture damage in asphalt mix design is summarized in the following steps:

e.1 Set the criteria for the allowable durability index based on the environmental conditions in the region where the pavement will be placed (for example 60% loss in stability or Mr with moisture exposure to specific days period).

e.2 From the laboratory results, Figures 8.14 and 8.15, define the binder content range (R_m) using the allowable level of durability index, step (e.1).

e.3 Interpolation may be used for other in-service pavement temperatures.

OPTIMUM BINDER CONTENT

The optimum binder content can be determined by the range method using each distress as the vertical axis and percent binder content as the horizontal axis. Since asphalt mixture design is a "balance" of the designed mixture properties for assuring specific performance requirements, a vertical line may be drawn at the point where the binder content provides the acceptable lower and upper limits for the different types of distresses. The mid point of the common overlap may be selected as the optimum binder content, Figure 8.16.

Table 8.11 Moisture Effect on the Conventional Asphalt Mixtures

a. *Immersion Temperature = 25 °C*

Binder Content	5.00%			4.50%		
Immersion Time (Day)	Stability (N)	Flow (0.25 mm)	Stiffness (N/mm)	Stability (N)	Flow (0.25 mm)	Stiffness (N/mm)
0	41620	23.94	5926	48165	21.34	6439
1	40807	26.23	5870	43478	24.38	5847
4	40014	26.11	5614	42311	25.07	5784
7	39705	28.66	5409	41481	27.04	5572
14	39398	29.54	5147	41012	27.82	5376

b. *Immersion Temperature = 60 °C*

Binder Content	5.00%			4.50%		
Immersion Time (Day)	Stability (N)	Flow (0.25 mm)	Stiffness (N/mm)	Stability (N)	Flow (0.25 mm)	Stiffness (N/mm)
0	14931	15.28	6918	15090	14.22	6944
1	11432	19.91	1918	11266	17.55	2352
4	11074	25.77	1469	10375	19.21	1946
7	10849	27.77	1282	9637	21.67	1427
14	9868	28.04	1064	8772	22.87	1216

Table 8.12 Moisture Effect on the Asphalt-Rubber Mixtures (Wet Process)

a. Immersion Temperature = 25 °C

Binder Content	6.13%			5.00%		
Immersion Time (Day)	Stability (N)	Flow (0.25 mm)	Stiffness (N/mm)	Stability (N)	Flow (0.25 mm)	Stiffness (N/mm)
0	31624	25.24	3948	26643	19.75	3641
1	31301	27.23	3893	25415	19.82	3618
4	30205	28.55	3768	24812	20.04	3575
7	30114	28.94	3603	23741	22.14	3346
14	30049	29.46	3646	23008	22.56	3227

b. Immersion Temperature = 60 °C

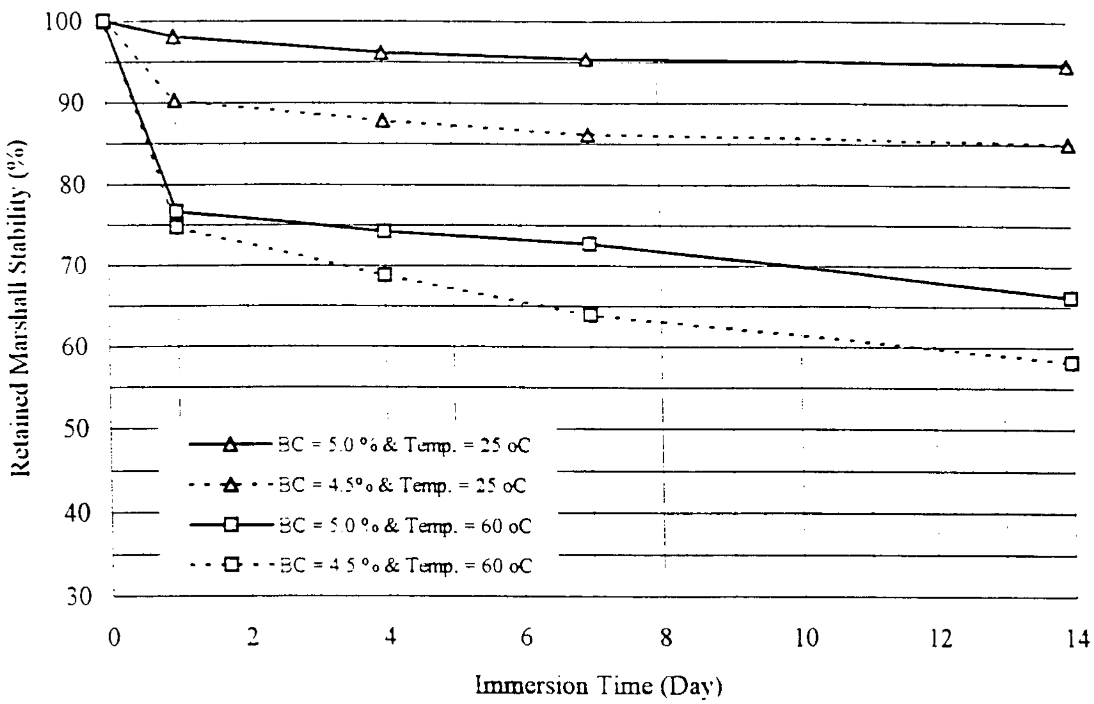
Binder Content	6.13%			5.00%		
Immersion Time (Day)	Stability (N)	Flow (0.25 mm)	Stiffness (N/mm)	Stability (N)	Flow (0.25 mm)	Stiffness (N/mm)
0	9984	19.78	2059	10129	13.41	2769
1	8950	16.66	1835	7321	19.91	1208
4	7983	23.98	1226	6738	22.08	1057
7	7648	24.99	1049	6095	23.11	988
14	6750	26.58	973	5604	24.38	903

Table 8.13 Durability Indices for the Conventional Asphalt Mixtures

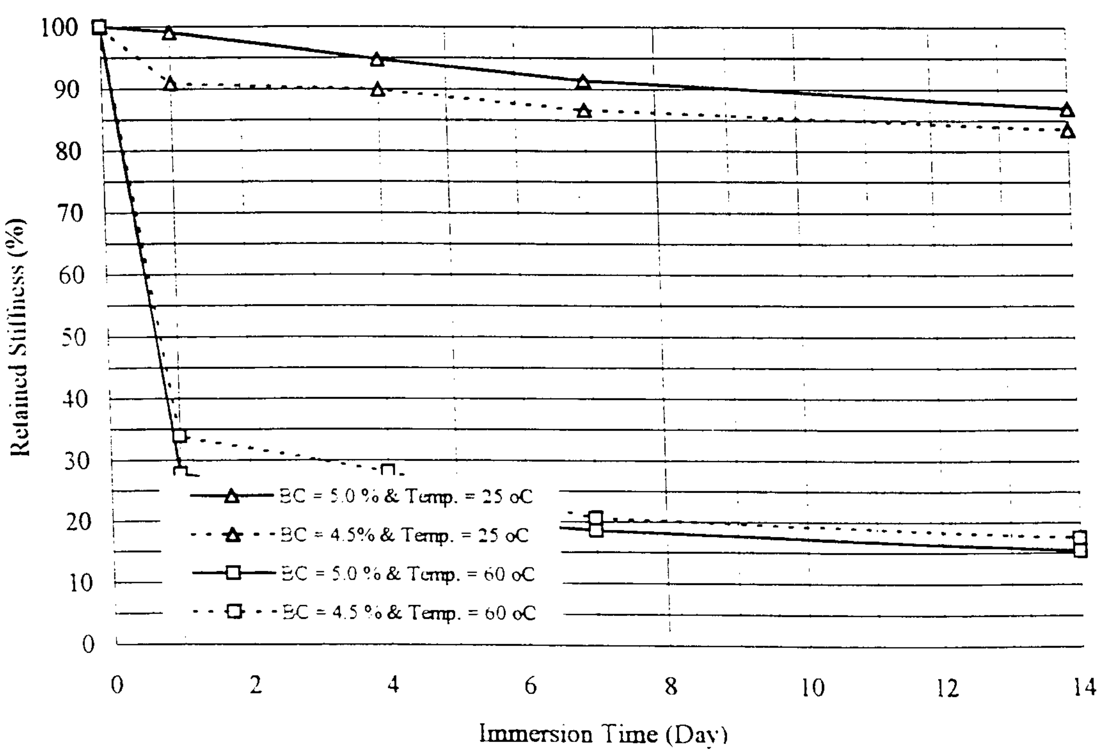
Temperature	25 °C				60 °C			
	Stability (N)		Stiffness (N/mm)		Stability (N)		Stiffness (N/mm)	
Binder Content	5.00	4.50	5.00	4.50	5.00	4.50	5.00	4.50
Immersion Time (Day)								
0	100	100	100	100	100	100	100	100
1	98	90	99	91	77	75	28	34
4	96	88	95	90	74	69	21	28
7	95	86	91	87	73	64	19	21
14	95	85	87	83	66	58	15	18

Table 8.14 Durability Indices for the Asphalt-Rubber Mixtures (Wet Process)

Temperature	25 °C				60 °C			
	Stability (N)		Stiffness (N/mm)		Stability (N)		Stiffness (N/mm)	
Binder Content	6.13	5.00	6.13	5.00	6.13	5.00	6.13	5.00
Immersion Time (Day)								
0	100	100	100	100	100	100	100	100
1	99	95	99	99	90	72	89	44
4	96	93	95	98	80	67	60	38
7	95	89	91	92	77	60	51	36
14	95	86	92	89	68	55	47	33

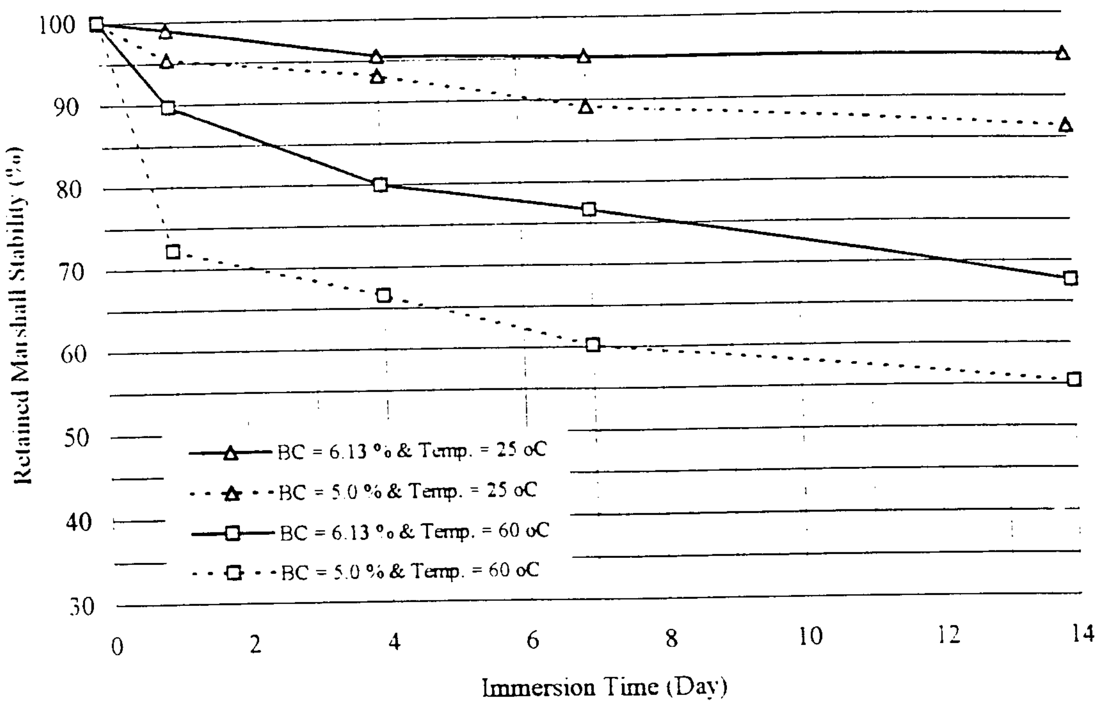


a. Retained Stability

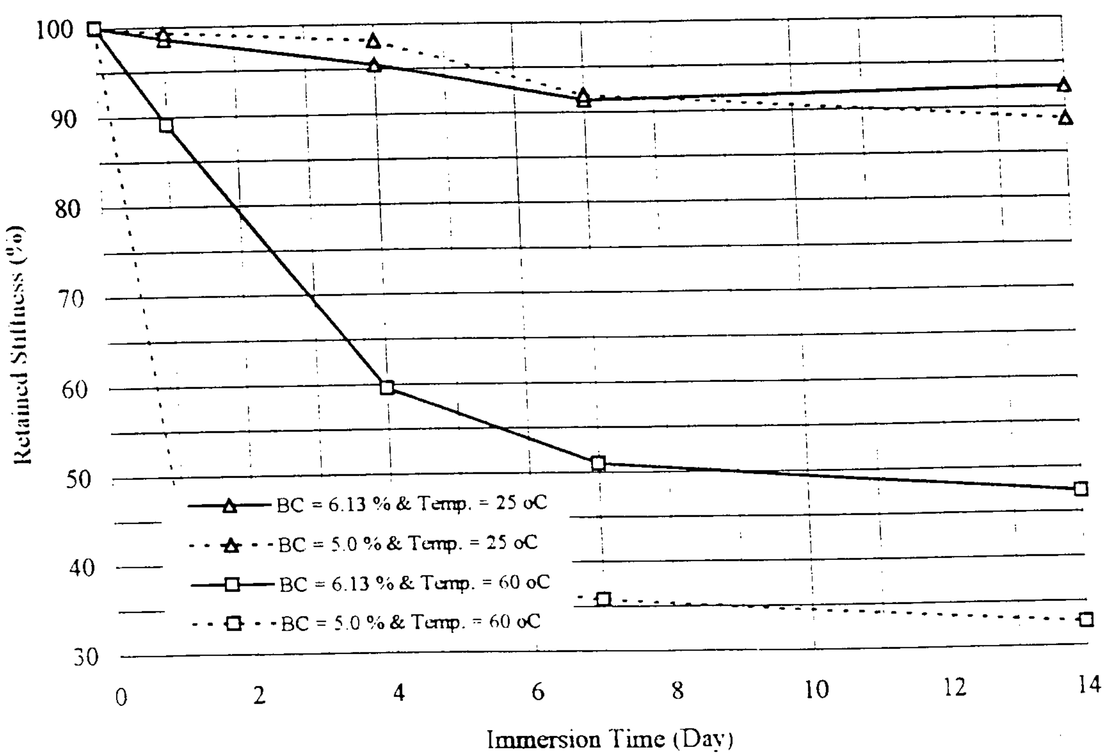


b. Retained Stiffness

Figure 8.14 Immersion Time Effects on the Conventional Asphalt Mixture Properties



a. Retained Stability



b. Retained Stiffness

Figure 8.15 Immersion Time Effects on the Asphalt-Rubber mixture Properties (Wet Process)

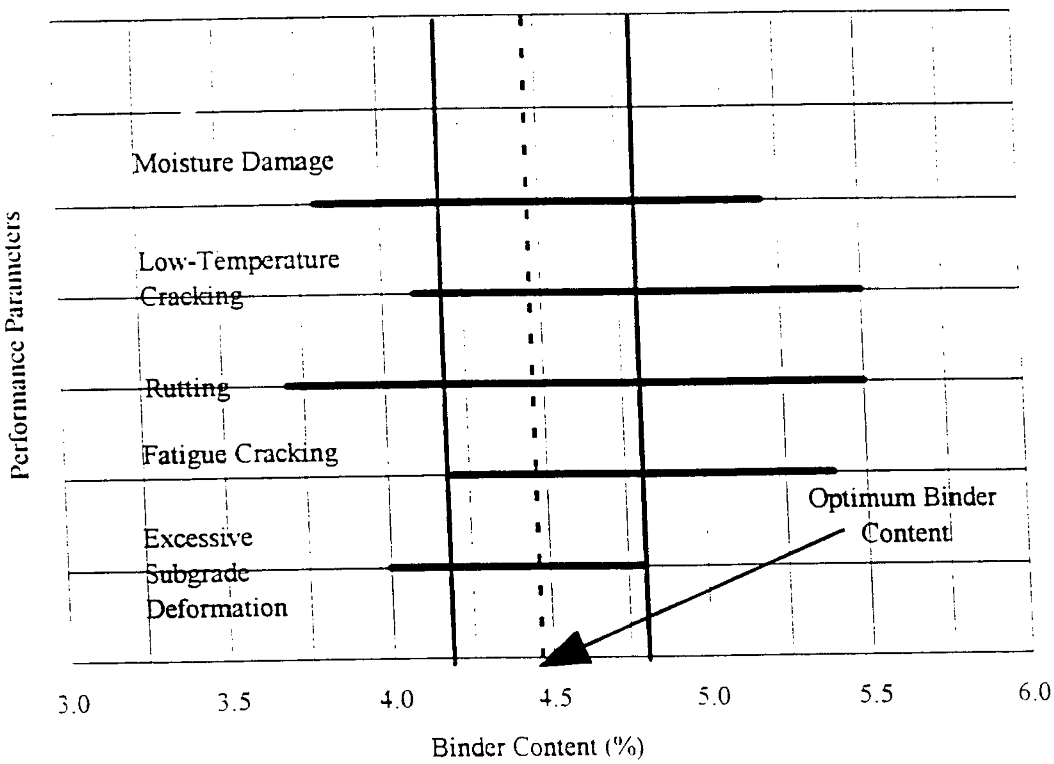


Figure 8.16 Schematic Drawing for Optimum Binder Content Selection

CHAPTER 9. CONCLUSIONS AND RECOMMENDATIONS

The effort of this study is to develop an asphalt mixture design technique that integrates the structural performance of the pavement. The philosophy of this design procedure is to design a hot mix asphalt mixture based on the expected structural performance of the pavement considering different distresses such as subgrade deformation, fatigue, rutting, low-temperature cracking, and moisture damage. Based on the comprehensive literature review, the laboratory testing program results, and the analytical evaluation, the following conclusions can be drawn.

CONCLUSIONS

- The use of rubber in asphalt pavement materials is being investigated since the 1960's. Results of ongoing investigations identified improvements in material properties and performance.

- Based on the mixtures used in this study, The kinematic viscosity of non-aged rubber asphalt binder increases during the first 45 minutes of blending. An equivalent effect was observed on the penetration results with the standard needle or cone The effects of aging indicated that a higher penetration and lower viscosity than non-aged samples, was obtained for the same blending time. Shorter blending time, providing a partially reacted rubber and asphalt, might be appropriate so that aging effects during production and laying operations provides the desired binder properties and stiffness.

- The Marshall optimum binder content was found to be 5.03, 6.13, and 5.0 percent by the total weight of the mixture, for the conventional, the wet process, and the dry-process asphalt rubber mixtures, respectively.

- The conventional mixture has about 1.6 times higher stiffness than the asphalt rubber mixtures indicating that modified mixtures may have a higher flexibility and consequently may provide higher resistance to fatigue and low temperature cracking than the conventional.

- The modified mixtures (wet process and PlusRide II) have higher toughness than the conventional mixture. Thus it is expected that asphalt rubber mixtures might be able to absorb higher energy before failure than the conventional one. The maximum toughness of the conventional mixture was found at an asphalt content of 5.1%, while for asphalt-rubber mixtures, wet process and PlusRide II were observed at a binder content of 6.1% and 5.22%, respectively, and close to the optimum asphalt contents evaluated using the standard Marshall method of mix design.

- The rate of absorbed energy increases with increasing the binder content up to certain limit after which the rate of absorbed energy decreases. At Marshall optimum binder contents, the rate of absorbed energy of asphalt rubber mixtures is higher than that of the conventional, indicating that asphalt rubber mixtures have the ability to resist higher loading rates and longer loading duration due to the increased mixture flexibility.

- The results from the Marshall mix design method could be contemplated by considering toughness and stiffness of the mixtures. The Marshall optimum binder contents, 5.03%, 6.13%, and 5.00% for the conventional, and the asphalt rubber mixtures; wet process and PlusRide II, respectively, are very close to the binder content corresponding to the maximum toughness, (5.08%, 6.13%, and 5.10%, respectively). However, the binder contents corresponding to the maximum stiffness were found to be 0.23%, 0.40%, and 0.18% lower than the Marshall optimum binder content for these mixtures.

- The maximum values for the indirect tensile strength for the wet process and PlusRide II asphalt-rubber mixtures were obtained at about 6.09 % and 5.03% binder contents, respectively. The static resilient modulus of PlusRide II is lower than that of the wet process due to the larger size crumb rubber aggregate.

- At the low level of testing temperature, 5 °C, load magnitude had no effect. The maximum values of resilient modulus are within the range of 5.5% to 6.0% and 4.2% to 5.0% binder content for the wet process and the PlusRide II.

- The rate of axial strain increases with increasing binder content. Compared with the wet process, PlusRide II asphalt-rubber mixtures have higher axial compressive strain under loading. This is probably associated with the aggregate gradation (gap-graded for PlusRide II against dense-graded for wet process) and the presence of the coarse rubber aggregate. These coarse rubber particles make the PlusRide II more flexible.

- Fatigue life was evaluated at 25 °C since the effect of binder content on fatigue is more pronounced at higher temperatures. Increasing the binder content has improved the fatigue life to a certain limit (optimum) after which fatigue life is reduced. The binder contents for the maximum fatigue life are 5.6% and 5.2% for the wet process and the PlusRide II, respectively.

- Vertical compressive strain at the top of the subgrade; horizontal tensile strain at the bottom of the asphalt layer; and vertical compressive stress at the mid-depth of the asphalt layer were investigated under three different traffic levels.

- Subgrade excessive deformation, fatigue cracking, rutting, thermal cracking, and moisture damage are the most critical pavement distresses. Sound and reliable models for pavement performance prediction were used for defining the performance-based asphalt mixture design methodology.

- The optimum binder content of the performance-based methodology proposed herein can be determined by the range method using each distress at the vertical axis and percent binder content at the horizontal axis.

RECOMMENDATIONS

- Additional investigation, using the SHRP testing methods, and with asphalt-rubber binders using different rubber gradations, rubber contents, extender oils, and blending conditions will fine-tune the criteria of asphalt-rubber binder preparation, storage, and usage.

- Testing different materials (aggregates, asphalt cements, crumb rubbers, extender oils) with variable asphalt mixes (dense and open graded with surface and binder course mixes) will assist in validating performance models.

- Additional investigation with stiffness and toughness for enhancement of the standard Marshall method of mix design is suggested.

- Different testing conditions (testing temperature and load amplitude, frequency, and duration) for the performance-related tests (resilient modulus, fatigue, creep, low-temperature cracking, and moisture damage) will provide more accurate performance models.

- Implementation of this design technique and field performance evaluation of constructed test sections will add in the fine-tuning of the mix design method.

LIST OF REFERENCES

1. "A Mixture Design Procedure Based on The Failure Envelope Concept" Proceedings., Association of Asphalt Paving Technologists, Vol. 52, 1983.
2. "A Simple Laboratory Test to Indicate the Susceptibility of Asphalt-Aggregate Mixtures to Moisture Damage During Repeated Freeze-Thaw Cycling" H. G. Plancher and R. L. Miydkc, Proceedings of Canadian Technical Asphalt Association, 1980.
3. "A Study of The Use of Recycled paving material" FHWA-RD-93-147.
4. "AASHTO Guideline for Design of Pavement Strucrures-1986" American Association of State Highway and Transportation Officials, Washington D. C. 1986.
5. "Advances in Technology of Asphalt Paving Materials Containing Used Tire Rubber" H. B. Takallou and A. Sainton, Transportation research Record, Vol. 1339, pp. 23-29, 1992.
6. "An Improved Asphalt Concrete Mix Design Procedure" Mahboub K. and Little D. N., jr., Asphalt Paving Technology 1990
7. "Asphalt Concrete Mix Design Development of More Rational Approaches" W. Gartner, Special Technical Report, STR 1041, ASTM, 1987.
8. "Asphalt Mix Design and The Indirect Tensile Test: A New Horizon" G. Y. Baladi and R. S. Harichandran, ASTM, STP 1041, 1989.
9. "Asphalt Mix Design: An Innovative Approach" G.Y. Baladi, R. W. Lyles, and R. S. Harichandran, Transportation Research Record, Vol. 1171,pp160-167,1988.
10. "Asphalt Mixtures: Comparative Analysis of Characterization for Design" Randy B. Machemehl and Thomas W. Kennedy, Transportation research Record, Vol. 821, pp. 22-29, 1983.
11. "Asphalt Pavement Modified with Coarse Rubber Particles Design, Construction, and Ice Control" D. C. Esch, Federal Highway Administration, State of Alaska Department of Transportation, Report # AK-RD-85-07, August 1984.
12. "Asphalt Rubber Binder Laboratory Performance" T. S. Shuler, C. K. Adams, and M. J. Lamborn, Texas Transportation Institute, Texas A & M University, College Station, Report # 347-1, March 1985.

13. "Asphalt-Aggregate Mixture Analysis System: AAMAS" H. L. Von Quintus, J. A. Scherocman, C. S. Hughes, and T. W. Kennedy, Federal Highway Administration & American Association of State Highway and Transportation Officials, Report # PB91-171397 (NCHRP Report 338), March 1991.

14. "Asphalt-Rubber Binder Laboratory Performance" Texas Transportation Institute, State Department of Highway and Public Transportation, Feb 25, 1986.

15. "Characterization of Asphaltic Mixtures for Prediction of Pavement Performance" Khosla, N. P. and Omer, M. S. , Transportation Research Record, Vol. 1034, 1985, pp. 47-55.

16. "Comparison of Mix Design Methods for Rubberized Asphalt Concrete Mixtures" M. Stroup-Gardiner, N. Kruptz, and J. Epps, National Seminar on Asphalt Rubber, Kansas City, Mo., October 1989.

17. "Design and construction of Asphalt Material With Crumb Rubber Modifier" Michael Heitzamn, Transportation Research Record, Vol. 1339, pp1-8,1992.

18. "Design and Construction of Asphalt Paving Material with Crumb Rubber Modifier" State of the art, report No. Federal Highway Administration (FHWA)-SA-92-022, May 1992.

19. "Design Methods for Hot-Mixed Asphalt-Rubber Concrete Paving Materials" J. G. Chehovits, Asphalt Rubber Producer Group,, National Seminar on Asphalt Rubber, Kansas City, MO., 1989.

20. "Design of Large Stone Asphalt Mixes to Minimize Rutting" Prithvi S. Kandhal, Transportation Research Record , Vol. 1259 ,pp153-162,1990.

21. " Development of a simplified Test Method to Predict Rutting Characteristics of Asphalt Mixes" J. S. Lai, Report # FHWA/GA/86-8503, Federal Highway Administration, July 1986.

22. "Development of Improved Mix and Construction Guidelines for Rubber- Modified Asphalt Pavement" H. B. Takallou and R. G. Hicks, Transportation research Record, Vol.1171, pp. 113-120, 1980.

23. "Development of New Procedure for Bituminous Mix Design" S. F. Brown, K. E. Kooper, J. N. Prestopn, and C. A. Pell, Evrobitume Symposium, Madrid, pp. 494-504, 1989.

24. "Development Work on a Test Procedure to Identify Water-Susceptibility on Asphalt Mixtures" J. A. Epps and J. W. Button, Texas A & M University, College Station, Report # 287-1, November 1980.

25. "Direct Tension Test Device" Report # SHRP B-004

26. "Dynamic Shear Rheometer" Report #SHRP-003

27. "Effect of Loading Magnitude on Measured Resilient Modulus of Asphaltic concrete Mixes" J. A. Almudaiheem and F. H. Al-Sugair, Transportation research Record, Vol. 1317, pp. 139-144, 1991.

28. "Effect of Mix Ingredients on The Behavior of Rubber-Modified Asphalt Mixtures" H. B. Takallou, R. G. Hicks, and D. C. Esch, Transportation research Record, Vol. 1096, pp. 68-80, 1987.

29. "Effect of Temperature and Mixture Variables on Fatigue Life Predicted by Diametrical Testing" Y. R. Kim, N. P. Khosla, and N. Kim, Transportation research Record, Vol. 1317, pp. 128-138, 1991.

30. "Effect of Test Parameters on Resilient Modulus of Laboratory Compacted Asphalt Concrete Specimens" R. L. Boudreau, R. G. Hicks, and A. M. Furber, Transportation research Record, Vol. 1353, pp. 46-528, 1992.

31. "Evaluation of Marshall and Hveem Mix Design Procedures for Local Use" H. Al-Abdul Wahhab, and Ziauddin A. Khan, Transportation research Record, Vol. 1317, pp. 68-76, 1991.

32. "Evaluation of Moisture Susceptibility of Asphalt Mixtures Using the Texas Freeze-Thaw Pedestal Test" T. W. Kennedy and F. L. Roberts, Association of Asphalt Paving Technologists, February 1982.

33. "Evaluation of Rutting Characteristics of Asphalt Mixes Using Loaded-Wheel Tester" J. S. Lai, research project 8609, final report, Georgia Department of Transportation (GDOT), Atlanta 1986.

34. "Evaluation of the Effect of Moisture Conditioning on Black Base Mixtures" J. N. Anagnos, Center of Transportation Research, University of Texas at Austin, Report # 183-13, October 1981.

35. "Experimental Construction of Rubber Modified Asphalt Mixtures for Asphalt Pavements in New York State'" J. F. Shook, New York State Department of Transportation, Albany, NY, May 1990.

36. "FAA Mixture Design Procedure for Asphalt Rubber Concrete" F. L. Roberts and R. L. Lytton, Transportation research Record, Vol. 1115, pp. 216-225, 1987.

37. "Fatigue and Dynamic Testing of Bituminous Mixtures" ASTM special technical publication 561, June 1973.

38. "Fatigue Behavior of Asphalt Rubber Hot Mix and Conventional Asphalt Concrete" Raad L., Sohaundjian S., and Briggs R., Transportation Research Center of Northern Engineering, Alaska University, Final Report, September 1992.

39. "Fatigue Characteristics of Alaskan Pavement Mixes" Y. R. Geotzee and B. G. Conner, Transportation research Record, Vol. 1269, pp. 168-175, 1990.

40. "Fatigue Characteristics of Bituminous Mixes" P. S. Pell, American Association of Pavement Technology, pp. 310-323,1962.

41. "Field Performance of Polymer-Modified Asphalt" Jones, Torshizi, Elmore, Kennedy, Hazelett, Transportation Research Board, Paper No.93 0099,Washington D.C. 72nd Annual Meeting, Jan 10-12,1993.

42. "Field Performance of Rubber Modified Asphalt Paving Materials" T.S. Shuler, R. D. Paulouich, and J. A. Epps, Transportation Research Record, Vol. 1034, pp. 96-102, 1985.

43. "Field Trials of Plastic- and Latex-Modified Asphalt Concrete" Krater, D. L. Wolfe, and J. A. Epps Transportation Research Record 1115, pp. 216-225, 1987.

44. "Fifteen-Year Pavement Condition History of Asphalt-Rubber Membranes in Phoenix, Arizona" R. H. Schnormeier, Transportation Research Record, Vol. 1096, pp. 62-67,1991.

45. "Florida's Approach Using Ground Tire Rubber in Asphalt Concrete Mixtures" Gale C. Page, Byron E. Ruth, and Randy C. West, Transportation Research Record, Vol. 1339, pp. 16-22, 1992.

46. "Grafting of Waste Rubber" G. Adam, A. Serbenik, U. Oseredkar, Z. Veksli, F. Rawogajec, American Association of Paving Technology, Vol. 63, pp. 660-668, 1984.

47. "Improved Asphalt Concrete Mix Design Procedure" Mahboub K. and Little D. N., Report No. FHWA/TX-87/474-1F, Federal Highway Administration, July 1988.

48. "Improved Asphalt Mix Design" C. L. Monismith, J. A. Epps, and F. N. Finn, American Association of Paving Technologies, Vol. 54, pp. 347-405, 1985.

49. "Investigation of Materials and Structural Properties of Asphalt-Rubber Paving Mixtures" FHWA-RD-86-027, Vol. 1,Vol 2

50. "Investigation of Materials and Structural Properties of Asphalt-Rubber Paving Mixtures" . Federal Highway Administration. FHWA-RD-86-027, Vol. 1and 2, 1986.

51. "Investigation of Rutting Potential Using Static Creep Testing on Polymer Modified Asphalt Concrete Mixtures" Neil C. Krutz, Raj Siddharthan, and Mary Stroup-Gardiner, Transportation Research Record, Vol. 1317, pp. 100-108, 1991.

52. "J-Integral and Cyclic Plasticity Approach to Fatigue and Fracture of Asphaltic Mixtures" A. A. Abdulshafi and Kamran Majidzadeh, Transportation Research Record, Vol. 1034, pp112-123, 1985.

53. "Laboratory and Field Study of Pavement Rutting in Saudi Arabia" B. A. Anani and F. A. Balghunaim, Transportation research Board, Vol. 1259, pp. 79-90, 1990.

54. "Laboratory Evaluation of An Asphalt-Rubber Sal" R. A. Jimenez and W. R .Meier, Jr., Transportation Research Record , Vol. 1034, pp86-95, 1985.

55. "Laboratory Testing for Use in The Prediction of Rutting in Asphalt Pavement" S. F. Brown, Transportation Research Board, Vol. 610, pp22-27, 1976.

56. "Maryland Crumb Rubber Modified Asphalt Concrete Demonstration Projects" M. W. Witczad, H. A. Smith, Presented at 4r Conference and Road Show, Philadelphia, Pennsylvania, Dec 5,1993, Session 5,Asphalt Rubber.

57. "Material Characterization and Inherent Variation Analysis of Asphaltic Field Cores" William O. Hadly, Transportation Research Record, Vol. 1317, pp109-121, 1991.

58. "Mixture Design Procedure for Coarse Matrix High Binder Asphaltic Concrete" Texas DOT, Division of materials and Tests.

59. "Mixture Design Procedure for Crumb Rubber Modified Asphaltic Concrete" Texas DOT, Division of Material and Tests.

60. "Modification of Paving Asphalts By Digestion With Scrap Rubber" Join W. H. Oliver, Transportation Research Record, Vol. 821, pp.37-44, 1981.

61. "Moisture Effects on Asphalt Concrete Mixtures" R. B. McGennis and R. B. Machemehi, Center of Transportation Research, University of Texas at Austin, Report # 253-2.

62. "New Relationships Between Structural Properties and Asphalt Mix Parameters" Baladi, G. Y., Harichandran, R. S. and Lyles. R. W., Transportation Research Record, Vol. 1171, 1988, pp. 168-198.

63. "Occupational Exposure Assessment of Asphalt. Ground Tire Rubber, Blending, Mixing, and Paving Operation" Vol. 1.Prepared By Westinghouse Remediation Services Inc., and Florida DOT, Radian Corporation, May 28,1993.

64. "Overlay Rehabilitation Technologies Using Asphalt Rubber Surface Courses" JOE O. Cano, 4R Conference, Road Show, Philadelphia, Pennsylvania, Dec 5-7,1993.

65. "Overview of A Rational Asphalt Concrete Mixture Design for Texas" Kamyar Mahboub and Dallas N. Little, Transportation Research Record, Vol. 1171, pp149-159, 1988.

66. " Pavement Performance and Asphalt Concrete Mix Design" Proceedings., Association of Asphalt Paving Technologists, Vol. 52, 1983.

67. "Performance of Binder Modifiers in Recycled Asphalt Pavement 1987-1992" M. Farrar, R. Harvey, K. Ksaibati. B. Ramansundaram, Transportation Research Board, Paper No 930277 Washington D.C. 72 ST Annual Meeting ,Jan 10-12,1993.

68. "Permanent Deformation Characterization of Recycled Tire Rubber Modified and Unmodified Asphalt Concrete Mixtures" Neil C. Krutz and Mary Stroup-Gardiner, Transportation research Record, Vol. 1339, pp. 38-44, 1992.

69. "Prediction of Rutting in Virgin and Recycled Asphalt Mixtures for Pavement Using Trixial Tests" A. Wijeratne and M. Sargious, American Association of Paving Technologies, Vol. 54, PP 111-129, 1984.

70. "Predicting Permanent Deformation of Asphalt Concrete From Creep Tests" James S. Lai and William L. Hufferd, Transportation Research Board, Vol. 610, pp41-43, 1967.

71. "Predicting Moisture-Induced Damage to Asphalt Concrete" NCHRP, Report # 192, 1978.

72. "Prediction of Rutting and Development of Test Procedures for Permanent Deformation in Asphalt Pavements" W. J. Kenis and M. G. Sharma, Federal Highway Administration and US Department of Transportation, Report # RD-76-146, 1976.

73. "Procedure for Prediction Rut Depths in Flexible Pavement" Frank Meyer, and Ralph Haas, Transportation Research Board, Vol. 816, pp38-40,1988.

74. "Relationship Between Permanent Deformation of Asphalt Concrete and Moisture Sensitivity" Transportation Research Record, Vol. 1259, pp. 169-177,1990.

75. "Remaining Fatigue Life Analysis: A Comparison Between Conventional Asphalt Concrete and Asphalt Rubber Hot Mix" L. Raad, S. Saboundjian, and J. Corcoran, Transportation Research Board, paper no. 931059, 72 nd annual meeting, Washington D. C., Jan 10-12, 1993.

76. "Rut Depth Prediction" C. L. Saraf, W. S. Smith, and F. N. Finn, Transportation Research Board, Vol. 816, pp. 9-14, 1988.

77. "Rut Depth Prediction and Test Procedures For Permanent Deformation in Asphalt Pavement" W. J. Kenis and M. G. Sharma, Transportation Research Board, Vol. 816, pp. 28-30,1988.

78. "Rut-Resistant Asphalt Concrete Overlay in Wisconsin" Ashwani K. Sharma and Lynn Lynn L. Larson, Transportation Research Board, Vol. 816, pp. 163-168, 1988.

79. "Rutting Prediction in Asphalt Concrete Pavement" C. L. Monismith, Transportation Research Board, Vol. 816, pp. 2-8, 1988.

80. "Some Effect of Rubber Additives on Asphalt Mixes" R. J. Salter and J. Mat, Transportation research Record, Vol. 1269, pp. 79-86, 1990.

81. "Static Creep Test" Texas DOT, Division of Materials and Tests

82. "Structure Design of Flexible Pavement: Simple Predictive System" Jacob Uzan and Ropert L. Lytton, Transportation Research Record, Vol. 888, pp. 56-63, 1984.

83. "Study on Mix Design Criteria for Controlling The Effect of Increased Tire Pressure on Asphalt Pavement" O. K. Kim, C. A. Bell, J. E. Wilson, and G. Boyle, Transportation research Record, Vol. 1171, pp. 149-159, 1988.

84. "Testing Methods for Asphalt-Rubber" R. A. Jamenez, Arizona Department of Transportation, Report # ADOT-RS-15(164), 1978.

85. "The Correlation Between Rutting and Creep Tests in Asphalt Mixes" J. F. Hills, D. Brian, and P. J. Van de Loo, Institute of Petroleum, London, Report # IP-74-001, 1974.

86. "The Creep Test: A Key Tool in Asphalt Mix Design and in The Prediction of Pavement Rutting" P. J. Van De Loo, American Association of Paving Technologies, Vol. 47, pp. 522-557,1978.

87. "The Pressuremeter Test for Highway Application" O. J. Svec and R. Veizer, FHWA-IP-89-008

88. "The Road to Less Waste, Recycling N. Y. State's Scrap Tires Into Asphalt Paving Material" NYS Legislature Commission on Solid Waste Management, Jan.1991

89. "Thickness Design -Asphalt Pavements for Highways and Streets" Manual Series No. 1, The Asphalt Institute, Sept. 1981.

90. "Use of Asphalt Rubber" Journal of Transportation Engineering, Vol. 114 No.3, May 1988.

91. "Use of Loaded-Wheel Testing Machine to Evaluate Rutting of Asphalt Mixes" J. s. Lai and Thay Minglee, Transportation research Record, Vol. 1269, pp. 116-124, 1990.

92. "Use of Wastes and Byproducts as Pavement Construction Materials" J. Emery, M. Pervez, D. Vanderveer, R. Pichette, 45th Canadian Geotechnical Conference, Toronto, Ontario, Oct 1992.

93. "Use, Availability, and Cost-Effectiveness of Asphalt Rubber in Texas" Cindy K. Estakhri, Joe W. Button, and Emmanuel G. Fernando, Transportation Research Record, Vol. 1339,pp 30-37,1992.

94. "Utep Asphalt Research Proposal to Texas DOT" October 1993

95. "Variables Affecting Marshall Test Results" Z. Siddiqui, M. Trethewey, and D. Anderson, Transportation Research Record, Vol. 1171, pp. 139-148, 1988.

96. "Verginia's Experimentation with Asphalt Rubber Concrete" G. Maupin Jr.,, Transportation research Record, Vol. 1339, pp. 9-15, 1992.