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16. Abstract

The objective of this study was to optimize the performance of crumb rubber modified (CRM) asphalt-concrete pavements through the development of materials and construction specifications, mixture design and testing procedures, binder testing procedures, and quality control and construction guidelines. These objectives were accomplished through an extensive laboratory investigation and somewhat limited field investigation.

This report presents guidelines, draft material specifications, and test protocol regarding the use of crumb rubber modifier in asphalt concrete pavements. Researchers found that CRM has the potential to improve the fatigue and thermal cracking performance of asphalt concrete pavements. Performance of crumb rubber modified asphalt concrete pavements is predicted using the Texas Flexible Pavement System (TFPS). Pavements were analyzed under a variety of climatic and structural conditions.

Although state DOTs must comply with the existing legislative requirements, tire rubber, as any additive, should be used whenever possible to address a given mixture deficiency or expected deficiency in a given situation.

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SHORT-TERM GUIDELINES TO IMPROVE CRUMB RUBBER MODIFIED ASPHALT CONCRETE PAVEMENTS

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IMPLEMENTATION STATEMENT

The goal of this study is to provide short-term guidelines to improve the performance of hot mix asphalt pavements which have been modified with crumb rubber. This report documents partial completion of this goal. Mixture design procedures, test procedures, and material properties of CRM binders and mixtures have been evaluated.

The findings of this study indicate that crumb rubber can be incorporated into hot-mix asphalt concrete without having a detrimental effect on pavement performance (when the mixture is designed and placed properly). The findings also indicate that crumb-rubber modified binders may be designed to produce asphalt mixtures that inhibit cracking.

Implementation of these research results will aid the Texas Department of Transportation, as well as other state DOTs, in meeting the requirements of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA). ISTEA provides for a minimum utilization requirement for asphalt pavement containing crumb rubber modifier as a percentage of the total tons of asphalt laid in such state. Guidelines, materials specifications and test protocol are provided to aid in implementation.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Texas Department of Transportation (TxDOT) or the Federal Highway Administration (FHWA). This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes.

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SUMMARY

One of two methods, wet or dry, are most commonly used to incorporate crumb rubber into asphalt paving mixtures. The wet process defines any method that adds the CRM to the asphalt cement prior to incorporating the binder in the asphalt paving project. The dry process defines any method of adding the CRM directly into the hot mix asphalt mixture process, typically pre-blending the CRM with the heated aggregate prior to charging the mix with asphalt. This study includes both of these methods. Two CRM sources were used in the study: -#80 mesh rubber (Rouse Rubber of Vicksburg, Mississippi) and -#10 mesh rubber (Granular Products of Mexia, Texas).

In this report, data is presented regarding the use of creep testing to predict rutting for crumb rubber modified asphalt concrete mixtures. Researchers found that at low stress levels, the damage induced in the sample is low as compared to high stress levels. Mixtures that perform well at low stress levels do not necessarily perform well at high stress levels.

A fatigue evaluation of CRM laboratory mixtures revealed that CRM has the potential to improve the fatigue performance of asphalt concrete pavements. CRM also has the ability to improve resistance to thermal cracking.

The Texas Flexible Pavements System (TFPS) was used to evaluate the performance of CRM mixtures. Data indicate that caution should be exercised when using CRM mixtures over asphalt-treated bases, particularly in hot-wet climates. Structures with granular bases (only) will yield better performance with stiffer mixtures.

An extensive evaluation of the dynamic shear rheometer was performed and was found to be an acceptable method for analyzing crumb rubber binders. A test protocol was developed to determine the viscosity using the dynamic shear rheometer.

1

Introduction

The objective of this study was to optimize the performance of crumb rubber modified (CRM) asphalt-concrete pavements through the development of materials and construction specifications, mixture design and testing procedures, binder testing procedures, and quality control and construction guidelines. This objective was accomplished through an extensive laboratory investigation and somewhat limited field investigation. Much of this work has been previously documented in research report 1332-1 (Estakhri et al. 1993). A brief discussion of the research documented in report 1332-1 follows.

CRM binders were fabricated in the laboratory and evaluated according to ten different binder tests. Six CRM binders and one control asphalt cement binder were characterized in the laboratory, and test procedures were also evaluated. Two sources of CRM and three CRM concentrations were used to fabricate the six blends. It was determined that some of the test procedures routinely used for CRM binders seem to have no apparent relationship to mixture properties or field performance. The SHRP bending beam rheometer and direct tension tests were also used to characterize CRM binders. The SHRP direct tension test and, to a lesser degree, the force-ductility test appeared to measure CRM binder characteristics which may be attributed to improved cracking performance in CRM mixtures.

Nine CRM mixtures were evaluated using the Asphalt Aggregate Mixture Analysis System (AAMAS) characterization procedures: four wet-process mixtures, four dry-process mixtures, and one control mix. Six of the nine mixtures were designed according to TxDOT's recently developed mixture design procedure for crumb rubber mixtures. These mixtures may be classified as coarse-matrix, high binder and are similar in gradation to a stone-matrix type mixture. These six mixtures were designated as follows:

- 10%FW (*10% fine* rubber, by weight of asphalt, via *wet* process),
- 10%CW (*10% coarse* rubber, by weight of asphalt, via *wet* process),
- 18%FW (*18% fine* rubber, by weight of asphalt, via *wet* process),
- 18%CW (*18% coarse* rubber, by weight of asphalt, via *wet* process),
- 18%FD (*18% fine* rubber, by weight of asphalt, via *dry* process), and
- 18%CD (*18% coarse* rubber, by weight of asphalt, via *dry* process).

The fine rubber is a -#80 sieve rubber from Rouse Rubber of Vicksburg, Mississippi. The coarse rubber is a -#10 sieve rubber from Granular Products of Mexia, Texas.

The remaining two crumb rubber mixtures were dense-graded and contained the maximum amount of crumb rubber that could be added while still conforming to standard mixture design criteria. The optimum amount of rubber which could be incorporated in these dense-graded mixtures was about 0.5 percent by weight of the aggregate. This would be equivalent to about 10 percent rubber by weight of the asphalt. These two mixtures are designated as follows:

- DGF (Dense graded with fine rubber)
- DGC (Dense graded with coarse rubber)

These mixture designations will be referred to often throughout this report to discuss additional laboratory tests and additional data analysis performed in the second year's research effort.

This report summarizes the research effort of the second year in this study. Also included in this report are draft specifications, test procedures, and guidelines. Appendix A contains a literature review.

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Mixture Performance Evaluation

Previous interim report 1332-1 (Estakhri et al. 1993) describes much of the performance evaluation of the CRM mixtures fabricated in the laboratory. However, additional laboratory testing and further analysis of the data is described in this chapter. The following is a discussion of the rutting, fatigue and thermal cracking analysis based on the laboratory tests performed in this study.

2.1 Rutting

Several test procedures were used to characterize the rutting potential of crumb rubber modified laboratory mixtures. These included unconfined compressive strength tests, resilient modulus, and static creep tests as recommended by AAMAS (Von Quintus et al. 1991). In addition, repeated load uniaxial creep tests were performed. These tests were performed on 10.2 cm (4 inch) high by 10.2 cm (4 inch) diameter samples which were molded to air void contents less than 3% to simulate traffic densification. The data for these laboratory tests are presented and discussed in previous report 1332-1. However, the following is a brief summary of this data.

According to AAMAS criteria, the creep moduli of all the mixtures tested were considered to be in a range of low to moderate rutting potential. Use of the uniaxial creep test to define stability and rut susceptibility of asphalt concrete mixtures has long been a popular approach because of its relative simplicity and because of the logical ties between the creep test and permanent deformation in asphalt concrete pavements. The major difficulty in developing criteria associated with the creep test by which to evaluate the rutting potential of asphalt concrete mixtures is in relating this criteria to field performance. This is true for all types of lab testing which must be correlated to field results. However, even without the benefit of correlations between lab creep tests and field results, it is evident that a stable and rut resistant mixture should not demonstrate tertiary creep if tested under stresses and at temperatures in the laboratory which simulate field conditions (Little and Youssef 1992). None of the mixtures tested in this study reached the tertiary creep region within the one-hour loading period (at 414 kPa or 60 psi stress level).

A log-log slope of the creep strain versus time of loading curve of less than 0.25 is indicative of a mixture which will not become unstable within the testing period of 3,600 seconds (Little and Youssef 1992). All of the mixtures tested in this study had a slope of less than 0.25. Please see report 1332-1 (Estakhri et al. 1993) for a full discussion of these data.

Tex 231-F Static Creep Test

TxDOT recently developed a static creep test which has been used for both crumb rubber mixtures and coarse-matrix, high-binder (CMHB) mixtures. This test is performed on a 10.2 cm (4 inch) diameter by 5.1 cm (2 inch) high sample at a stress level of 70 kPa (10 psi). The specimen is loaded for one hour with a 10 minute recovery period. The Materials and Tests Division of TxDOT has established preliminary criteria for acceptance of crumb rubber mixtures based on this laboratory test:

Creep Slope, maximum	3.5E-08 m/m/sec
Creep Stiffness, minimum	41.4 MPa (6000 psi)
Permanent Strain, maximum	0.0005 m/m.

TxDOT creep results for the laboratory mixtures are shown below in Table 1.

Table 1. Tex 231-F Static Creep Test Results for Laboratory Mixtures.

Mixture Type	Creep Slope, m/m/sec	Stiffness, MPa (psi)	Permanent Strain, m/m
Control	3.3E-08	52.3 (7582)	0.00032
Dense-Graded with Fine CRM (DGF)	6.8E-08*	29.8 (4320)*	0.00091*
Dense-Graded with Coarse CRM (DGC)	9.3E-08*	20.9 (3036)*	0.00088*
10% Fine CRM - Wet Method (10%FW)	4.4E-08*	50.6 (7339)	0.00043
10% Coarse CRM - Wet Method (10%CW)	3.1E-08	54.9 (7969)	0.00038
18% Fine CRM - Wet Method (18%FW)	4.3E-08*	62.2 (9019)	0.00049
18% Coarse CRM - Wet Method (18%CW)	10.9E-08*	34.7 (5032)*	0.00082*
18% Fine CRM - Dry Method (18%FD)	5.2E-08*	43.2 (6262)	0.00063*
18% Coarse CRM - Dry Method (18%CD)	14.2E-08*	27.3 (3959)*	0.00109*

* Fails TxDOT Criteria.

Criteria for creep test results are not well established. AAMAS probably provided the most well documented criteria as correlated to field performance at the time of this study. The AAMAS creep approach was derived from TxDOT-sponsored study 1170 (Mahboub and Little 1987). Based on the data presented in report 1332-1 and criteria as developed by AAMAS and Little and Youssef (1992), all of the mixtures appeared to be generally rut resistant; however, as shown in Table 1, most of these mixtures failed TxDOT criteria for the Tex-231-F static creep test. This may be an indication that the TxDOT criteria are more conservative. A further analysis of the TxDOT static creep test and data is given below. Comparisons are made here between the TxDOT static creep test data and the AAMAS static creep data; however, Table 2 summarizes some very important differences in the two test procedures.

Table 2. Differences Between TxDOT Creep and AAMAS Creep Tests.

Difference in Test Procedures	Tex 231-F TxDOT Static Creep	AAMAS Static Creep
Sample Size	5.1 cm (2 inch) high	10.2 cm (4 inch) high
Stress Level	70 kPa (10 psi)	414 kPa (60 psi)*
Recovery Period	10 minute	60 minute
Sample Compaction Method	Texas Gyratory	California* Kneading

* Applies to this study only.

For comparison purposes, creep modulus or stiffness is shown in Figure 1 for both the TxDOT and AAMAS static creep tests. Note for the Lufkin field mixture, the TxDOT creep test was modified and tested at a stress level of 414 kPa (60 psi) as well as the standard stress level of 70 kPa (10 psi). The mixtures tested according to AAMAS had a significantly higher stiffness than when tested according to TxDOT creep. Note, however, that the Lufkin mix which was tested according to TxDOT creep but at a higher stress level (414 kPa in lieu of 70 kPa) had a very high stiffness. These data indicate that the differences between the TxDOT and AAMAS creep tests are significant enough that the same acceptance criteria could not be applied to both tests.

According to Tex-231-F, the minimum creep modulus for accepting a mixture is 41.4 MPa (6000 psi). From this we can calculate the maximum allowable total strain after one hour of loading which is 0.00167. Using an elastic layered analysis, we can calculate the pavement response under a wheel load. Nine laboratory mixes were modeled in this pavement as a surface layer. An actual pavement section (FM 1709 near the Dallas-Fort Worth area) is considered for the analysis of stresses and strains in the surface layer using a computer program called "CHEVPC."

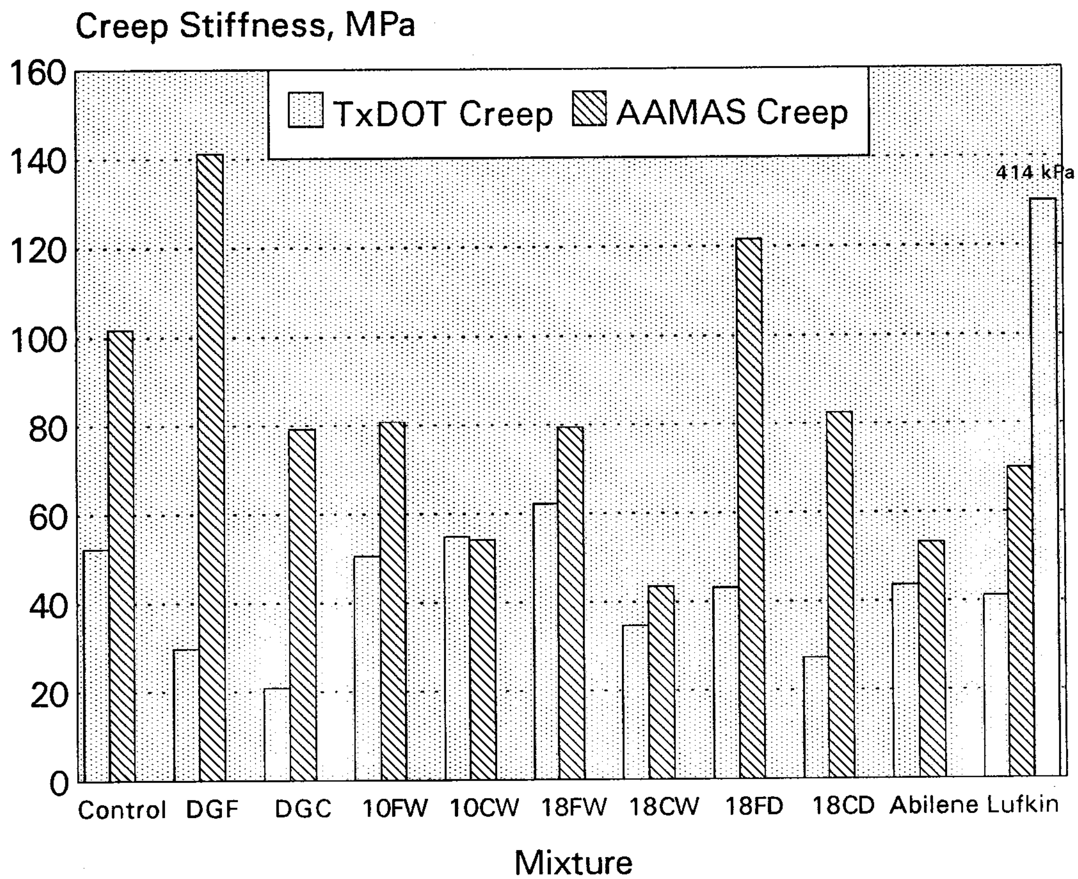


Figure 1. Creep Modulus or Stiffness for TxDOT and AAMAS Static Creep Tests.

The pavement section modeled is shown below.

Wheel load = 4082 kg (9000 lb)
Tire pressure = 896 kPa (130 psi)
↓

10 cm (4") HMAC or CRM HMAC ν = 0.35 E = Varies with mix.

15 cm (6") Flexible Base ν = 0.35 E = 241 MPa (35,000 psi)

15 cm (6") LTB ν = 0.25 E = 689 MPa (100,000 psi)

Semi-infinite subgrade ν = 0.45 E = 117 MPa (17,000 psi)

Resilient modulus values used in this model are tabulated along with the total vertical strain under the wheel load in the surface layer. The resilient modulus was measured at 40°C (104°F) for all mixes shown below in Table 3.

Table 3. Pavement Response for Different Mixtures.

Mix	Modulus at 40°C, MPa	Stress at Bottom of Layer, kPa	Vertical Strain, m/m	Max Allowable TxDOT Strain
Control	972	483	0.001831	
DGF	1140	462	0.001741	
DGC	892	496	0.001878	
10%FW	945	490	0.001846	
18%FW	723	531	0.001995	
10%CW	715	531	0.001989	0.00167
18%CW	770	517	0.001956	
18%FD	841	503	0.001909	
18%CD	718	531	0.001993	

There are certain limitations to the above analysis in that this represents a particular pavement response. From this analysis, we cannot draw any general conclusions. But in the present case, it is applicable because the only variable, or the response we are interested in, is the surface layer. The characteristics of that layer depend on the modulus of the mix.

From the above table, we can clearly see that the total vertical deformation under the wheel load is greater than that of the maximum allowable strain in the TxDOT creep. From the above data, we can say that pavements can withstand a higher deformation than what the present criterion calls for; therefore, the present criterion is conservative. However, it is also evident that the stress which is induced in the surface layer due to the wheel load is much higher than what is presently being used in the Tex-231-F creep test.

In order to simulate the field conditions, a higher stress level is proposed for the static creep test. Fortunately, as part of this study on rubber mixes, static creep tests were performed (AAMAS) at a higher stress level of 414 kPa (60 psi). This stress more closely approximates the stress state shown above in the elastic layered analysis. As mentioned previously, there is no correlation between results of the two different creep tests. This may be better explained by considering the material characteristics and plastic theory.

The stress-strain behavior of asphalt concrete is time dependent. Under load, strain has three components: elastic, viscoelastic, and viscoplastic. The recovery part of the creep curve also has three components: elastic, viscoelastic, and irrecoverable (or plastic deformation). Assuming the loading and recovery times are constant, as the stress intensity increases, the compliance of the material increases and also the plastic or irrecoverable deformation. As the stress intensity approaches the strength of the material, the log strain rate increases exponentially.

At low stress levels, the material exhibits elastic and viscoelastic response and very little viscoplastic response. When the material is unloaded, most of the strain is recovered and plastic strain is negligible. As we increase the stress level, keeping the loading and recovery time constant, permanent deformation or plastic damage increases. This is very much evident from the percent recovery in the two static creep tests performed in this study. Table 4 tabulates them below.

Table 4. Percent Creep Recovery at Two Different Stress Levels.

Mix	% Recovery (70 kPa or 10 psi, 10 minute recovery)	% Recovery (414 kPa or 60 psi, 60 minute recovery)
Control	76.3	12.5
DGF	61.7	20.8
DGC	74.8	34.0
10%FW	68.5	8.6
18%FW	57.5	19.6
10%CW	70.5	3.9
18%CW	69.2	2.9
18%FD	61.6	29.1
18%CD	58.3	15.2

From the above table, we can clearly see that under 70 kPa (10 psi), recovery is between 57 to 76% with only a 10 minute recovery period. But at a 414 kPa (60 psi) stress level and even with a one hour recovery time, the recovery ranges only between 3 and 34%. In 10 minutes, most of the recovery is due to the elastic portion, and only a small amount is viscoelastic. If the recovery time is extended to one hour, recoverable strain would be much higher because the viscoelastic recovery and permanent damage would be even less.

Since the mixes are exhibiting plastic deformation or plastic damage, it is appropriate to consider the plasticity theory to explain the plastic behavior of the mixes. Plasticity theory helps describe the observed plastic deformation by the stress-strain relationships under complex stress states induced in the material. In plasticity theory, stresses on octahedral planes are used to describe the stress state at a particular point. This is because stress at a point can be better described using octahedral stresses, and since octahedral stress is defined at a specific orientation, it is invariant.

An octahedral plane is a plane whose normal makes equal angles with each of the principal axes of stress. The planes with normal $|1/\sqrt{3}|(1,1,1)$ in the principal coordinate system are called octahedral planes. The normal stress on the face of the octahedral is given by:

$$\sigma_{oct} = 1/3\ (\sigma_1 + \sigma_2 + \sigma_3) \tag{1}$$

The shear stress on the face of the octahedral is given by:

$$\tau^2_{oct} = (1/9)[\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2] \tag{2}$$

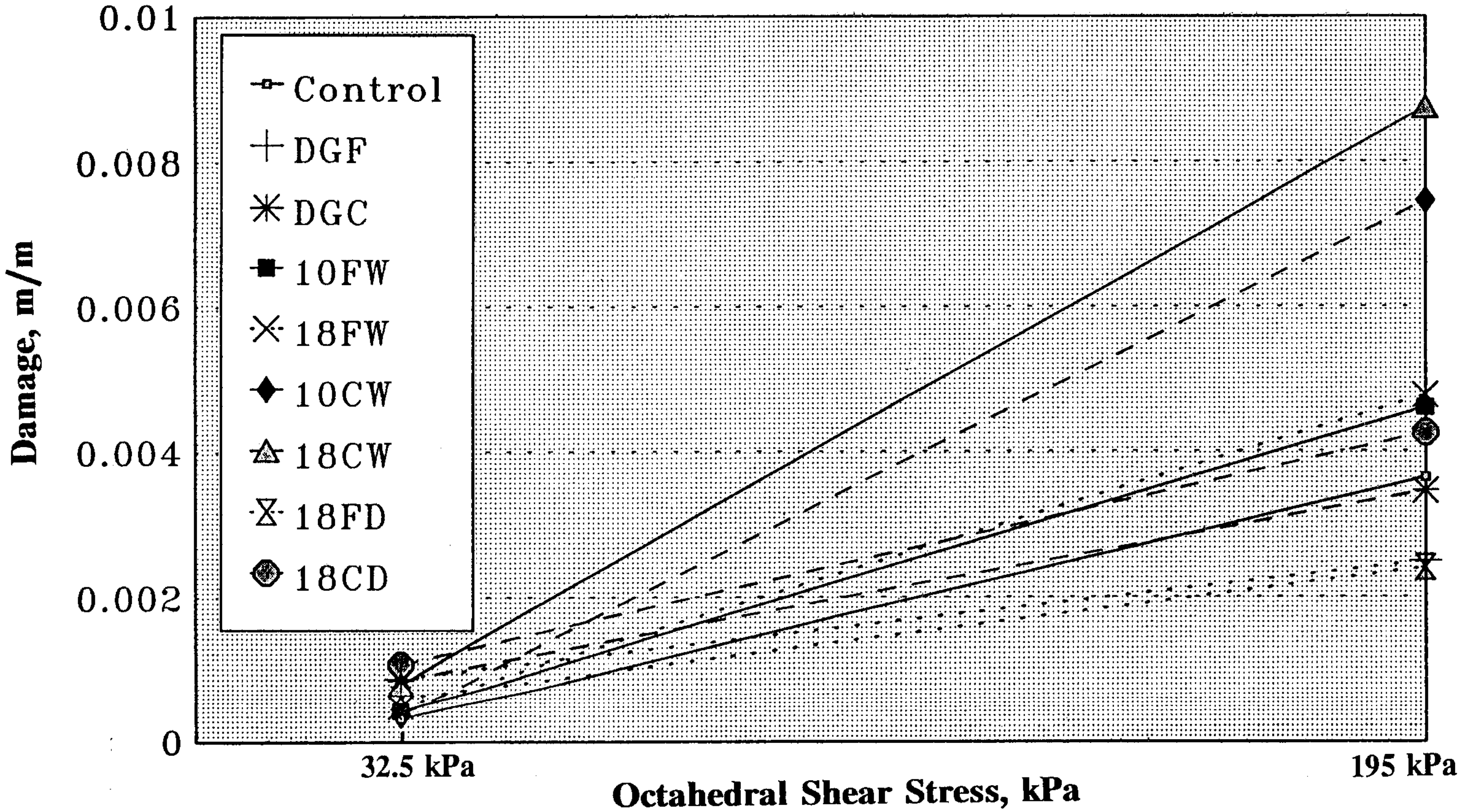
The octahedral shear stress at a point in terms of stress components (3-dimensional) referring to an arbitrary set of axes (x, y, and z) is shown below.

$$\tau^2_{oct} = (1/9)[\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 + 6(\tau^2_{xy} + \tau^2_{yz} + \tau^2_{zx}) \tag{3}$$

These criteria are now applied to describe the plastic deformation and stress state induced in the sample under creep test conditions. Calculate octahedral shear stress assuming that stresses are acting in the principal planes. Using equation (2), τ_{oct} is calculated. Now for all nine mixes, plastic damage or the irrecoverable portion of the creep strain is plotted on the Y axis and the τ_{oct} is plotted on the X axis in Figure 2. The recoverable strain is plotted versus τ_{oct} in Figure 3.

The following observations can be made from the above analysis:

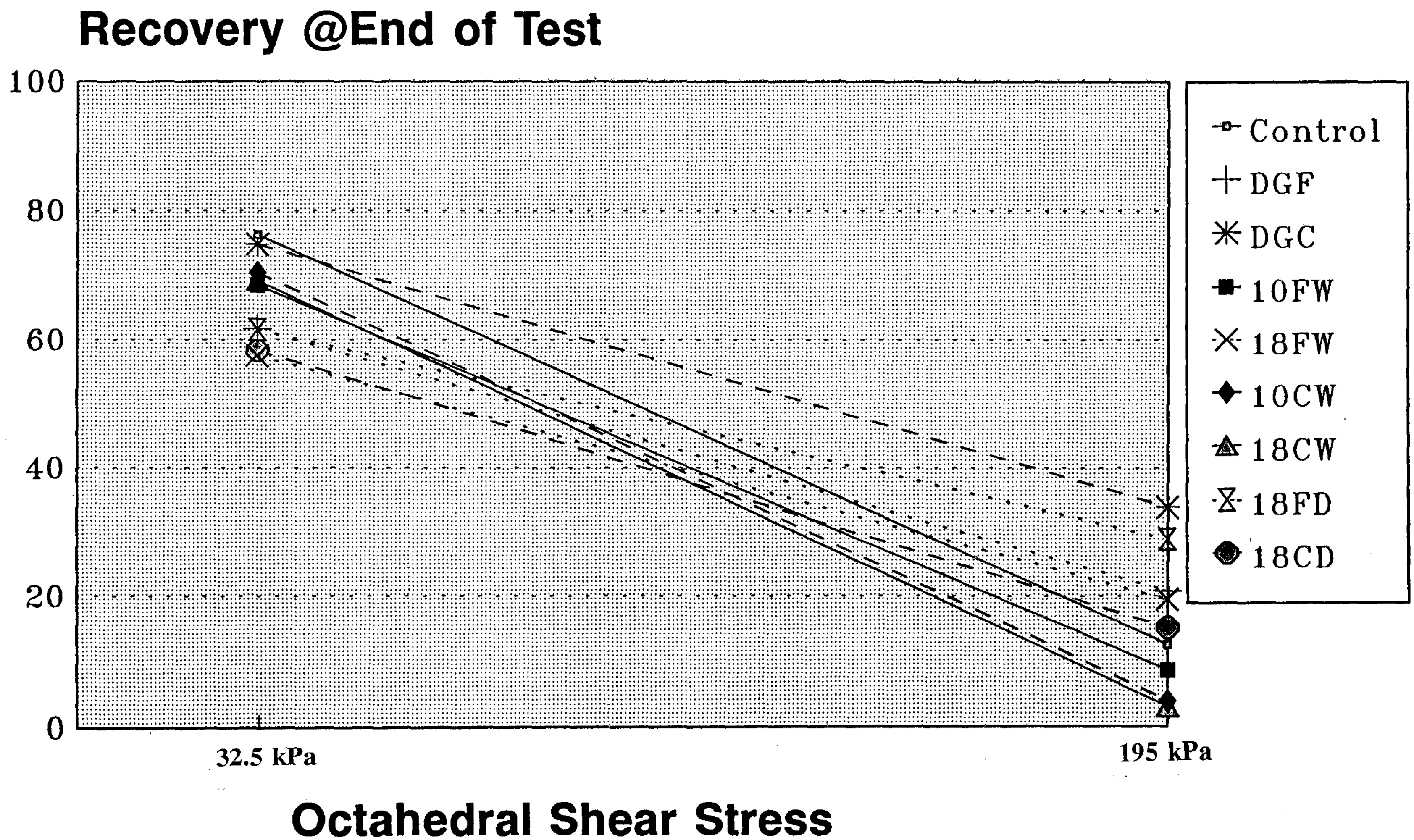
- At low stress levels, the damage induced in the sample is very low as compared to a high stress level.
- Mixtures that perform well at low stress levels do not necessarily perform well at high stress levels. Higher stress levels may be needed to identify mixes which are susceptible to permanent strain.



τ_{oct} for 70 kPa or 10 psi (TxDOT) = 32.5 kPa or 4.7 psi

τ_{oct} for 414 kPa or 60 psi (AAMAS) = 195 kPa or 28.3 psi

Figure 2. Damage versus Octahedral Shear Stress Using Static Creep Test Data.



τ_{oct} for 70 kPa or 10 psi (TxDOT) = 32.5 kPa or 4.7 psi

τ_{oct} for 414 kPa or 60 psi (AAMAS) = 195 kPa or 28.3 psi

Figure 3. Recovery Versus Octahedral Shear Stress Using Static Creep Test Data.

Generally, the static uniaxial creep test is sufficient to prioritize different mixtures in terms of relative resistance to permanent deformation. The creep tests performed in this study were without confining pressure. The uniaxial creep test is highly dependent on the cohesion of the binder and the mastic portion of the mixture. However, recent testing on stone mastic and open graded mixtures demonstrates that in certain cases, a realistic comparison of stone mastic type mixtures requires application of a confining pressure to more closely simulate the actual field condition (Little and Youssef 1992; Krutz and Sebally 1993).

The CMHB rubber mixtures analyzed in this study are similar in gradation to a stone matrix-type mixture. It has already been published and demonstrated in the field that SMA mixtures are more rut resistant compared to dense graded mixtures (*Report on the 1990 European Asphalt Study Tour* 1991; Carpenter 1993; Emery et al. 1993). But similar results are not seen in the laboratory testing (Brown 1993). This can be explained easily considering the composition of mixtures as well as the load transfer phenomenon through the mixtures. It has already been mentioned that permanent deformation occurs due to one dimensional consolidation and plastic flow. All of the discussion above and evaluation of static creep accounts only for one-dimensional consolidation. Assuming that rutting occurs only in the wheel path, one-dimensional consolidation occurs partly due to the void structure. If proper construction practices are used and the mixture is designed properly, one dimensional consolidation should not be a major problem. Support conditions also play a very important role in terms of rut depth.

Applying similar confining pressures for both dense and open-graded mixtures does not necessarily simulate field lateral support for the mixtures because of the differences in the aggregate structure in the mixtures. One-dimensional consolidation is invariably associated with plastic flow. This can be seen from Figure 4.

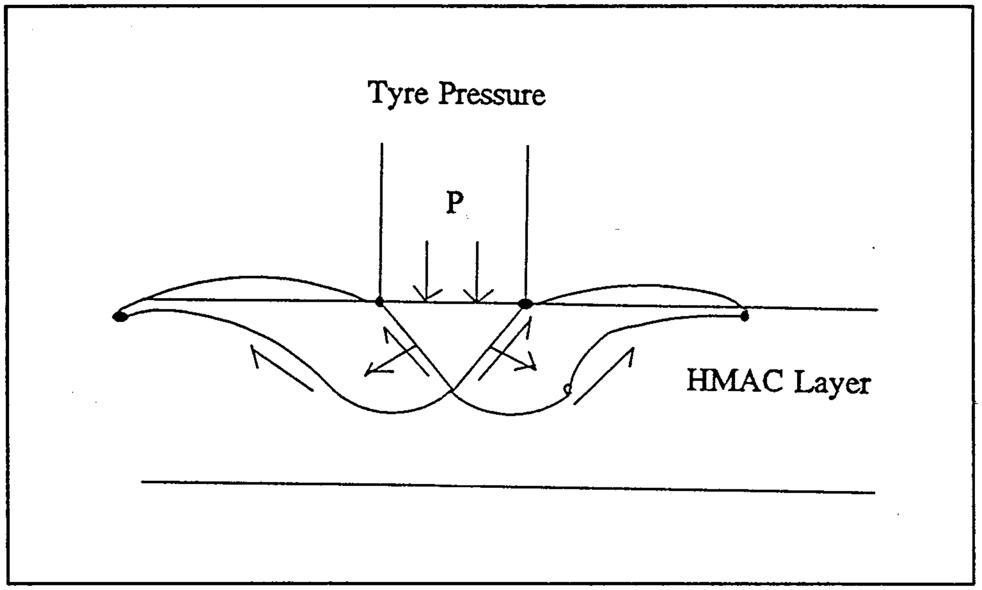


Figure 4. **Failure Plane for a Rough Foundation on Weightless Frictional Soil (Jumikis 1984.)**

From soil mechanics, the above figure shows a typical shear or failure plane for a rough foundation on weightless frictional soil (Jumikis 1984). The same failure criteria can be extended to explain the mechanism of rutting for SMA-type mixtures. This is an over simplification of what is happening in the field (the SHRP study considers a three dimensional plastic flow analysis). One-dimensional consolidation occurs right below the tire. Plastic flow occurs in the rest of the region as shown in the figure. The area underneath the tire is moving downwards with forces acting perpendicular to the side. Because of the boundary conditions that exist in the pavement (semi-infinite in the direction perpendicular to the wheel path and infinite in the direction of the wheel path), the only direction that the mixture can flow is towards the surface.

Now consider a dense graded mixture and CRM mixture in the above loading conditions in the pavement. In SMA mixtures, because of the particulate arrangement, after forming the stone skeleton, the shear forces on the shear plane are resisted by the load transfer across the shear plane in pure shear by the aggregate. In SMA mixtures, high quality, crushed aggregates are used and load transfer is assured because of the good quality crushed aggregate. Considering dense-graded mixtures, stone to stone contact in the shear plane may not occur. So load transfer across the shear plane has to be resisted by both aggregate and binder; however, at elevated temperatures, strength of the binder is greatly reduced. This explains the reason why SMA mixtures perform well in terms of rutting compared to dense-graded mixtures in the field.

Shifting the discussion back to laboratory test conditions, the test data indicate that CRM mixtures have higher initial strains. From the field, SMA mixtures in St. Louis, Missouri in 1991 developed 3 to 6 mm rutting almost immediately after placement (Carpenter 1993). However, no further rutting has been reported. Combining both these statements, it may be inferred that once the aggregates form the skeleton (after the initial loading), all the above discussed mechanisms come into play to resist creep strains and plastic deformations. Considering the failure strains in compression, which are in Table 5, one can see the effect of higher film thicknesses in SMA-type mixtures. This explains the reason for higher creep strains for CRM mixtures than dense-graded mixtures in the laboratory test conditions.

From the above discussion, we can conclude that the static creep test alone cannot predict the performance of the mixture against rutting. The uniaxial repeated load permanent deformation test still suffers from the inability to fully evaluate mineral aggregate interaction and internal friction due to lack of confinement. The repeated loading effect does perhaps provide some insight into the mixture that the uniaxial creep test does not provide, that is, the ability to evaluate the effect of repeated loading on plastic deformation among aggregate particles.

The only way to improve the creep test to better account for mineral interlock is through applying confinement. Similar confining pressures cannot be used for both dense and gap-graded mixtures because of differences in Poisson's ratio. Therefore, while applying confinement, care should be taken that the Poisson's ratio of the materials considered are the same. Or, perhaps another way to approach the analysis of mixtures is to use the creep test as a means to evaluate the role of the binder and the mastic in deformation resistance and to couple this test with a simple shear strength test, such as a simple tri-axial test to evaluate the mineral aggregate internal friction.

Thus, one of the most complete laboratory evaluations of permanent deformation for SMA or CMHB types of mixtures could be the simple shear test as prescribed in SHRP.

Table 5. Compressive Strain at Failure for CMHB CRM Mixtures and Dense Graded Mixtures 40°C.

Mixture Type	Compressive Strain at Failure
Control (Dense-Graded)	0.0221
Dense-Graded with Fine CRM (DGF)	0.0203
Dense-Graded with Coarse CRM (DGC)	0.0346
10% Fine CRM-Wet Method (10%FW) (CHMB gradation)	0.0359
10% Coarse CRM-Wet Method (10%CW) (CMHB gradation)	0.0371
18% Fine CRM-Wet Method (18%FW) (CMHB gradation)	0.0389
18% Coarse CRM-Wet Method (18%CW) (CMHB gradation)	0.0312
18% Fine CRM-Dry Method (18%FD) (CMHB gradation)	0.0372
18% Coarse CRM-Dry Method (18%CD)(CMHB gradation)	0.0276

2.2 Fatigue Cracking

A longer term distress mode considered by most design and evaluation procedures is fatigue cracking. Fatigue failures are accelerated by high air voids which, in addition to creating a weaker mixture, also increase the oxidation rate of the asphalt film. The analysis was done according to the method suggested in AAMAS, and previous research report 1332-1 (Estakhri et al. 1993) provides some discussion. Under SHRP contract A-005 (Lytton et al. 1993), analysis techniques were developed to analyze the fatigue cracking by applying the fracture mechanics to viscoelastic materials. Even though the testing in this study was done to obtain the material properties using AAMAS, an attempt is made to analyze the data using the fatigue model described in SHRP. Both of these analyses are described separately. Conclusions are drawn, keeping both these analyses in perspective.

AAMAS Criteria

The development of fatigue cracks is related to the tensile strain at the bottom of the asphaltic concrete layer. Under the wheel load, the HMAC layer is in flexure; therefore, the tension zone will be at the bottom of the HMAC layer. Cracks start under the wheel load in tension and propagate to the surface of the pavement. Low temperatures are critical for fatigue cracking to develop. Figure 5 presents the evaluation criteria by which fatigue potential is evaluated in AAMAS based on the mixture properties of indirect tensile strain at failure and diametral resilient modulus. The relationship between indirect tensile strain at failure and diametral resilient modulus in Figure 5 is derived based on the generalized fatigue relationship (Von Quintus et al. 1991):

$$N = K_1(\epsilon_t)^{-n} \tag{4}$$

where N is the number of loading applications or cycles, ϵ_t is the tensile strain at the bottom of the asphalt concrete pavement layer, and K_1 and n are fatigue regression constants and are given by the following equations.

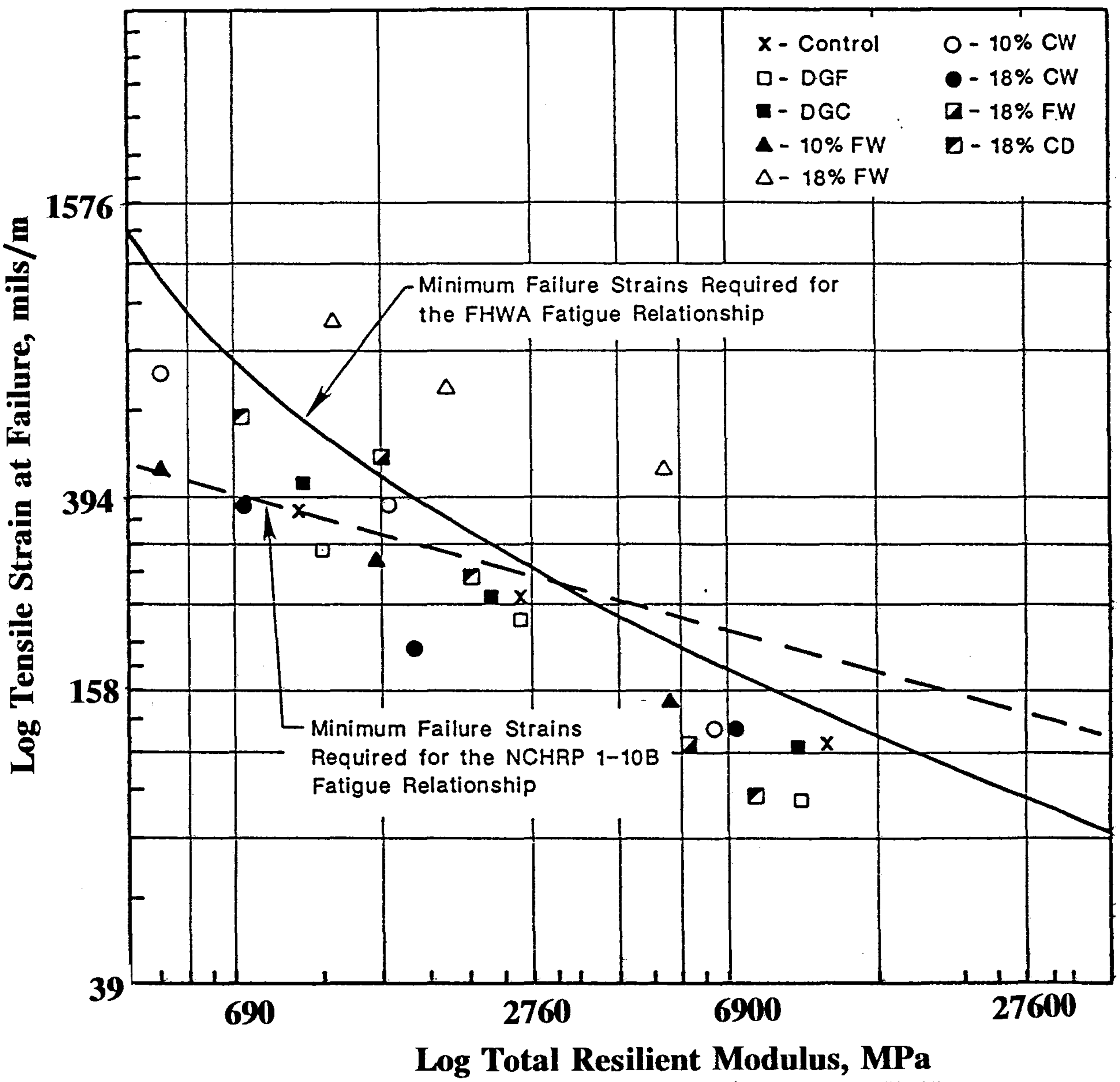


Figure 5. AAMAS Chart for Evaluating Fatigue Cracking Potential for Asphalt Concrete Mixtures.

$$K_1 = K_{1R}(E_R / E_{Rr})^{-4} \quad (5)$$

$$n = 1.75 - 0.252 \log K_1 \quad (6)$$

Where: E_R = Resilient modulus of asphalt concrete at a selected temperature
E_{Rr} = 500,000 psi; reference modulus from AASHTO road test
K_{1R} = 7.87 X 10^{-7}; reference coefficient for $E_R = E_{Rr}$

For purposes of AAMAS, the standard mixture is the dense-graded asphaltic concrete placed at the AASHTO Road Test. The fatigue curves from NCHRP 1-10B (Finn et al. 1977) were developed from these data, which have been used in other research and design studies (Rauhut et al. 1984; Austin Research Engineers 1975). Figure 5 shows two relationships between the total resilient modulus and indirect tensile strain at failure for the standard mixture. The difference is that the NCHRP 1-10B assumed a constant slope of the fatigue curves; whereas, the FHWA study varied the slope of the fatigue curves.

If the total resilient modulus and indirect tensile strains at failure for a particular mixture plot above the standard mixture (FHWA fatigue curve is recommended), it is assumed that the mixture has better fatigue resistance than the standard mixture.

From Figure 5, it appears that all of the mixtures, except one, have about the same fatigue potential and are inferior to the standard mix in terms of fatigue resistance potential as characterized by the FHWA relationship. This means that most of the crumb rubber modified mixtures tested in this study are more fatigue susceptible than the AAMAS standard mixture but may not be any more susceptible than conventional dense-graded Type D mixtures currently used in Texas. The mixture produced with 18 percent fine CRM by the wet method has a significantly better fatigue resistance than the others.

The number of repetitions to failure is calculated using equation (4). Resilient modulus values of temperature conditioned samples are used in this analysis. The tensile strain at the bottom of the pavement is obtained by modeling a pavement using

ChevPC, an elastic layered analysis program. The pavement section described on page 13 was modeled to obtain the tensile strain at the bottom of the HMAC layer. The inputs for this program include resilient modulus and Poisson's ratio for each layer. A tire pressure of 896 kPa (130 psi) was assumed. This program is capable of calculating stresses, strains, and deflections in vertical, radial, and tangential directions. The number of load repetitions to failure, strain at the bottom of the HMAC layer, and resilient modulus of temperature-conditioned samples are given in Table 6.

Table 6. Number of Repetitions to Failure for Control and Crumb-Rubber Mixtures.

Mixture Type	Resilient Modulus at 5° C (Temperature Conditioned), MPa (psi),	Tensile Strain at Bottom of HMAC Layer	Number of Repetitions @Failure
Control	16270 (2360000)	0.000107	8872157
DGF	14024 (2034000)	0.000118	6149229
DGC	14459 (2097000)	0.000116	6634119
10FW	7967 (1155500)	0.000167	1763998
10CW	7754 (1124530)	0.000170	1669958
18FW	16531 (2397490)	0.000106	9350933
18CW	9315 (1351000)	0.000152	2414430
18FD	6702 (972000)	0.000185	1274492
18CD	10798 (1566000)	0.000139	3323062

From Table 6, it can be seen that the number of repetitions to failure (or fatigue life) is directly proportional to the resilient modulus or stiffness of the mixture. It has already been discussed in earlier sections that SMA and CMHB-type mixtures have lower stiffness due to higher asphalt film thickness. It is also known that SMA type mixtures do not depend on stiffness of the binder for stability. The above analysis technique was developed for dense-graded mixtures. Because of these reasons, the above analysis cannot be applied to SMA and CMHB-type mixtures. However, one mixture, 18% FW, has a resilient modulus value higher than control mixture. Keeping the discussions on resilient modulus and indirect tensile strength test in perspective, we may infer that in the 18% FW mixture, a three-dimensional matrix is being formed by the binder and fine rubber, increasing the elastic properties as well as reducing the temperature susceptibility.

The above analysis is a very good approximation for the fatigue analysis of dense-graded mixtures. This analysis assumes that the HMAC layer is in bending or flexure (beam on-grade). Even at low temperatures, HMAC is a linear viscoelastic material and not a brittle material like concrete. If, however, we assume that asphalt concrete is brittle at lower temperatures, cracks will form at the bottom of the HMAC layer. If a crack propagates in tension to the surface, only one condition is possible: that the failure is in the base course. Studies show that pavements have higher fatigue lives in the field than those estimated using models. It is suggested that micro-crack healing occurs in asphalt concrete and is responsible for higher fatigue lives (Lytton et al. 1993; Kim and Little 1990). Because of the above limitations and the nature of CMHB and SMA mixtures, it is not appropriate to come to any conclusion on the performance of these types of CRM mixtures in terms of fatigue lives.

SHRP Criteria

Under the Strategic Highway Research Program, models were developed to predict fatigue cracking and fatigue lives of the asphalt concrete mixtures (Lytton et al. 1993). These were developed using a fracture mechanics approach for calculating the crack growth. Laboratory tests used for this purpose were beam fatigue tests with constant strain and constant stress conditions. One of the greatest advantages of this analysis is that (in addition to resilient modulus) this model takes into account material properties like indirect tensile strength, compliance of the mixture, strain energy storage density, and size of the specimen. This analysis can be used to calculate fatigue lives at any given temperature. A brief description of the model is given in the following paragraphs.

SHRP Fatigue Model

This model considers the crack propagation in two phases: crack initiation and crack propagation. Because of repeated loading, microscopic cracks form at the bottom of the asphalt concrete layer. These micro cracks grow in size and form into visible cracks. This is the crack initiation phase. These visible cracks then grow and reach the free surface. This is called the crack propagation phase.

Crack Initiation Model

The number of load repetitions to reach crack initiation was developed from the results of beam fatigue tests performed under both constant stress and constant strain loading conditions. The number of load cycles to reach this condition depends on original stiffness, state of stress in terms of both mean principal stress and octahedral shear stress at the bottom of the asphalt concrete, the percent air voids and the asphalt binder in the mixture. The number of repetitions to reach this is given by the following equation:

$$\log_{10}N_i = b_0 + \{b_1 + b_2\,\sigma_m + b_3\,[(\sigma_m)^2 + 2(1+\mu)(\tau_{oct})^2]\}\,E + (b_4 \log_{10}\sigma_m + b_5 \log_{10}E)\,(\%AC) + \{b_6\,[(\sigma_m)^2 + 2\,(1+\mu)(\tau_{oct})^2]/E + b_7 \log_{10}\sigma_m\}\,(\%Air) + [b_8\,(\sigma_m/E) + b_9 \log_{10}\sigma_m]\,(\sigma_m/E) \tag{7}$$

Where

N_i	=	number of load cycles to crack initiation
σ_m	=	mean principal stress, psi
τ_{oct}	=	octahedral shear stress, psi
E	=	asphalt concrete modulus, psi
% Air	=	air voids content, percent
%AC	=	asphalt content by weight percent
μ	=	Poisson's ratio

b_0 = 4.415936	b_1 = -5.421 x 10^{-6}	b_2 = 1.11 x 10^{-7}
b_3 = -8.51796 x 10_{-11}	b_4 = -0.838837	b_5 = 0.314813
b_6 = 3.089278	b_7 = -0.114846	b_8 = 35787201
b_9 = -12144	b_{10} = 40.8396	

Crack Propagation Model

The crack propagation model is based on Paris and Erdogan (1963) given by:

$$N_p = \frac{1}{A}\int_{c_o}^{h}\frac{dc}{k^n} \tag{8}$$

Where:

N_p	=	number of load repetitions to propagate a crack of initial length c_o to the surface (c_o assumed to be equal to 0.3 in.)
h	=	layer thickness
c_o	=	initial crack length
k	=	stress intensity factor
A, n	=	material properties

This crack propagation occurs in two modes: tension and shear. Stress intensity factors were calculated for both these modes. The stress intensity factor in the tension mode is represented by K_I, and stress intensity factor in the shear mode is represented by K_{II}. The total number of repetitions for the crack to reach the surface is the sum of the repetitions in the tension and shear mode.

The number of repetitions in the tension mode [equation (9)] and shear mode [equation (10)] can be calculated as follows:

$$N_f = \frac{d^{1-\frac{n}{2}}}{AE^n r^n (1-nq)}\left[1-\left(\frac{c_o}{d}\right)^{1-nq}\right]\left[\frac{1}{\epsilon_t}\right]^n \tag{9}$$

$$N_f = \frac{d^{1-\frac{n}{2}}}{AG^n r^n (1-nq)}\left[1-\left(\frac{c_o}{d}\right)^{1-nq}\right]\left[\frac{1}{r}\right]^n \tag{10}$$

Where:

N_f	=	number of repetitions for the crack to grow
d	=	the length the crack must grow
r,q	=	coefficients found from the analysis of stress-intensity factor, K as it varies with crack length, c
c_o	=	initial crack size
E	=	elastic modulus, psi
A, n	=	fracture parameters for the asphalt concrete mixture

A and n for asphalt concrete mixtures is given by the following equations developed by Schapery:

$$A = [D_1 \lambda(m) \frac{\Pi^{1+2m}}{4}]^{\frac{1}{m}} \int_0^{\Delta t} W(t)^n \frac{dt}{\Gamma^{\frac{1}{m}} \sigma_t^2 I^2} \tag{11}$$

$$n = 2(1+1/m) \text{ or } 2/m \tag{12}$$

Where:

D_1	=	compliance coefficient, D_1, in the power-law creep compliance
m	=	slope of the log compliance vs. time graph
σ_t	=	the tensile strength of the material
Γ	=	released strain energy storage density of the material, also called fracture energy density
λ(m)	=	a function of m which has nearly a constant value of 0.33
Δt	=	the time the load is applied
W(t)	=	the normalized wave-form of the applied load with time. Its value ranges between 0 to 1
I	=	the value of the integral of the dimensionless stress-strain curve of the material. Its values range between 1 and 2.

Since the evaluation was based on laboratory testing prescribed in AAMAS, some modifications were made for the present analysis. All the variables to calculate A, given by equation 11 were not available. For that reason, a simplified equation developed by Moolenar (1984) from the experimental results was used.

$$\log A = 4.389 - 2.52 * \log(E * \sigma_m * n) \tag{13}$$

Where:

E	=	elastic modulus, kpa
σ_m	=	tensile strength, kpa
n	=	fracture parameter

A description of the analysis performed in this study along with the modifications are described in a step-by-step fashion:

1. The number of repetitions to failure in terms of tension is calculated by equation (9).
2. 'A' is calculated by equation (13) derived by Moolenar. Care was taken to convert all the units to S.I. In order to account for the strain energy density, the area under the stress-strain curve from indirect tensile strength test is calculated. The value of 'A' is normalized with the dense-graded mixture. From equation (11), it is seen that $\Gamma^{1/m}$ is a constant and can be taken out of the integration sign. The constant 'A' is modified by the function given in the following equation:

$$A_{21} = A_2 \left[\frac{\Gamma_1^{\frac{1}{m_1}}}{\Gamma_2^{\frac{1}{m_2}}} \right] \tag{14}$$

Where:

A_{21}	=	modified A of the mixture being normalized
A_2	=	A value of the mixture before modification
Γ_1	=	area under stress-strain curve for reference (control) mixture KN-m/m^3
Γ_2	=	area under stress-strain curve for mixture to be modified.

m_1 = slope of the log compliance curve for reference mixture

m_2 = slope of the log compliance curve for mixture to be modified.

3. The pavement section shown on page 10 is used in the present case also. It is also assumed that the pavement section is in the wet, no-freeze zone. The equation for 'n' for the wet, no-freeze zone is given by

$$n = -1.615 + (1.98/m) \tag{15}$$

4. Initial crack size c_o is taken as 7.6 mm (0.3 inches). This is found from previous testing. The total length the crack has to grow is assumed to be 102 mm or 4 inches.
5. r and q, coefficients of stress-intensity factor K are derived by Tseng and Lytton (1990). They are 4.397 and 1.18, respectively.
6. Tensile strain (ϵ_t) at the bottom of the HMAC layer is calculated using Mich-Pave an elastic-layered analysis computer program.
7. Table 7 presents all of the results.

From Table 7, it can be seen that the CRM mixtures performed better than the dense graded control mixture. The only exception is the mixture containing 18% coarse rubber added dry. Mixtures containing 18% fine rubber added both wet and dry performed better than any other mixtures. Mixtures containing rubber contents added by wet method and the dense graded mixture containing fine rubber have similar performance compared to a dense graded control mixture. In general we can say that fine rubber yields a better performance compared to coarse rubber. Also, the wet method appears to be superior with the exception of 18% fine rubber added dry. It is interesting here to note that the coarse aggregate content for both the 18FW and 18FD are approximately same (91%).

Table 7. Number of Repetitions to Failure for Control and CRM Mixtures Using SHRP Criteria.

Mixture Type	Slope of log Compliance Curve, m	Fracture Parameters		Number of Repetitions to Failure, N_f
		n	A	
Control	0.25131	6.264	3.28E-24	3.25E+07
DGF	0.35185	4.012	1.97E-23	5.57E+11
DGC	0.27551	5.572	6.89E-24	6.03E+08
10FW	0.34978	4.046	9.81E-23	2.28E+11
10CW	0.41998	3.099	1.05E-21	2.26E+12
18FW	0.33000	4.385	2.07E-26	6.29E+13
18CW	0.37875	3.613	6.58E-22	2.23E+11
18FD	0.47936	2.516	3.16E-21	1.64E+13
18CD	0.25241	6.229	1.01E-22	3.16E+06

From the above analysis it is clear that one property influences the predicted performance the most: compliance of the mix. There is a linear relationship between the log number of repetitions to failure and n. This is shown in Figure 6. It can also be seen that the effect of the other fracture parameter, A, is not as influential as n or the compliance. This is the reason why, even with a very low tensile strength and modulus, the fatigue performance of 18%FD is superior. All the mixtures follow the same reasoning except the 18%FW mixture. This is due to fracture parameter A. 18%FW has the highest modulus and tensile strength. From equation (13) and equation (14,) we can say that A is inversely proportional to modulus and strength and inversely proportional to the number of repetitions to failure. Thus 18%FW has the lowest A value. Also this particular mixture has a higher compliance value than the dense-graded control mixture.

So from the above discussion we can conclude that both A and n, compliance, modulus and tensile strength of the mixture are very important to predict the fatigue performance of mixtures in tension. Also it may be inferred that there is a range of A and compliance for which optimum performance of mixtures can be expected. If shear failure is considered, CRM mixtures would perform better than dense graded mixtures because of the particulate structure.

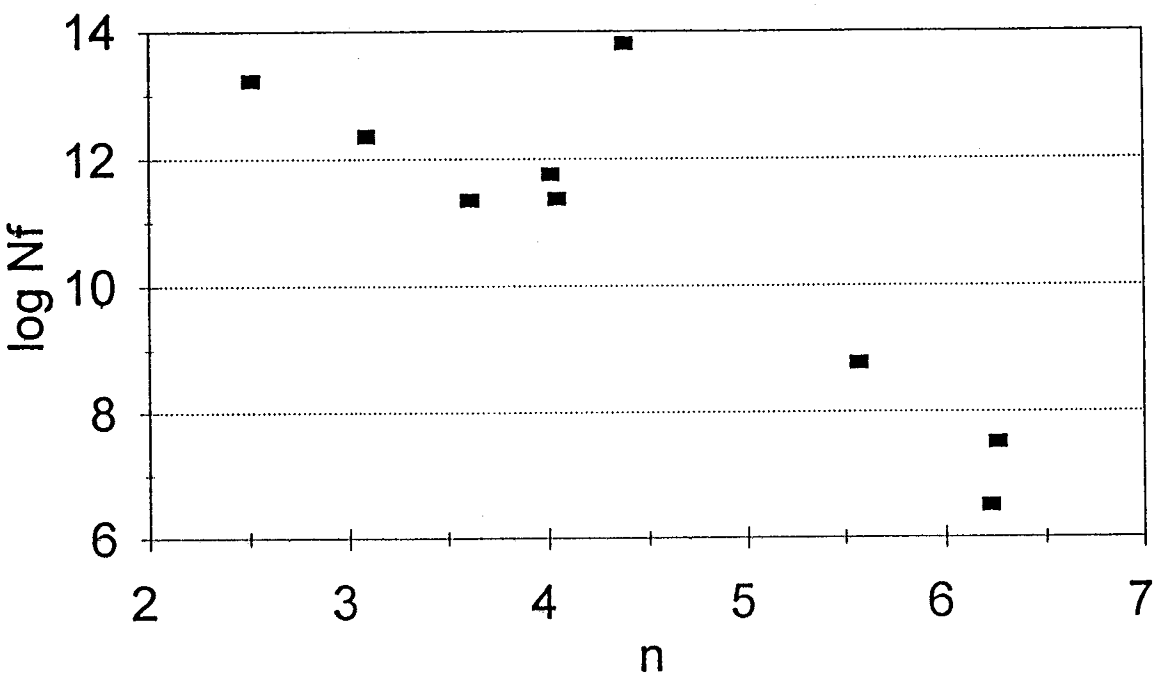


Figure 6. Variation of Log Number of Repetitions to Failure with 'n.

2.3 Thermal Cracking

Thermal cracking is considered a non-traffic-associated fracture distress that is common, but not confined, to the northern United States (Von Quintus 1991). Low temperature cracking results when the tensile stresses, caused by temperature drops, exceed the mixture's fracture strength. The rate at which thermal cracks occur is dependent on the asphalt rheology properties, mixture properties, and environmental factors.

The mixture properties which are used to evaluate thermal cracking include indirect tensile strength, low-temperature creep modulus, failure strains, Ring and Ball softening point of the binder, and coefficient of thermal contraction. These mixture properties are measured on age-hardened specimens (environmental aging simulation). A discussion of the thermal cracking analysis is below. Thermal coefficient of contraction is not usually measured in the laboratory, but assumed for the thermal stress calculations.

The other three properties are measured in the laboratory. Mixture strength is measured using the indirect tensile strength test on aged/temperature hardened specimens at a loading rate of 1.27 mm (0.05 in) per min. Creep modulus is evaluated using the indirect tensile creep modulus test after a loading time of 3600 seconds and a recovery time of 3600 seconds. Ring and ball temperature is obtained by performing a softening point test on the binders.

The temperature at which crack initiates can be estimated by using the following mathematical relationship:

$$\Delta T = \left[\frac{E_{ct}(T_i)}{E_o}\right]^{\frac{1}{n_t}} \frac{t_r^{\,n_c}}{\alpha_A E_o(T_i)} \tag{16}$$

Where ΔT is the critical temperature change at which cracking can occur; $E_{ct}(T_i)$ is the indirect tensile creep modulus measured at temperature T_i; E_o and n_t are the regression constants from the graph plotted on a log-log scale between resilient modulus and tensile strength of the mixtures (unconditioned) at three different temperatures. E_o is the intercept in psi, and n_t is the slope. Relaxation time t_r is usually assumed as 3600 seconds. n_c is the slope of the indirect tensile creep curve at temperature T_i. $E_o(T_i)$ is the intercept of indirect tensile creep curve in psi units.

The coefficient of contraction α_A is assumed same for all the mixtures. A typical value for α_A is between 1.0 and 1.8 x 10^{-5} in/in/°F. For the purpose of analysis, an average value of 1.4 x 10^{-5} is assumed. $E_{ct}(T_i)$ is calculated from the following equation

$$\log E_{ct} = \log E_o + n_t \log S_t(T_i) \tag{17}$$

where $S_t(T_i)$ = tensile strength, at temperature T_i.

The critical temperature change at which pavement cracks was calculated to be less than absolute zero (273 °C), a temperature which does not exist. Calculations were checked again such that there are no computational errors. Thus, this analysis was abandoned and the following was pursued.

Thermal Cracking - Strain Energy Criteria

To compare the performance of the mixtures against thermal cracking, researchers decided to use strain energy to crack the mixture. Thermal cracks are induced in the pavement when the tensile stress induced in the mixture is greater than the tensile strength of the mixture. Thermal cracking also depends on the strain at failure. So both tensile strength and tensile strain of the mixtures at failure are important when considering the thermal cracking. A simplified analysis combining both of these parameters is considered. The indirect tensile strength test was performed on temperature conditioned samples. The area under the indirect tensile strength curves were calculated. This is energy dissipated or work done to fail the sample. Areas under these curves were calculated and listed in Table 8.

From Table 8, it is clear that only one mixture, 18FW is significantly different from all other mixtures; therefore, superior performance should be expected from 18FW. Other CRM mixtures are comparable to the dense-graded control mixture.

Table 8. Energy Required to Fail the Sample in Indirect Tension Mode for CRM and Control Mixtures.

Mixture Type	Area Under Stress-Strain Curve (lb-in/in^3)	Area Under stress-Strain Curve (KN-m/m^3)
Control	0.17603	1.215
DGF	0.19261	1.329
DGC	0.18435	1.272
10FW	0.21562	1.488
10CW	0.12625	0.871
18FW	0.77398	5.340
18CW	0.12849	0.887
18FD	0.15383	1.061
18CD	0.12421	0.857

3

Performance Evaluation of CRM Mixtures Using TFPS

Crumb rubber modified mixtures, discussed in the previous chapter, were evaluated for performance using the Texas Flexible Pavement System (TFPS). These mixtures were evaluated for different climatic conditions and substructures. TFPS is a computer program that was developed as a pavement design aid. It is a comprehensive design tool combining structural design with material characterization. Pavement design is accomplished using the principles of elastic-layered analysis, considering the material characteristics of different layers. Then the given structure is checked against deterioration over the design life period. TFPS considers 3 important distresses: rutting, fatigue cracking, and low temperature cracking. These distresses are accounted for in terms of the present serviceability index loss. This program is specifically developed for pavements in Texas; therefore, all of the climatic data is stored in the program and can be automatically accessed for a given district and county. Pavement damage induced by swelling soils can be analyzed using this program. It also incorporates overlay design and can perform cost analysis for different design alternatives. At this point, it should be noted that TFPS is *not* a material characterization program.

3.1 Performance Evaluation

The state of Texas can be broadly divided into two climatic zones: hot-wet and cold-dry. Five different structures were evaluated for the overall performance of the pavements using nine laboratory mixtures and one field mixture in the two climatic conditions. The performance of thick surface layers versus thin layers was also evaluated. TFPS is capable of analyzing base structures with (1) black base only, (2) combination of black base and granular base, and (3) granular base only. Each of these is considered a different structure. Analysis was performed only for the soft subgrade condition, which is the critical condition for the distresses reaching failure levels. The list of structures evaluated are shown in Table 9.

Table 9. Factorial Experiment Used for the Evaluation of Control and CRM Mixtures Using TFPS.

District: Beaumont County: Jefferson					
District: Amarillo County: Armstrong					
Structure	1	2	3	4	5
Surface Layer	Thin 38 to 64 mm	Thick >102 mm	Thin 38 to 64 mm	Thick >102 mm	Thick >102 mm
Black Base	Yes	Yes			
Black Base + Granular Base			Yes	Yes	
Granular Base Only					Yes
Subgrade	Soft	Soft	Soft	Soft	Soft

Several assumptions were made to analyze the structures. These are briefly described below.

Design Criteria

The design period was assumed to be 20 years which is typical for a rural highway carrying medium to heavy traffic volumes. This roadway is assumed to be a high-speed facility for the permanent deformation calculation purposes. A high reliability of 95% is assumed, taking into consideration the design life period and traffic volume. Initial serviceability index is assumed to be 4.2 which is typical for asphalt concrete pavements. The terminal serviceability index is assumed to be 2.5 which is typical for any type of pavement at the end of its serviceability. The serviceability index level after an overlay should be equal to the initial serviceability index which is equal to 4.2. It was assumed that the pavement failed when it reached a cracking area of 40% or a rut depth of 13 mm (0.5 in), or a combined effect which creates a situation where present serviceability index drops to a value of 2.5.

Traffic

A traffic level of 8000 ADT (15% trucks) with a growth rate of 3% is assumed for the analysis. From the traffic data input, the total number of equivalent single axle loads (ESALS) accumulated over the design life period were calculated to be 7 million. This traffic level is assumed for all the repetitions.

Material Characterization

Surface Course

For calculation purposes, TFPS considers a single input modulus at 21°C (70°F), and using the Asphalt Institute equation, it calculates the modulus for a given temperature. Ring and ball softening point is also an input. These two values are determined from laboratory tests and vary from mixture to mixture as shown in Table 10.

Table 10. Resilient Modulus and Ring and Ball Softening Point for Laboratory Tested Mixtures.

Mixture	Resilient Modulus, MPa	Ring and Ball Softening Point, °C
Control	4136	46.1
DGF	3654	51.5
DGC	3171	51.5
10FW	1861	51.5
10CW	2068	53.5
18FW	2275	61.3
18CW	2413	62.6
18FD	1930	61.2
18CD	2895	62.6
Abilene Field Mix	3791	62.6

Asphalt Treated Base Course

A typical Type B base course is assumed and modulus is calculated by the program using the Asphalt Institute equation at 21°C (70°F). The program also calculates the fatigue coefficients necessary to calculate fatigue cracking.

Overlay

It is assumed that the mixture used for the surface layer is also used to overlay when the distresses reach critical levels. This is done to predict the performance over the design life period and also for the compatibility of the materials (CMHB CRM mixtures versus dense-graded mixtures).

Granular Base

It is assumed that high quality aggregate is available throughout the state. The type of aggregate assumed is GP from the Corps of Engineers classification system.

From the soil classification and percent fines in the aggregate, TFPS calculates the modulus value of the base material. A value of 207 MPa (30,000 psi) is used for all the structures and mixtures.

Subgrade

As mentioned earlier, a soft subgrade was assumed: CH according to the Unified Soil Classification System. A plasticity index of 25 and modulus of 55 MPa (8000 psi) at 21°C (70°F) was assumed for the analysis.

3.2 Discussion of Results

TFPS was run with 9 laboratory mixtures and 1 field mixture. To identify the distress mode for a given structure, thicknesses were varied to meet all the design requirements with all the mixtures. The same procedure is repeated with the other 4 structures and in both climates. The comparisons of the predicted performance of the mixtures were on the basis of the amount of time until the first overlay was needed or the number of repetitions until the first overlay. Appendix B tabulates the results of the TFPS analysis.

Analysis of Structure I

The layer thicknesses used in the analysis of Structure I are shown below in Table 11.

Table 11. Thickness of Various Layers for Structure I.

Layer	Jefferson County Layer Thickness (mm)	Armstrong County Layer Thickness (mm)
Thin Surface Layer	64	64
Asphalt Treated Base	305	254
Soft Subgrade	semi-infinite	semi-infinite

From Figure 7, for Jefferson county, the number of ESALs to the first overlay is directly proportional to the resilient modulus of the mixtures at 21°C (70°F). As mentioned earlier, resilient modulus is the only mixture parameter that is an input for this program. The predicted performance follows the conventional wisdom that the higher the modulus, the lower the rutting potential. In the case of Jefferson county, because of the hot-wet climate, the primary distress mode is rutting. Also, as stiffness increases, the cracking potential also increases. But the results indicate that cracking is not critical for the performance in this climatic condition.

Now consider the same structure in Armstrong county (cold-dry). Even though the primary distress mode is rutting, the structure needs 51 mm (2 inches) less base to carry approximately the same number of ESALs as in the case of Jefferson county. This reduction in base thickness is due to the reduced accumulated damage during winter months. Overall performance of the mixtures is inversely proportional to the modulus value, which is opposite to the situation in Jefferson county. This is because the lower the stiffness, the lower the potential to crack. Even though the primary distress mode is rutting, the combined effect of the fatigue cracking, thermal cracking, and rutting contribute to the pavement reaching critical levels of distress and reduction in serviceability index.

Analysis of Structure II

The layer thicknesses used in the analysis of Structure II are shown below in Table 12.

Table 12. Thickness of Various Layers for Structure II.

Layer	Jefferson County Layer Thickness (mm)	Armstrong County Layer Thickness (mm)
Thin Surface Layer	102	102
Asphalt Treated Base	279	229
Soft Subgrade	semi-infinite	semi-infinite

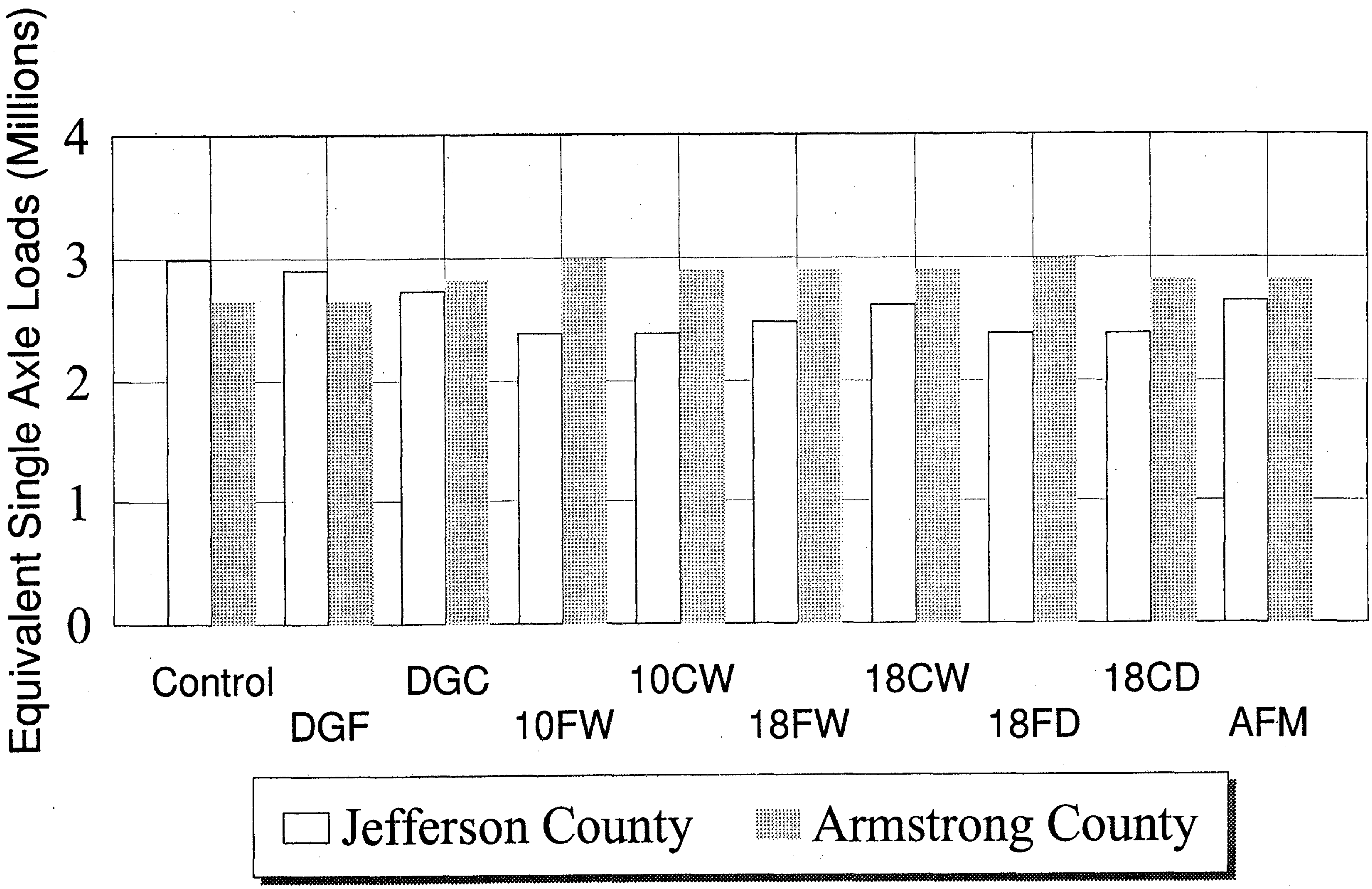


Figure 7. Summary of TFPS Results for Structure I.

From Figure 8, it can be seen that the performance of this structure follows the same explanation as for Structure I for both climatic conditions. In addition, some other observations can be made. A reduction of 51 mm (2 inches) in the base is observed to carry approximately the same number of ESALs for Structure II across both climatic zones. By increasing the surface layer thickness 38 mm (1.5 inches), a reduction of only 24 mm (1 inch) in base is observed in Jefferson county whereas a 51 mm (2 inch) reduction is observed in Armstrong county. The time to the first overlay can be delayed by 2.5 years in Jefferson county and 1.25 years in Armstrong county by increasing the surface layer thickness. This would be very useful in calculating life-cycle costs.

Analysis of Structure III

The layer thicknesses used in the analysis of Structure III are shown below in Table 13.

Table 13. Thickness of Various Layers for Structure III.

Layer	Jefferson County Layer Thickness (mm)	Armstrong County Layer Thickness (mm)
Thin Surface Layer	64	51
Asphalt Treated Base	279	279
Granular Base Course	203	152
Soft Subgrade	semi-infinite	semi-infinite

Addition of the granular base course along with the asphalt stabilized base does not alter the primary distress mode, which is rutting. Adding a 203-mm (8-inch) granular base decreased the thickness of the black base by only 25 mm (1 inch). In Armstrong county, the thickness of the black base increased by 25 mm (1 inch), and the surface layer thickness decreased by 12 mm (1/2 inch). Because of the modular ratio and the characteristics of the granular base, as expected, there is not considerable improvement

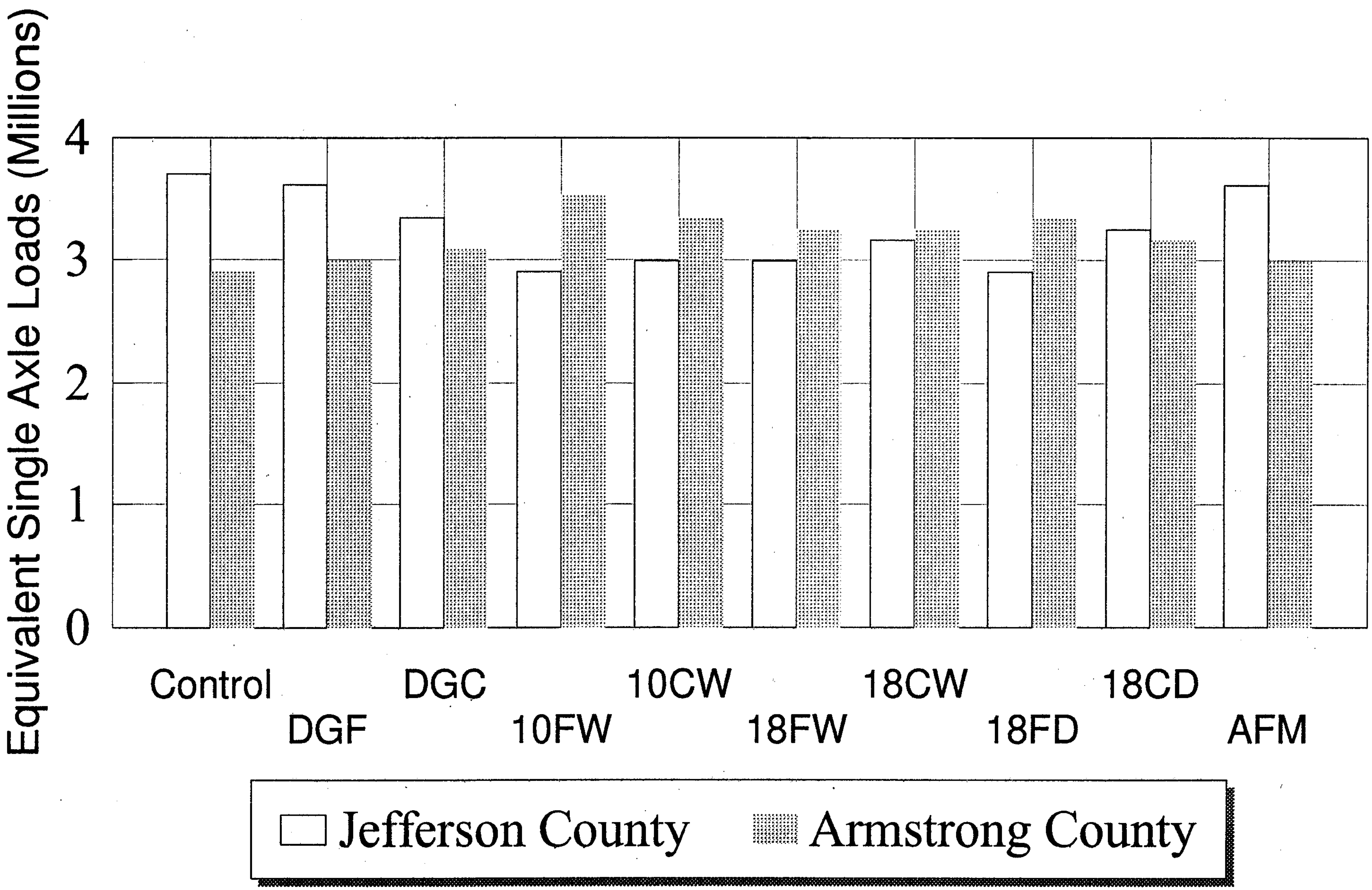


Figure 8. Summary of TFPS Results for Structure II.

in the structural capability and overall performance of the pavement; however, some improvement is observed in Armstrong county. As shown in Figure 9, the general trend in overall performance follows the same explanation as Structure II. At this point it is interesting to note that even in Jefferson county, performance is inversely proportional to the modulus which is a deviation from Structures I and II. This can be attributed to the addition of the granular base course.

Analysis of Structure IV

The layer thicknesses used in the analysis of Structure IV are shown below in Table 14.

Table 14. Thickness of Various Layers for Structure IV.

Layer	Jefferson County Layer Thickness (mm)	Armstrong County Layer Thickness (mm)
Thick Surface Layer	64	51
Asphalt Treated Base	279	279
Granular Base Course	203	152
Soft Subgrade	semi-infinite	semi-infinite

The primary distress mode for this structure is rutting, with an increase of 38 mm (1.5 inch) for the surface layer and a reduction of 25 mm (1 inch) in black base for Jefferson County. A reduction of 76 mm (3 inches) of black base is observed for Armstrong county. There is no comparable improvement in the performance by increasing the thickness of the surface layer in either climatic conditions. The explanation for the performance of this structure follows that of Structure III. The number of ESALs to the first overlay for CRM mixtures is shown in Figure 10.

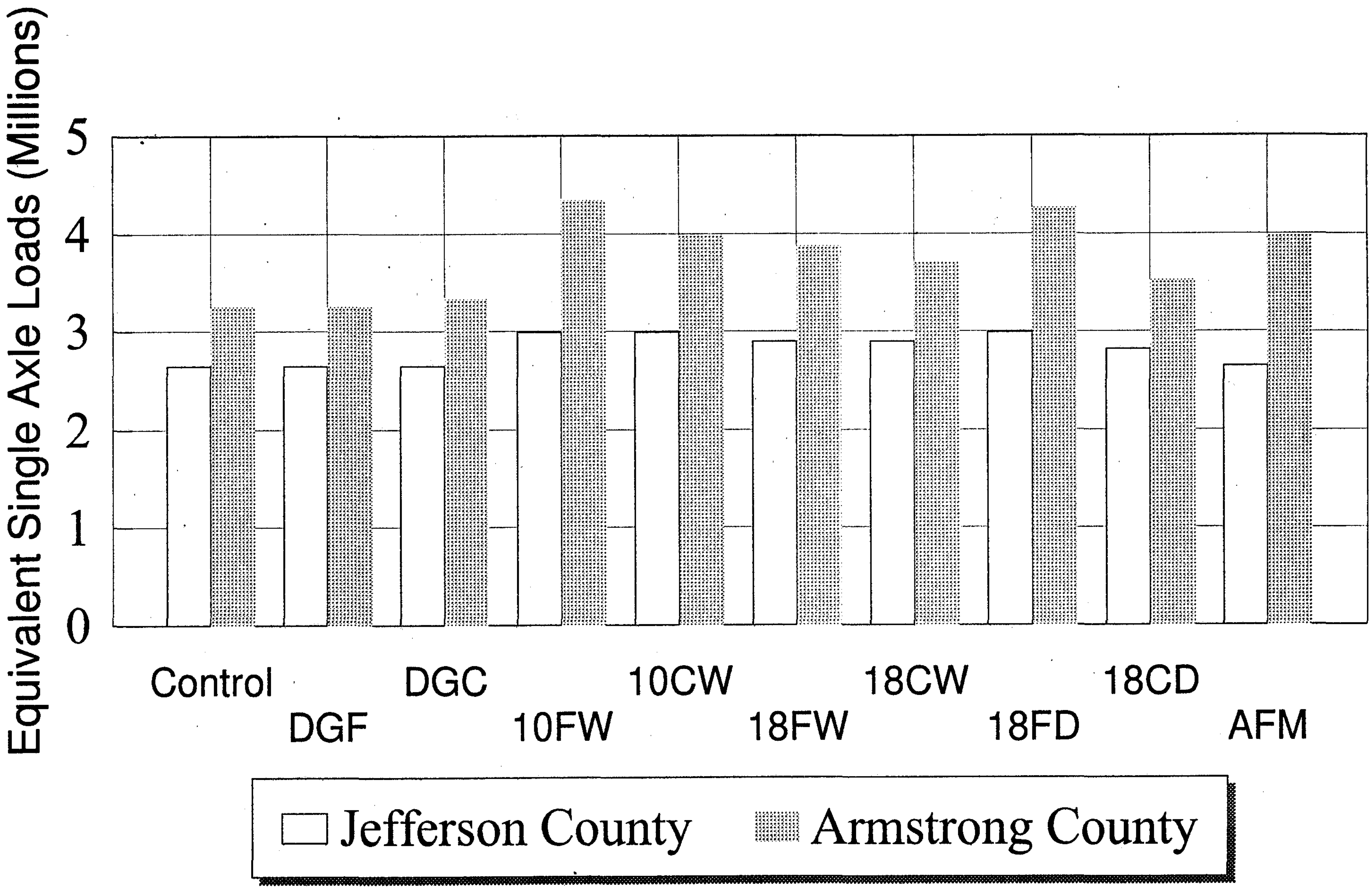


Figure 9. Summary of TFPS Results for Structure III.

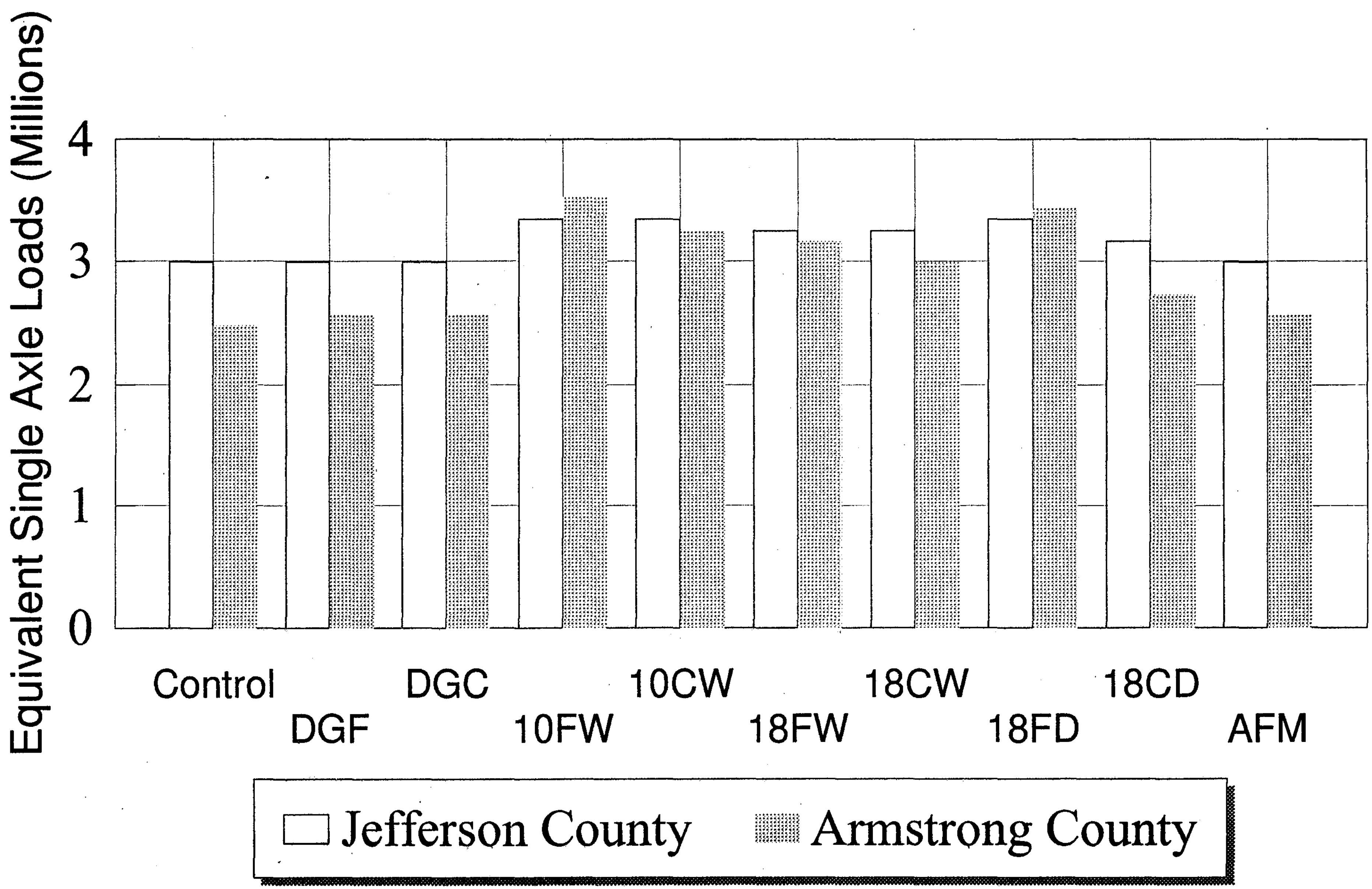


Figure 10. Summary of TFPS Results for Structure IV.

Analysis of Structure V

The layer thicknesses used in the analysis of Structure V are shown below in Table 15.

Table 15. Thickness of Various Layers for Structure V.

Layer	Jefferson County Layer Thickness (mm)	Armstrong County Layer Thickness (mm)
Thick Surface Layer	203	178
Granular Base Course	305	305
Soft Subgrade	semi-infinite	semi-infinite

The primary distress mode for this structure is fatigue cracking. This is a deviation from the other 4 structures and 2 climatic zones. This is due to the modular ratio of the surface layer to the granular base and the strain at the bottom of the asphalt concrete surface layer. Design requirements can be met with 25 mm (1 inch) less thickness in Armstrong county. This is because of the higher stiffness of the base and surface layer in winter, reducing the strain at the bottom of the asphalt concrete layer and improving the overall performance of the pavement. As shown in Figure 11, in both climatic regions, performance is directly proportional to the modulus of the mixture.

3.3 Summary

After a thorough understanding of the TFPS program along with its inherent limitations, the following conclusions can be drawn with respect to predicting the in-place performance of the dense-graded control mixture, the 8 laboratory CRM mixtures and the one field CRM mixture:

- Mixtures with higher stiffness values perform better in hot-wet climates, and mixtures with lower stiffness perform better in cold-dry climates.

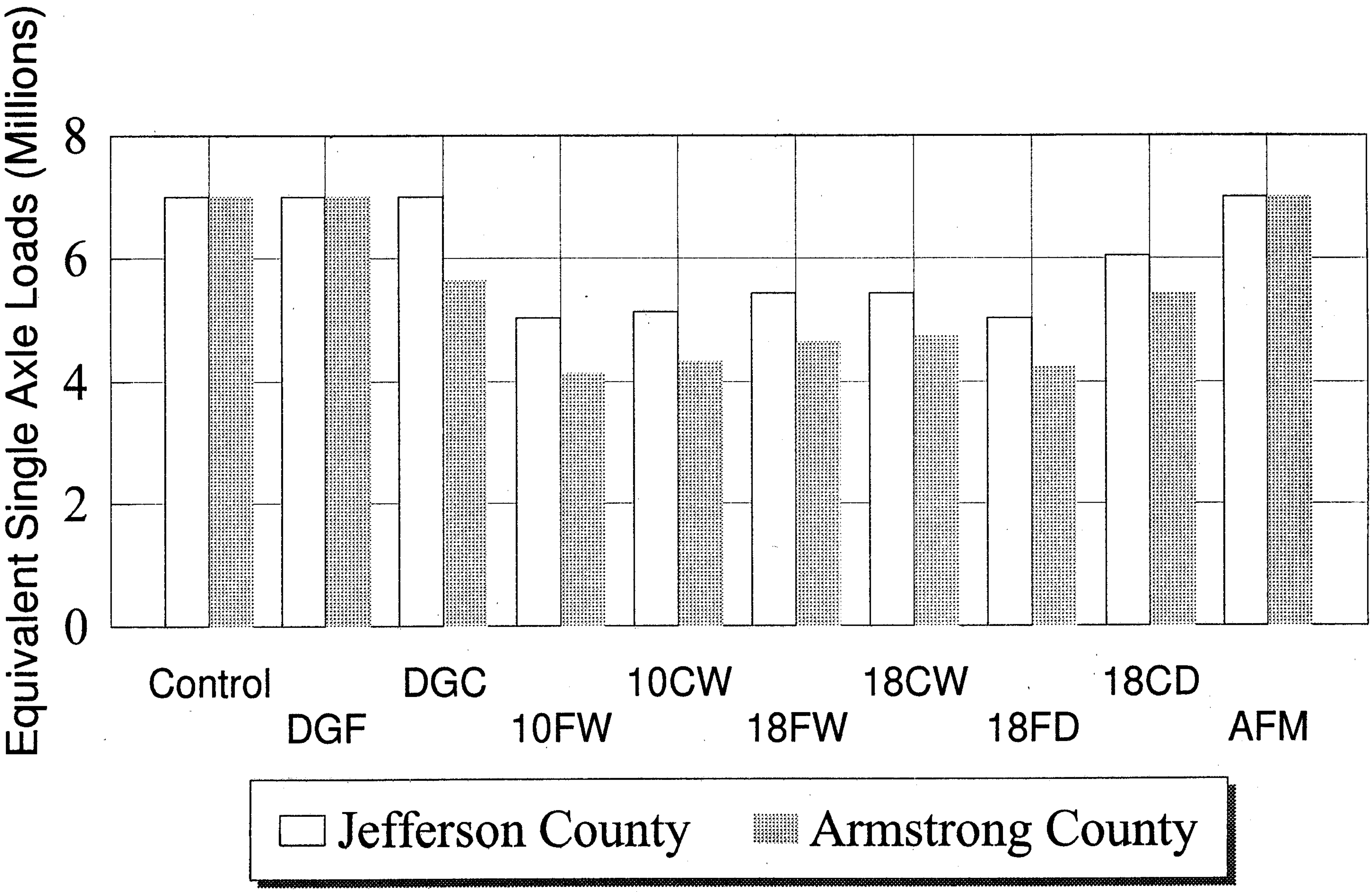


Figure 11. Summary of TFPS Results for Structure V.

- Besides modulus, support conditions have a great importance in predicting the in-place performance of the mixture. Rutting occurs if stiffness of the base is greater than the stiffness of the surface layer. Therefore, exercise caution when using CRM mixtures over asphalt-treated bases. This is particularly true in hot-wet climates.
- Structures with granular bases (only) will yield better performance with stiffer mixtures. CRM mixtures are not suggested for use with granular bases, if other alternatives are available.
- Some caution should be exercised when interpreting TFPS results because of the unusual characteristics of the CMHB crumb rubber mixtures. These mixtures have a higher film thickness which should provide better resistance to fatigue and thermal cracking, and the coarse stone matrix should resist rutting.

4

Use of Dynamic Shear Rheometer for Measurement of CRM Binder Viscosity

4.1 Introduction

Viscosity is an important specification in the use of asphalt cement for pavements. It depends upon a number of factors such as the molecular structure of asphalt, the temperature, and the type of filler added. The zero shear viscosity (η_o), or steady state viscosity, is the viscosity of the material at low enough shear rate such that the behavior is Newtonian, i.e., shear rate (or frequency) independent. This is a critical parameter because the viscosity is structure dependent at low shear rates, and there is less difference in the flow behavior at high shear rates. Hence, it is not possible to differentiate between different materials at high shear rates as would typically be observed in a capillary rheometer, for instance. Figure 12 shows the typical flow behavior of the different types of asphalts.

The most common techniques used today to obtain η_o are either by dynamic or steady shear tests in a rheometer. Due to the limitation of the instrument in reaching very low shear rates or frequencies, η_o may not be attainable within a reasonable time period. This problem is more significant in the case of aged or filled asphalt specimens. Previous researchers have chosen an arbitrary frequency and compared the viscosities at the frequency or shear rate. This is not a recommended procedure because of the inconsistencies that would follow in choosing a shear rate. An alternative method is a creep test where the stress is maintained constant for a period of time and the deformation is monitored till a steady flow state is reached and the viscosity calculated from the data.

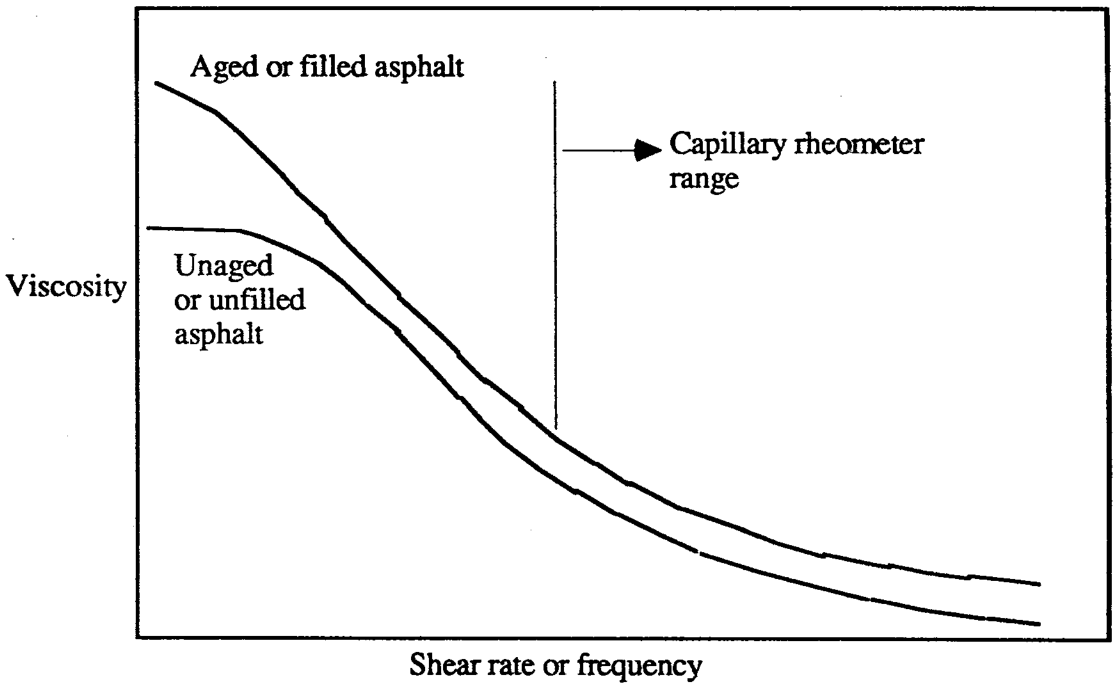


Figure 12. Typical Viscosity vs. Shear Rate (Frequency) Curves for Asphalt Binders.

Early researchers (Mamlouk 1984; Piazza 1980; Gaskins 1960) have used simple experimental setups to measure the creep response of asphalt. Though they were able to perform the test with reasonable accuracy, the tests were not up to research grade. With the increasing use of rheometers and the definite advantages due to accuracy, the theory and testing behind the calculation of η_o should be understood better. The objective of this portion of the study was to perform dynamic and creep rheological tests on the base asphalt and CRM binders using a stress controlled rheometer and to develop a suitable technique for measuring viscosity of these binders.

4.2 Theory

Asphalt has been proven to be of viscoelastic nature, and the phenomenological theory of viscoelasticity can be applied to asphalt (Goodrich 1988). Creep measurements provide a direct measurement of steady state viscosity. For a noncrosslinked viscoelastic material, the creep and recovery behavior can be modeled in terms of a series of Voigt elements and a Maxwell element connected as shown in Figure 13. For an imposed stress at time t=0, the response of a viscoelastic material can be given by:

$$\frac{\gamma(t)}{\sigma_o} = J(t) = J_o + \int_o^\infty J(\tau)(1 - e^{-t/\tau})d\tau + \frac{t}{\eta} \tag{18}$$

Where

$\gamma(t)$	is the deformation as a function of time,
σ_o	is the constant stress applied at time t=o,
$J(t)$	is the creep compliance as a function of time,
J_o	is the instantaneous elastic compliance,
τ	is the retardation time,
$J(\tau)$	is the distribution of retardation times, that is, the compliance associated with retardation times between τ and $\tau + d\tau$, and
η	is the viscosity associated with steady state flow of the material.

When the constant stress is removed after a sufficiently long time, elastic recovery takes place. The equation during recovery can be given as:

$$\frac{\gamma(t)}{\sigma_o} = J(t) = \int_o^\infty J(\tau)e^{-t/\tau}d\tau + J_o \tag{19}$$

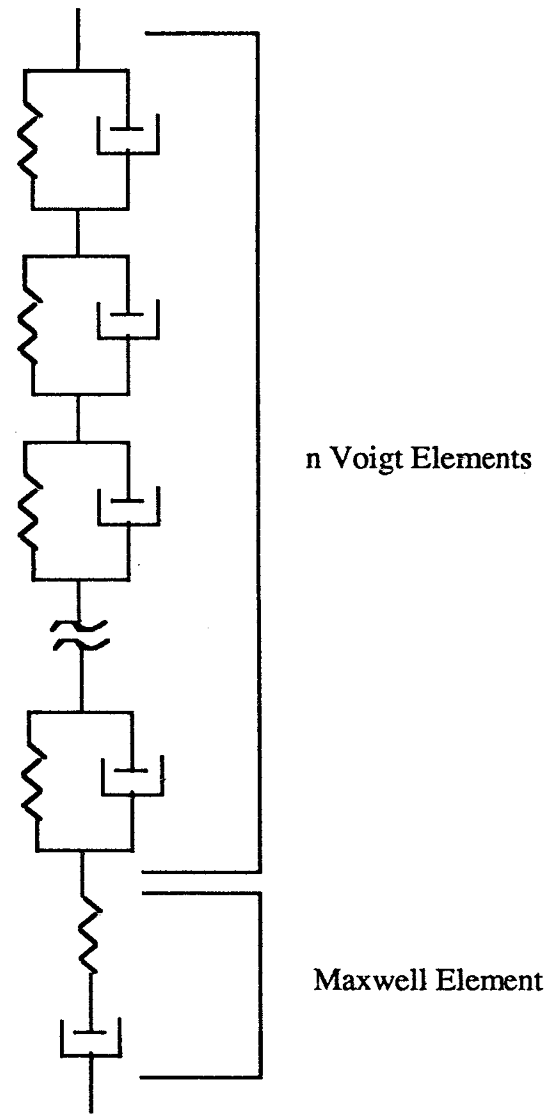


Figure 13. Schematic of Voigt Elements and Maxwell Element Connected In Series.

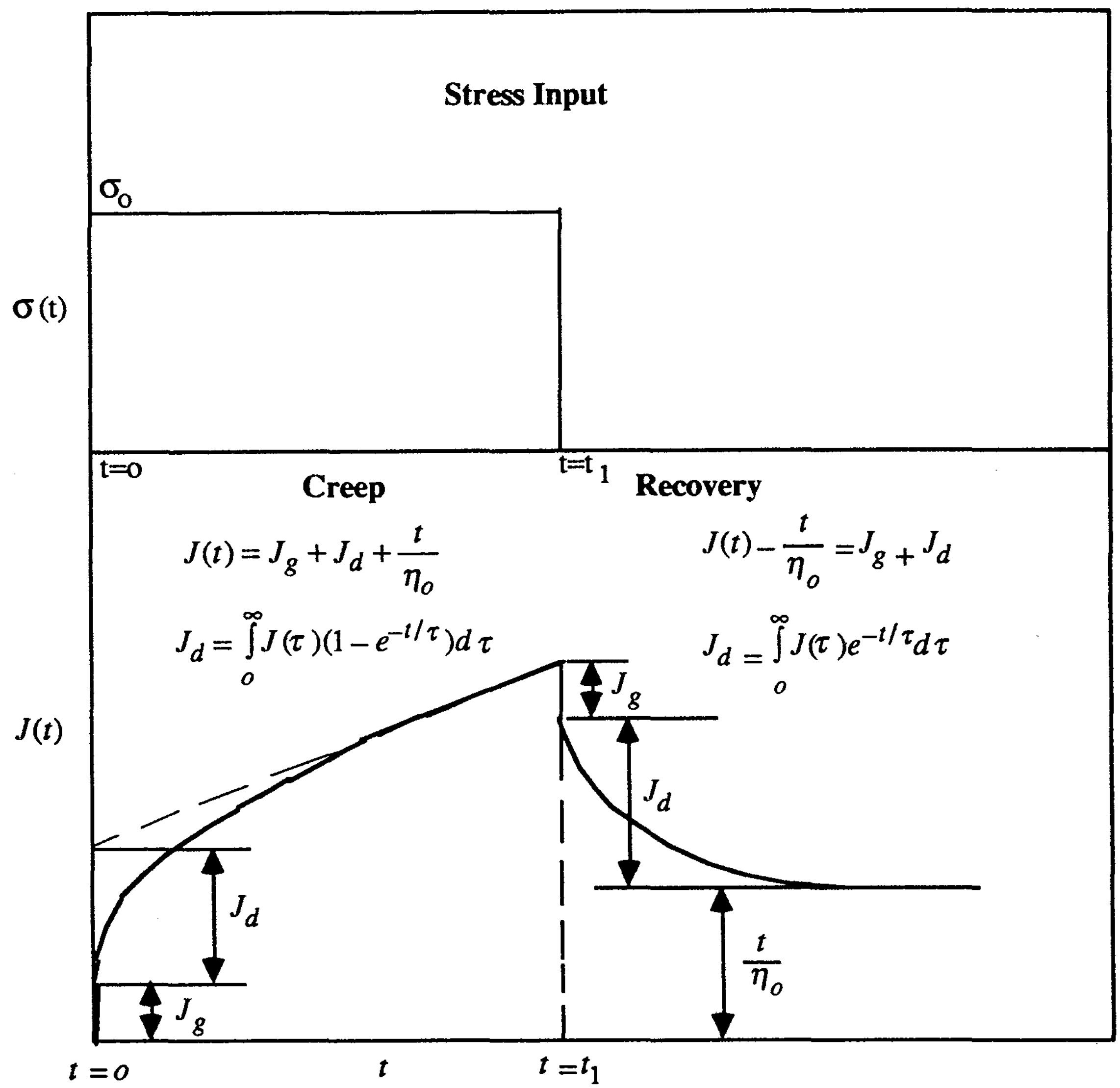


Figure 14. Creep and Recovery Response of a Viscoelastic Material.

The first term equals J_∞(steady state compliance) at long times when no more recovery is taking place.

The creep compliance consists of a glass compliance (J_g), a time dependent retarded elastic compliance (J_d), and Newtonian flow (η_o). The steady state viscosity (η_o) and the steady state compliance ($J_g + J_d$) can be determined from creep data, provided steady state flow has been attained. The slope of the linear portion of the creep data is the reciprocal of the viscosity. However, it is easy to be misled into believing prematurely that the linear portion of the creep has been reached. As a general rule, the linear portion is not attained until the flow term t/η_o is at least as large as the intercept, ($J_g + J_d$). An alternative way of finding η_o is to perform the elastic recovery experiment and the previous calculation can be confirmed. After the steady state condition is reached, as shown in Figure 14, t^1/η_o can be calculated. In this study, we are primarily concerned with the creep test and how η_o can be obtained from a creep test. Care should be taken to ensure that all the tests are performed within the linear viscoelastic region. This can be done by applying more than one stress, and the values of compliance and η_o obtained should be constant.

4.3 Materials and Experimentation

This research examined four different types of asphalt binders with Texaco AC10 as the base material: (1) unaged base asphalt, (2) SHRP Pressured Aging Vessel (PAV) aged base asphalt, (3) 4% Rouse or fine CRM asphalt, and (4) 18% Rouse or fine CRM asphalt. These materials were chosen to represent a wide range of steady state viscosities. The dynamic tests were performed in a Bohlin (DSR) rheometer with a parallel plate configuration. The asphalt sample is placed in between two 25 mm parallel plates and the gap is adjusted to 1 mm for the unaged, base asphalt and 2 mm for the PAV aged and CRM asphalts. A sinusoidal strain is imposed, and the stress response is measured. All the tests were conducted in the linear viscoelastic range by performing a strain sweep at each temperature. The range of frequencies chosen were from 0.1 Hz to 10 Hz at each temperature. The creep tests were performed in a Bohlin (DSR) rheometer equipped with a creep test capability in the parallel plate configuration. A constant stress is imposed on the sample instantly and maintained for about 300 seconds to 1000 seconds, depending on the time taken for the material to approach steady state. The deformation is recorded during that time which can be later analyzed to obtain the steady state viscosity. The creep tests were done at 25°C and 60°C. Two constant stresses were imposed for each sample at each temperature.

4.4 Results and Discussion

The dynamic data collected over a temperature range of 10°C to 90°C were shifted to a reference temperature of 25°C to obtain a master curve. The master curves for each sample are shown in Figures 15-18. The complex or dynamic viscosity, η^*, can be calculated from these master curves using the relationship:

$$\eta^* = \frac{|G^*|}{\omega} = \frac{\sqrt{G'^2 + G''^2}}{\omega} \tag{20}$$

Figure 19 shows the viscosity vs. frequency curves for all the asphalts. A zero shear viscosity is observed for the base asphalt, but for the aged and CRM asphalts, the data is required at even low frequencies to notice any steady state viscosity. A possible solution to this problem is to measure the storage and loss modulus at a higher temperature (in our case, above 90°C) and shift the data to very low frequencies. But this was not possible as an increase in temperature reduced the viscosity, and the asphalt was forced out of the gap between the parallel plates.

Let us move on to the creep test results and look at the flow behavior. In Figure 20, the creep compliance as a function of time is shown for the unaged asphalt for an imposed stress of 10,000 Pa at 25°C. Figure 21 shows the creep data for 18% Rouse CRM asphalt for a constant stress of 1000 Pa at 25°C. Here it can be seen that it takes longer to reach the steady state region. The stress chosen is based on the range

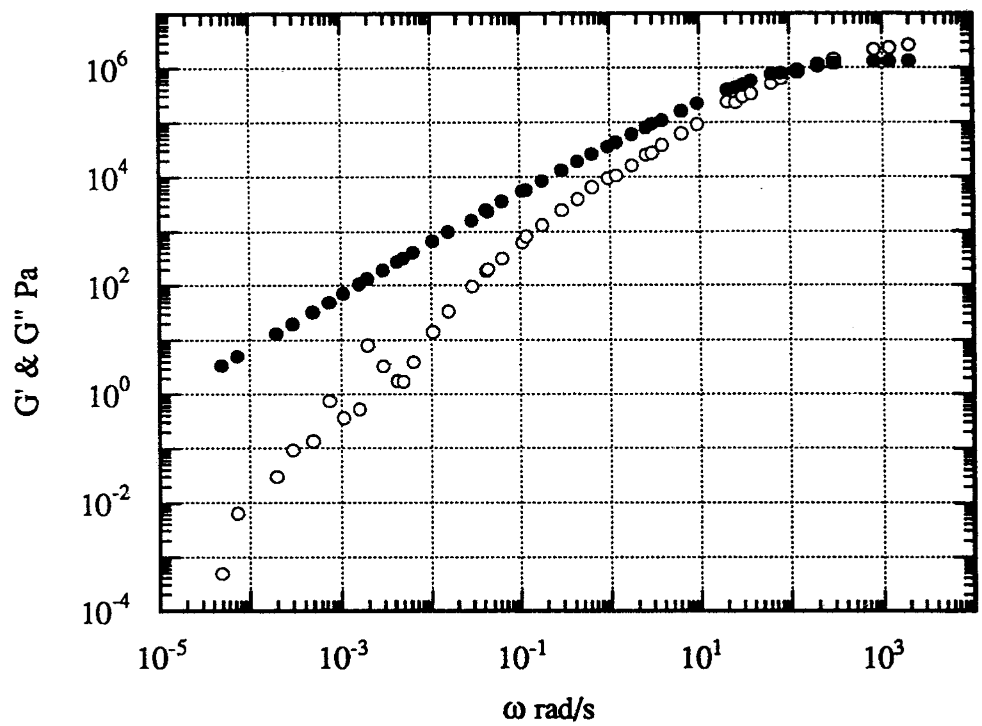


Figure 15. Master Curve of Base Asphalt Shifted to a Reference Temperature of 25°C.

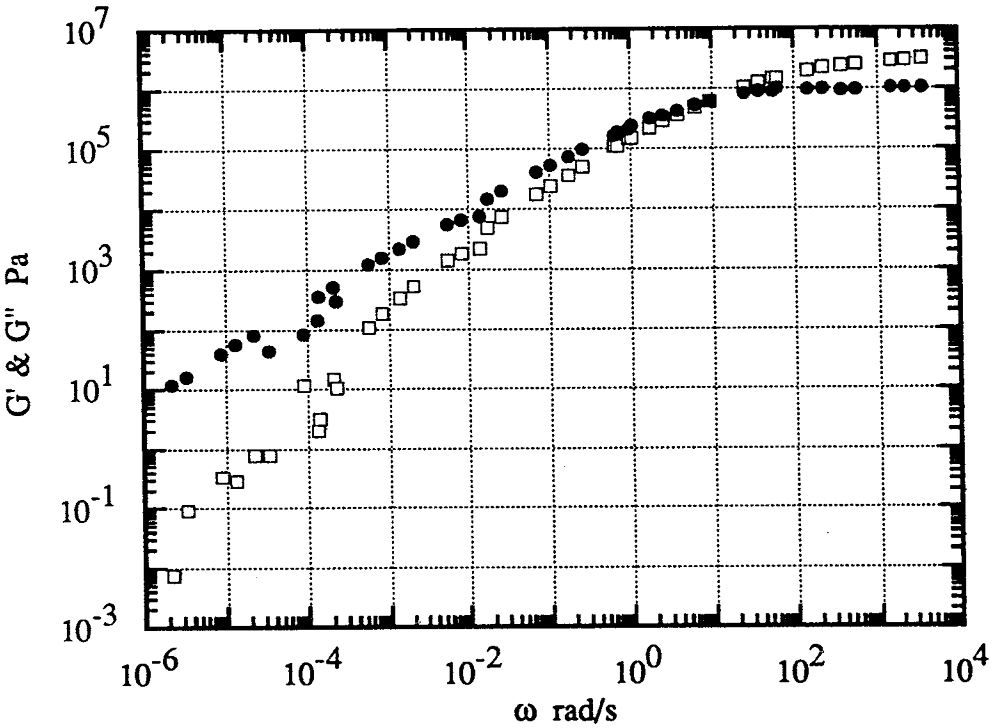


Figure 16. **Master Curve of PAV-Aged Asphalt Shifted to a Reference Temperature of 25°C.**

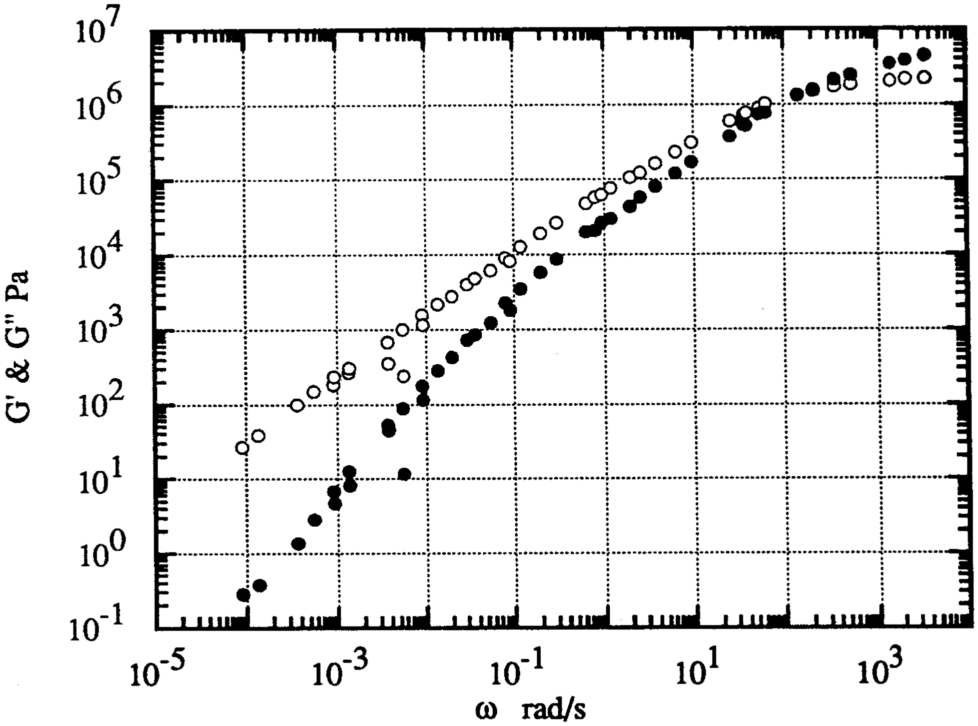


Figure 17. **Master Curve of 4% Rouse (Fine) CRM Asphalt Shifted to a Reference Temperature of 25°C.**

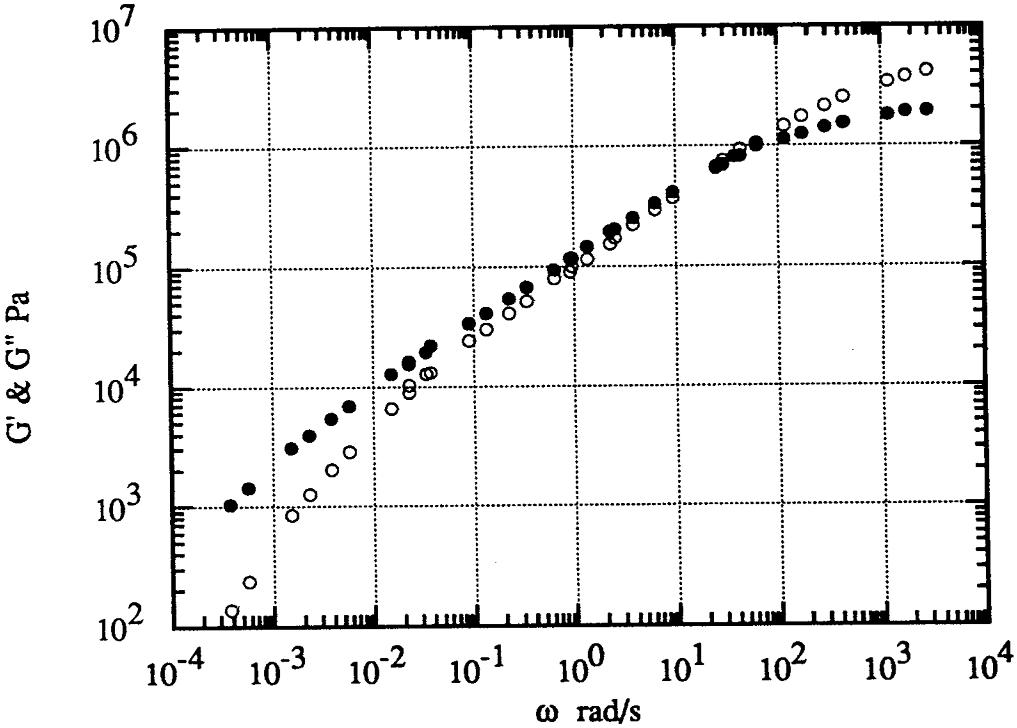


Figure 18. **Master Curve of 18% Rouse (Fine) CRM Asphalt Shifted to a Reference Temperature of 25°C.**

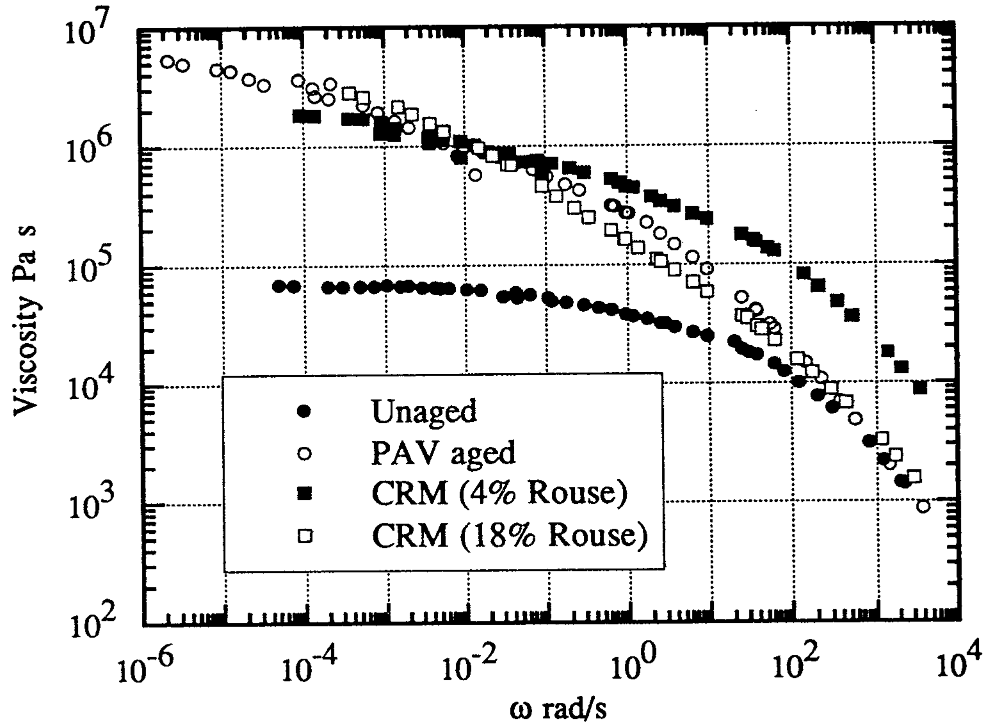


Figure 19. **Complex Viscosity Curves Calculated From Their Master Curves for the Different Types of Binders.**

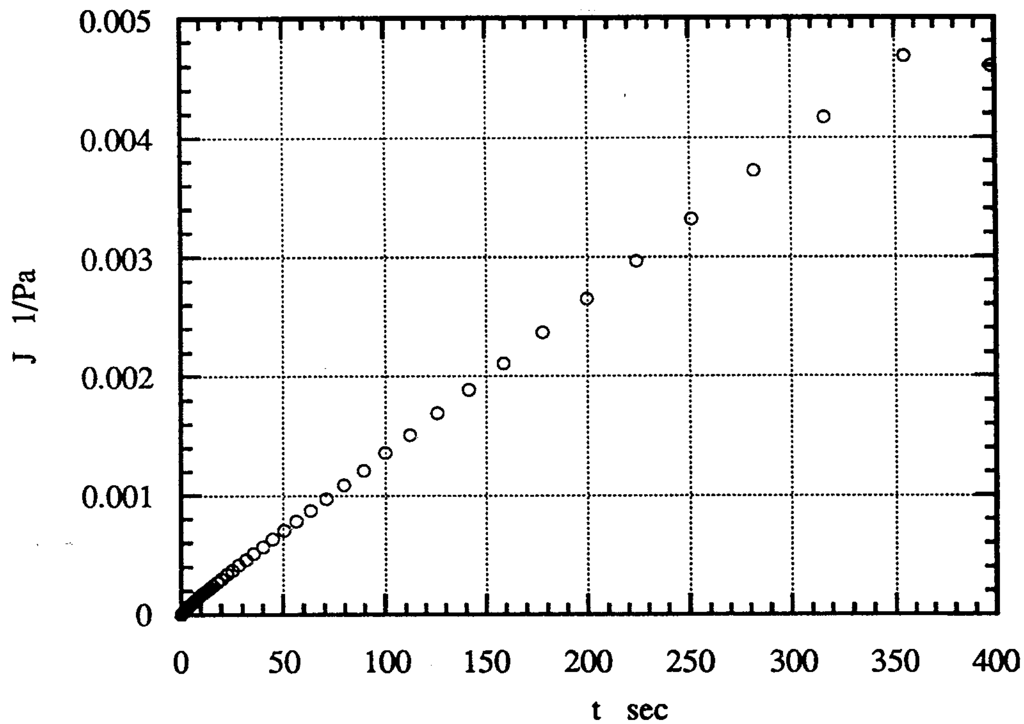


Figure 20. Creep Data for Base Asphalt at 25°C for a Constant Stress of 10000 Pa.

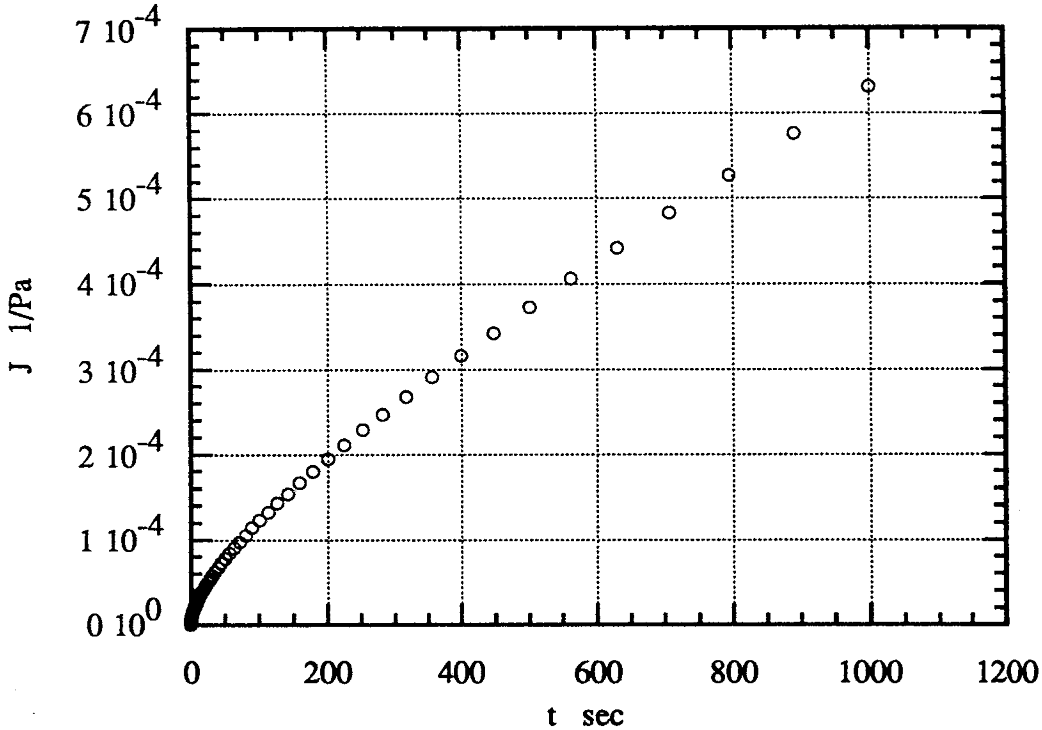


Figure 21. **Creep Data for 18% Rouse CRM Asphalt at 25°C for a Constant Stress of 1000 Pa.**

determined by the instrument based on its capability. This, in turn, is dependent on the gap size set between the parallel plates. Care should be taken in choosing the stress so that the test is performed within the viscoelastic limits. Two stresses were chosen for each sample at each temperature, one closer to the lower limit and the other closer to the higher limit. Steady state flow is reached when $m = d\ln(J)/d\ln(t) = 1$. In practice, as m gets closer to 1, the longer it takes to reach unity. The instrument continuously calculates the value of m and is shown on the computer screen. For $m = 0.7$ to 0.9, which is far removed from the steady state region, a conventional method developed by Ninomiya (Ninomiya, 1963) can be used here to calculate η_o. Creep compliance in shear at the steady state, denoted by J(t) can be written in the form:

$$J(t) = J_e + t/\eta_o \tag{21}$$

where t is the time. This equation can be written in the alternate form:

$$J(t)/t = J_{e/t + 1/\eta}O \tag{22}$$

Therefore, when J(t) is plotted against 1/t, the intercept at the ordinate gives the value of $1/\eta_o$. An example calculation for creep data in Figure 20 is shown in Figure 22 where a straight line fit is done for data with 1/t tending to zero. The η_o found here is 7.78E4 Pa s, and it closely matches with the η_o value of 7.31E4 Pa s found from the complex viscosity data in Figure 19. Table 16 shows the viscosities calculated at both 25°C and 60°C for each sample. The viscosity and m values at the end of the creep tests for the various asphalts are shown in the Appendix C.

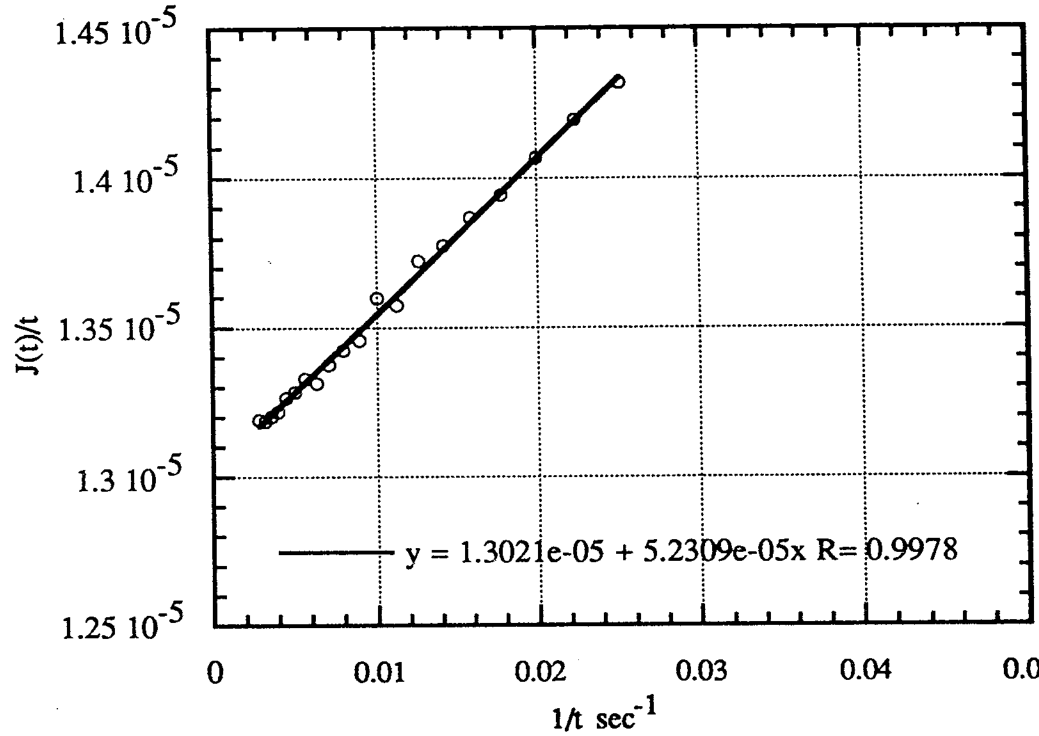


Figure 22. **Example Curve of J(t)/t vs. t at 25°C (for Base Asphalt) Closer to the Ordinate Axis to Find the Intercept.**

Table 16. Zero Shear Viscosities at 25°C and 60°C Calculated From Creep Data.

Binder	25°C			60°C		
	Stress (Pa)	Viscosity (Pa s)	$\frac{d\ln J(t)}{d\ln(t)}$	Stress (Pa)	Viscosity (Pa s)	$\frac{d\ln J(t)}{d\ln(t)}$
AC-10	2000 10000	7.677E4 7.780E4	0.981 0.983	500 1000	1.388E2 1.347E2	0.980 0.981
AC-10 (PAV)	1000 2000	1.623E6 1.787E6	0.832 0.852	200 500	1.319E2 1.162E2	0.981 0.981
4%CRM Binder	1000 2000	1.619E5 1.484E5	0.985 0.982	200 500	2.920E2 2.672E2	0.980 0.981
18%CRM Binder	1000 2400	1.953E6 1.733E6	0.746 0.818	200 500	3.420E3 2.790E3	0.980 0.982

In summary, η_o is a critical parameter which should be used as an asphalt grade specification. The creep test in a dynamic shear rheometer is shown in this study as an accurate and powerful technique to calculate η_o for asphalt and crumb rubber modified asphalt binders. The longer time taken to reach a steady state can be avoided by extrapolating the available data in the m = 0.7 to 0.9 region. Further research should be done on the effects of constant stress on the calculations. This would help provide a better understanding of the linear viscoelastic limits of the material. Recovery tests should also be done along with the creep tests to corroborate the information obtained from the creep tests.

5

Field Evaluation

5.1 Introduction

Three CRM asphalt concrete construction projects were evaluated during this study:

Abilene Project	SH 36 in Callahan County,
Lufkin Project	SH 63 in Angelina County, and
Nacogdoches Project	US 259 in Nacogdoches County.

All three projects were very similar in terms of the mixture design. They were 38 to 50 mm (1½ to 2-inch) thick overlays; the wet process was used in all three projects for incorporation of the CRM; and the type of mixture used was a coarse-matrix, high-binder (CMHB) mix. Other specific information regarding each project is shown in Table 17. The Abilene project was constructed in the fall of 1993 and has, therefore, been in service for one year at the time of this report. The Lufkin and Nacogdoches jobs were constructed in late summer of 1994 and have had very little service life as of this report.

Samples of the plant mix were obtained for the Abilene and Lufkin projects, and AAMAS tests and TxDOT creep tests were performed. Results are presented in this chapter. Information obtained during these construction projects was used for development of the quality control and construction guidelines discussed in Chapter 7.

Table 17. **Mixture Design and Other Job Information for CRM Construction Projects.**

Job Information	Abilene Project	Lufkin Project	Nacogdoches Project
Crumb Rubber Modified Binder Supplier	Duininck Brothers	International Surfacing, Inc.	International Surfacing, Inc.
CRM Supplier	Granular Products	Granular Products	Granular Products
CRM Type	Type II	Type II	Type II
CRM Content, % by weight of binder	17	15	15
Top Size Aggregate	19 mm (3/4")	19 mm (3/4")	19 mm (3/4")
Total Aggregate Retained on No. 10 Sieve	85.3	82.8	79.2
Total Aggregate Passing No. 200 Sieve	5.0	6.0	5.2
Binder Content, % by weight of mix	8	7.3	7.5
Voids in Mineral Aggregate (VMA), %	20.0	18.4	18.3

5.2 Short-Term Pavement Performance

The Abilene pavement has been in service for one year at the time of this report and is performing very well with no signs of distress. The Lufkin and Nacogdoches pavements were only recently constructed but no significant problems were observed during construction.

5.3 Laboratory Evaluation of Abilene and Lufkin Field Mixtures

Samples of the field mix were obtained from the plant in both Abilene and Lufkin and were brought back to TTI's laboratory for further testing. The materials were reheated and compacted to test according to the AAMAS testing program and the TxDOT Tex 231-F static creep tests. Chapter 2 presented the Tex 231-F creep data. Results of the AAMAS tests are summarized below in Figures 23 through 25 and a complete tabulation of the laboratory data is contained in Appendices D and E.

Figure 23 shows the rutting potential of the two field mixtures. The Lufkin mixture appears to be slightly more rut-resistant than the Abilene mix; however, both mixtures are considered to have low to moderate rutting potential.

Figure 24 shows two relationships between the total resilient modulus and indirect tensile strain at failure for a standard mixture (dense-graded asphaltic concrete mixture placed at AASHTO Road Test). The difference is that the NCHRP 1-10B assumed a constant slope of the fatigue curves, whereas the FHWA study varied the slope of the fatigue curves.

If the total resilient modulus and indirect tensile strains at failure for a particular mixture plot above the standard mixture (FHWA fatiguer curve is recommended), it is assumed that the mixture has better fatigue resistance than the standard mixture. The Lufkin mixture shows better fatigue resistance than the Abilene mixture; however, based on other tests performed in this study, both of these mixtures should be considered fatigue resistant. They are significantly more fatigue resistant than most of the laboratory mixtures evaluated earlier in this study as described in the previous report 1332-1.

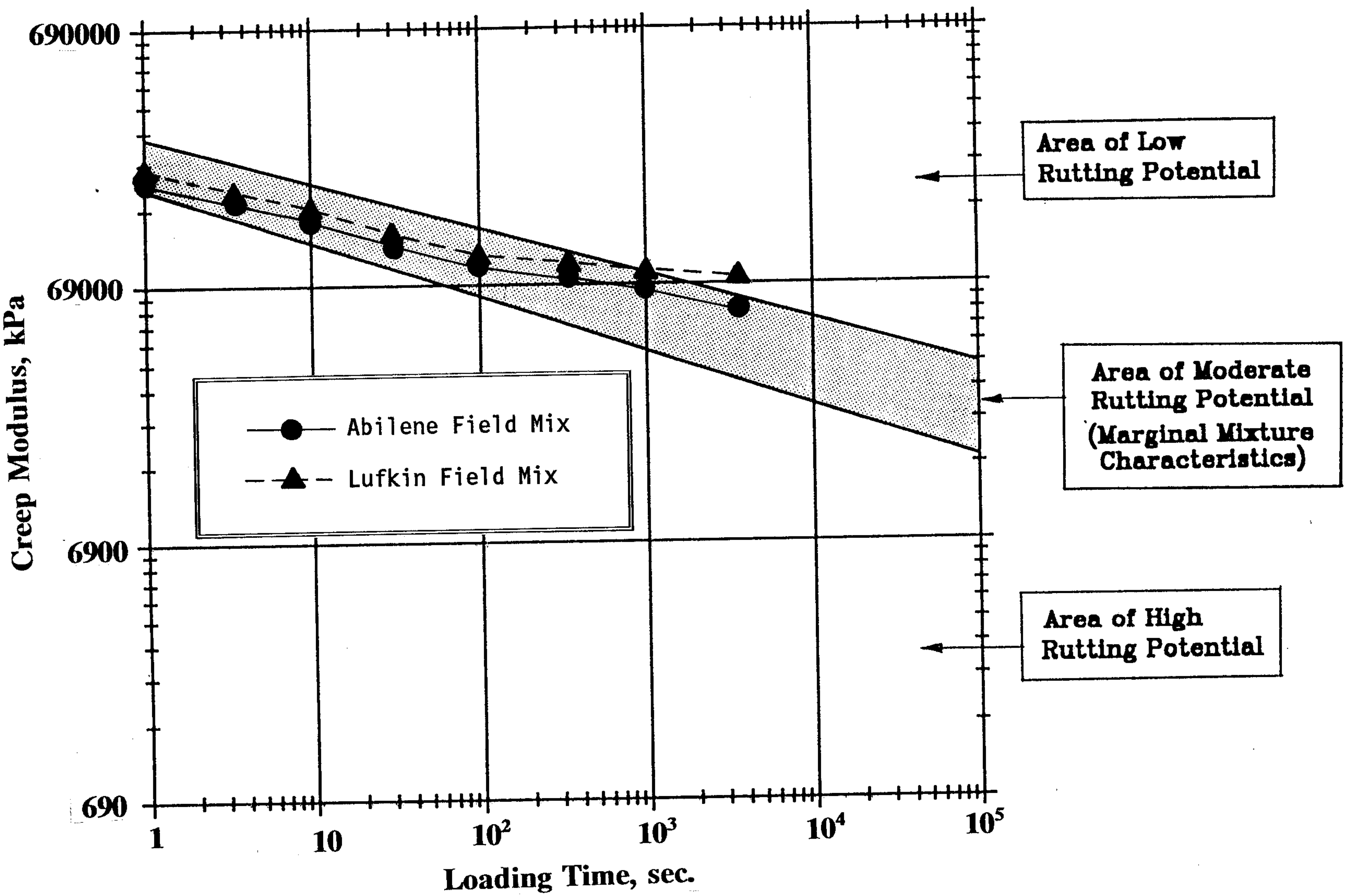


Figure 23. Asphaltic Concrete Mixture Rutting Potential for Abilene and Lufkin Field Mixtures.

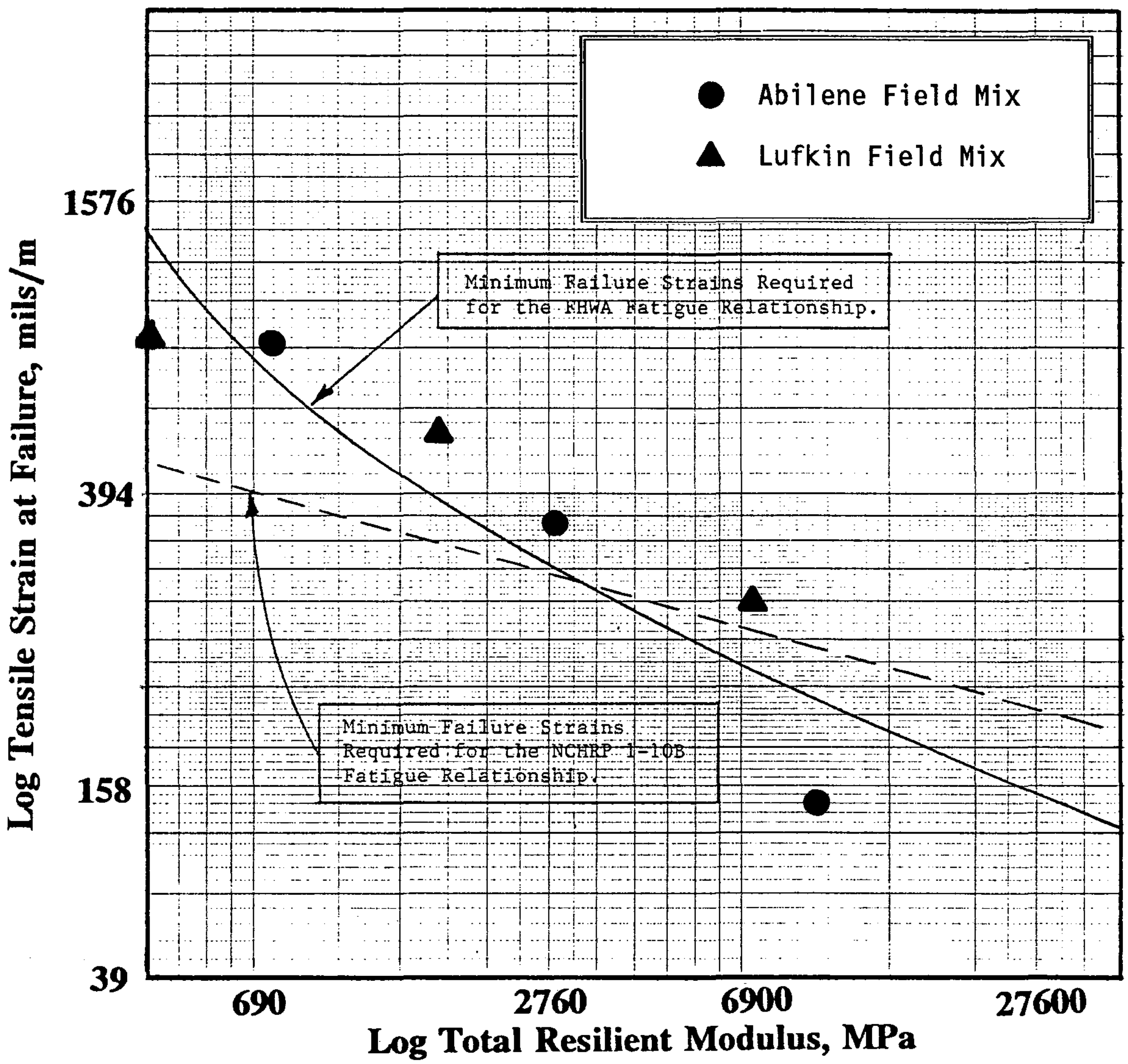


Figure 24. Relationship between Indirect Tensile Strains and Resilient Modulus for Abilene and Lufkin Field Mixtures.

Diametral resilient modulus tests were performed at 41°F (5°C), 77°F (25°C) and 104°F (40°C). These data are shown in Figure 25. The Abilene mixture appears to be stiffer than the Lufkin mix, particularly at higher temperatures.

Moisture damage is caused by a loss of adhesion or bond between the asphalt and aggregate in the presence of moisture. The moisture damage evaluation (tensile strength ratio) is used as a means of accepting or rejecting a mix. This value should exceed a value of 0.80. Both Lufkin and Abilene mixtures had tensile strength ratios greater than 0.85.

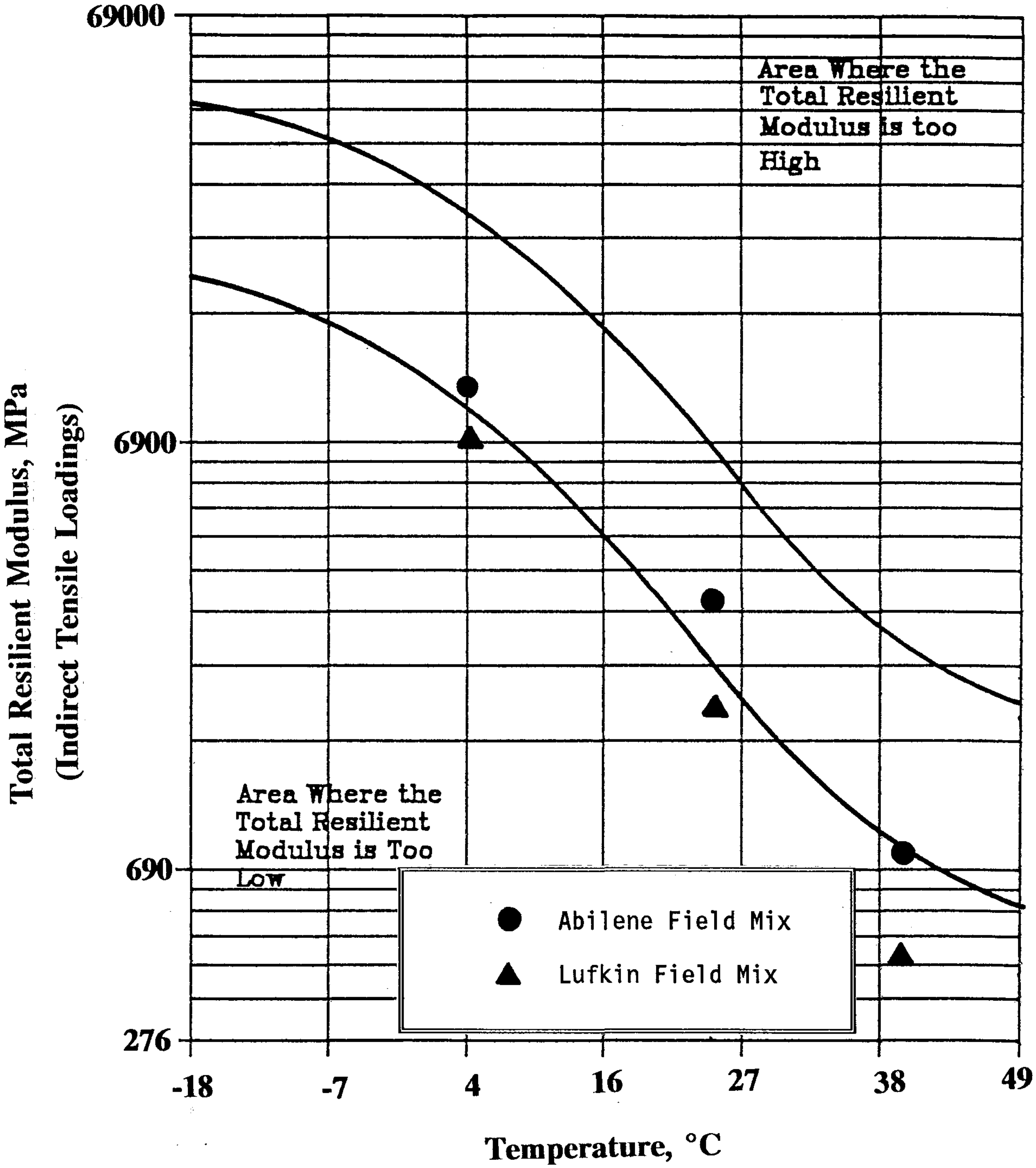


Figure 25. Total Resilient Modulus versus Temperature for Abilene and Lufkin Field Mixtures.

6

Draft Materials Specifications

6.1 Crumb Rubber Modified Mixture Evaluation

The following procedure is recommended for evaluation of the rutting potential of crumb rubber modified mixtures. This procedure is based on a careful review of the data in this study and developments of Little and Youssef (1992).

Fabrication of Samples

Fabricate samples for uniaxial creep testing in accordance with section 2.9 of NCHRP Report 338 using either the Gyratory Test Machine (GTM) (ASTM D 3387) or Gyratory Compactor (Tex 206-F).

If the GTM is used, the samples should be 101.6 mm (4 inches) in diameter and either 101.6 mm (4 inches) high or 203.2 mm (8 inches) high.

The Texas Gyratory Compactor (Tex 206-F) may be used if modified to fabricate samples at least 88.9 mm (3.5 inches) high. TTI was able to modify the Texas Gyratory in this manner.

Another alternative is the use of the large gyratory press as explained in Tex 126-E. This press produces a 152.4 mm (6 inch) diameter and 203.2 mm (8 inch) high specimen which can be tested as is for larger stone mixtures (maximum aggregate size larger than 25.4 mm (1 inch) or the sample can be cored to a 101.6 mm (4 inches) diameter and cut to a 101.6 mm (4 inch) height to meet AAMAS requirements for mixtures with maximum aggregate sizes of less than 25.4 mm (1 inch).

The reinforcing effect of short specimens reduces deformation potential; therefore, the criteria presented here would not be applicable to specimens shorter than 88.9 mm (3.5 inches) high.

Unconfined Creep Test

Perform the unconfined creep test in accordance with the procedure described in NCHRP Report 338, Section 2.9, paragraph 2.9.3.4 with the exception that the stress level applied to the sample should be determined based on Table 18.

Table 18. Suggested Uniaxial Creep Stress Levels.

Pavement Structure	Suggested Uniaxial Stress for Unconfined Compressive Creep Laboratory Test
Crumb Rubber Modified AC Overlay on AC Base	345 kPa (50 psi)
Crumb Rubber Modified AC Overlay on PCC Base	345 kPa (50 psi)
Crumb Rubber Modified AC Overlay on Flexible Base	483 kPa (70 psi)

Obtain a continuous read-out over the 3,600 second test period and plot the creep data on an arithmetic plot. The purpose of this plot is to identify tertiary creep if it exists during the one-hour creep loading period. The mixture should be rejected if it exhibits tertiary creep within the loading period.

The following criteria in Table 19 may be used for the evaluation of the creep stiffness data.

Table 19. Creep Stiffness Criteria at One-Hour Creep Loading.

Level of Rut Resistance	Traffic Intensity Level	Required Minimum Creep Stiffness for Test Constant Stress Level of:	
		345 kPa (50 psi)	483 kPa (70 psi)
Highly Rut Resistant	IV	121 MPa (17500 psi)	155 MPa (22500 psi)
	III	70.0 MPa (10000 psi)	96.5 MPa (14000 psi)
	II	44.8 MPa (6500 psi)	60.3 MPa (8750 psi)
	I	27.6 MPa (4000 psi)	41.4 MPa (6000 psi)
Moderately Rut Resistant	IV	70.0 MPa (10000 psi)	96.5 MPa (14000 psi)
	III	50.0 MPa (7250 psi)	70.0 MPa (10000 psi)
	II	41.4 MPa (6000 psi)	51.7 MPa (7500 psi)
	I	20.7 MPa (3000 psi)	27.6 MPa (4000 psi)

Notes:
I - Low traffic intensity: $< 10^5$ ESALs
II - Moderate traffic intensity: Between 10^5 and 5×10^5 ESALs
III - Heavy traffic intensity: Between 5×10^5 and 10^6 ESALs
IV - Very heavy traffic intensity: $> 10^6$ ESALs

6.2 Crumb Rubber Modified Binder Evaluation

There are generally four testing requirements which have been recently used by TxDOT for crumb rubber binder specifications: Haake viscosity, cone penetration, softening point and resilience.

Viscosity

Of all the binder tests evaluated in this study, viscosity appears to be one of the best indicators of the material's characteristics. As shown in this study previously, viscosity is very much dependent on the type and concentration of CRM used in the binder. It can also be dependent on the base asphalt used. Therefore, it is recommended (at this time) that specifications be based on the job mix formula of the binder used. Test protocol has been developed in this study for viscosity measurements of crumb rubber binders at the following temperatures:

25°C (77°F)	Use 1332 test protocol for dynamic shear rheometer,
60°C (140°F)	Use 1332 test protocol for dynamic shear rheometer,
177°C (350°F)	Use 1332 test protocol for Brookfield viscosity.

The Haake viscosity should be used for field quality control and specification requirements should also be based upon the job mix formula. See this report for a test protocol for Haake viscosity.

Softening Point and Penetration

Results of this study did not provide a clear link between mixture performance and either softening point or penetration. These two properties also can be very much dependent on the CRM type and concentration which should be kept in mind when specifying binder for a particular job.

Resilience

It is recommended that this test be eliminated as a specification requirement for crumb rubber binders. Based on the results of this study, resilience is not related to performance of the mixtures. In fact, some of the binders which appeared to provide a significant improvement in predicted performance had zero resilience as measured with this test. It should only be used to qualify a particular binder.

Other Binder Specification Considerations

As presented in previous report 1332-1, one particular binder seemed to provide a significant improvement in binder and mixture properties and predicted performance (in terms of cracking distress). This binder contained 18% fine CRM. We believe that, at a certain concentration of rubber particles in the wet process, a 3-dimensional network is created within the crumb rubber binder. For a given concentration of rubber, the smaller the rubber particles, the more particles there are per unit weight and the closer their mutual proximity in a crumb rubber asphalt system. It is this close proximity of the soft swollen particles that promotes the formation of the 3-dimensional network.

Previous research has shown that at approximately 5 to 6% neat styrene-butadiene styrene (SBS) rubber in asphalt, a 3-dimensional rubber network is generated which has a marked effect on rheological properties of the modified binder. Based on this fact and the data in this study, it is surmised that if tire rubber particles could be reduced to microscopic sizes (as when neat SBS is melted and blended into asphalt), the 3-dimensional network of tire rubber would be formed at a concentration near 6%.

This finding is very important in that it is one of the few which actually supports that crumb rubber may have the potential to provide improved pavement performance and therefore be cost-effective.

Based on the data in this study, the following criteria may be considered as a specification requirement for crumb rubber binders. However, it should be noted that this requirement would eliminate many of the crumb rubber binders currently in use. The department may need to consider more research into this area before adoption of this specification.

Direct Tension, SHRP B-006 (after PAV aging)

Failure Strain at -15°C, minimum of 3%,

Force Ductility (Shuler et al. 1985)

Area Under Stress-Strain Curve at 4°C and 5 cm/min, minimum of 2400 kPa (350 psi).

7

Guidelines on the Use of Crumb Rubber Modifier in Asphalt Concrete Pavements

Guidelines were developed in this study and are presented in Appendix F. The guidelines cover the following subjects:

- Uses of CRM in Asphalt Concrete Pavements,
- Mixture Design,
- Construction, and
- Quality Control.

This document is intended to provide short-term guidelines to TxDOT engineers to aid them in immediate implementation of new federal legislation requiring the use of CRM in bituminous pavements. These guidelines are based primarily upon results from literature reviews, a laboratory investigation, performance prediction modeling, and a limited field investigation. Modifications to the guidelines will be needed as TxDOT personnel advance the state-of-the-art through field experimentation and as further research is performed. While these guidelines are intended to partially address both wet and dry processes, it should be noted that only the wet process was evaluated in the field. Therefore, the guidelines are more applicable to wet-process CRM asphalt concrete pavements.

8

Testing Protocol for CRM Binders and Mixtures

All of the crumb rubber modified mixtures were tested in this study according to standard testing protocol; however, binder tests required modification. Test procedures are presented in Appendix G for the following:

- Brookfield Viscosity Test for Crumb-Rubber Modified Binders (175°C),
- Viscosity of Bituminous Binders Using a Shear Rheometer (25 and 60°C),
- Viscosity of Crumb-Rubber Modified Binder using Hand-Held Rotary Viscometer, and
- Sieve Analysis and Loose Fiber Content of Crumb Rubber Modifier.

9

Summary and Conclusions

This report presents guidelines, draft material specifications, and test protocol regarding the use of crumb rubber modifier in asphalt concrete pavements. Many of the important conclusions of this study are contained in research report 1332-1. Conclusions reached as a result of the work presented in this report follow.

Creep Testing to Predict Rutting for Crumb Rubber Mixtures

- At low stress levels, the damage induced in the sample is very low as compared to a high stress level.
- Mixtures that perform well at low stress levels do not necessarily perform well at high stress levels. Higher stress levels may be needed to identify mixes which are susceptible to permanent strain.
- Static creep testing alone may not be able to predict the performance of CMHB type CRM mixtures against rutting. The uniaxial repeated load permanent deformation test still suffers from the inability to fully evaluate mineral aggregate interaction and internal friction due to a lack of confinement. One of the most complete laboratory evaluations of permanent deformation for SMA or CMHB types of mixtures may be the simple shear test as prescribed by SHRP.

Fatigue Evaluation of CRM Laboratory Mixtures

- Compliance is the most important mixture property in terms of prediction of fatigue life.
- CRM has the potential to improve the fatigue performance of asphalt concrete pavements. Except for one mixture (18% coarse rubber added dry), all of the CRM mixtures analyzed in this study exhibited a marked improvement in predicted fatigue performance.
- In general, the finer CRM may yield better fatigue performance than the coarse CRM, whether added wet or dry or to dense-graded or CMHB type mixtures.

Thermal Cracking Evaluation of CRM Laboratory Mixtures

- CRM mixtures have the ability to improve the resistance to thermal cracking. One mixture (18% fine rubber added wet) performed exceptionally well. There was no significant difference among the rest of the mixtures considered.

Performance Evaluation of CRM Mixtures Using Texas Flexible Pavement System (TFPS)

- Mixtures with higher stiffness values perform better in hot-wet climates, and mixtures with lower stiffness perform better in cold-dry climates.
- Besides modulus, support conditions have a great importance in predicting the in-place performance of the mixture. Rutting occurs if the stiffness of the base is greater than the stiffness of the surface layer. Therefore, exercise caution when using CRM mixtures over asphalt-treated bases. This is particularly true in hot-wet climates.
- Structures with granular bases (only) will yield better performance with stiffer mixtures. CRM mixtures are not suggested for use with granular bases, if other alternatives are available.
- Some caution should be exercised when interpreting TFPS results because of the unusual characteristics of the CMHB crumb rubber mixtures. These mixtures

have a higher film thickness which should provide better resistance to fatigue and thermal cracking, and the coarse stone matrix should resist rutting.

Viscosity Measurement of Crumb Rubber Binders

- A test protocol was developed for measurement of the viscosity of crumb rubber binders at 25 and 60°C.
- The creep test in a dynamic shear rheometer is shown in this study as an accurate and powerful technique to calculate viscosity for asphalt and crumb rubber modified asphalt binders. Recovery tests should also be done along with the creep tests to corroborate the information obtained from the creep test.

Field Evaluation

- Three crumb rubber asphalt concrete pavement projects were evaluated in this study. All three were constructed without problem and are performing very well. These pavements were constructed using TxDOTs crumb rubber mixture design Tex 232-F.
- AAMAS laboratory evaluations of two of the field mixtures showed the mixtures to be both rut resistant and fatigue resistant.

Other Research Considerations

- The AASHTO structural design procedure should be modified to include other material properties to design pavements using CRM mixtures. The only material property considered in the AASHTO structural design method is the stiffness of the mix at 21°C (70°F).
- Creep tests for CRM mixtures should be modified to consider Poisson's ratio.
- More testing and analysis is needed to measure the shear strength characteristics of CRM mixtures.

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Appendix A
Literature Review

The use of rubber in asphalt paving dates back to 1840 (Heitzman 1992). For the past several decades, natural rubber (latex) and synthetic rubber (polymers) were used to improve the elastic properties of the asphalt cement and the asphalt concrete. Used tire rubber or scrap tire rubber is called crumb rubber. Crumb rubber as a modifier has been in use for the past 30 years. In the early 1960's, Charles McDonald along with Sahuaro Petroleum company developed a highly elastic surface treatment called Stress Absorbing Membrane (SAM) using CRM. Arizona Department of Transportation (ADOT) placed the first SAM in 1968. The CRM used in this process was vulcanized rubber of a size ranging between #16 and #25 sieve size. ADOT also placed the first stress absorbing membrane interlayer (SAMI) in 1972 (Scofield 1989).

Crumb Rubber In Hot Mix Asphalt Concrete

ADOT first used crumb rubber in hot mix asphalt (HMA) in 1975. The Alaska Department of Transportation has been using crumb rubber in pavements since 1976, and it is still one of the state highway agencies that is actively involved in the development of technology in this area. There are several factors that affect the properties of the hot mix asphalt concrete when crumb rubber is added to the mixture such as shape, size, chemical composition, and texture of rubber particles. Other factors that influence the performance and construction practices of the crumb rubber modified asphalt concrete (CRMAC) include resilient modulus and voids in mineral aggregate (VMA). A detailed discussion of these factors is below. Several states agencies are now placing CRMAC in their pavements on an experimental basis with varying degrees of success.

Crumb Rubber

Tire rubber is the principal component in CRM. Used tire rubber is the main source of raw material. The source of crumb rubber can be of passenger car tires or commercial tires. There are three commonly used methods to process scrap tires into CRM: cracker mill process, granulator process, and micro-mill process. The crackermill process is the most common method. As scrap tire rubber is processed into CRM, steel belting and fiber reinforcing are separated and removed from the rubber. Inert mineral powder, like talc, is added to the CRM to reduce the tendency of the rubber particles to stick together. Depending upon the specifications of a particular project, the rubber particles are milled to the right size. Particles produced between the sizes 4.25 mm to 425 μm are called ground CRM and particles produced between 9.25 mm to 2 mm are known as granulated CRM (Heitzman 1992).

Studies conducted so far have indicated that the viscosity of the binder increases with the addition of the crumb rubber. The Brookfield viscometer is used to measure the viscosity of the CRM modified binders (Stroup-Gardiner et al. 1993). The amount of rubber content and size of the rubber particles influence the viscosity of the binder. As the concentration of the rubber particles increases, viscosity of the binder increases. Also, as the particle size decreases, the viscosity of the binder decreases (Roberts et al. 1989 and Shuler 1985). Viscosity also depends on the type of rubber. Industrial tire-modified binders show less increase in viscosity compared to the binders modified with passenger tire rubber (Stroup-Gardiner et al. 1989). Viscosity increases greatly with vulcanized rubber over devulcanized rubber. This can be seen in Figure A.1 (Piggot et al. 1977).

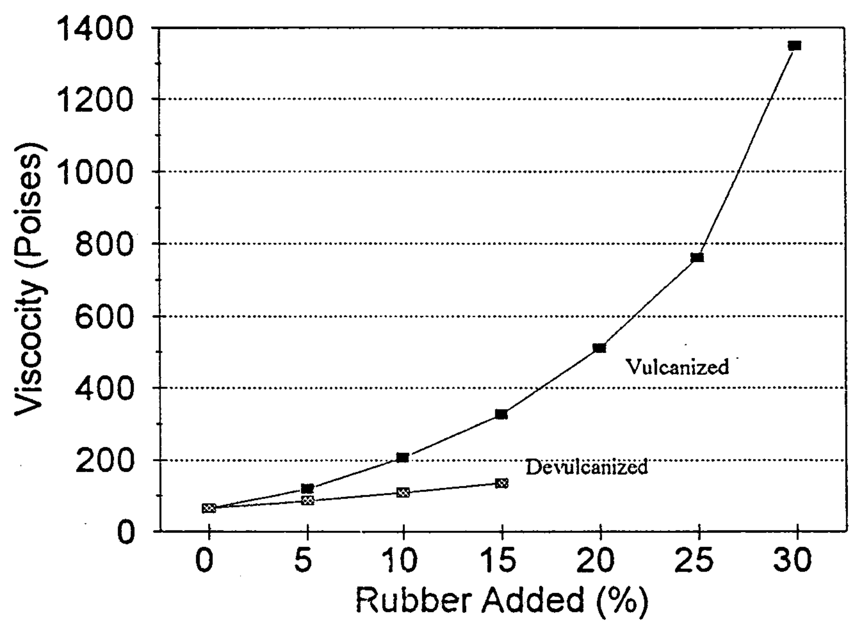

Figure A.1. **Change in Viscosity with Rubber Content for Vulcanized and Devulcanized Rubber (Piggot et al. 1977).**

The scrap tire processing method significantly affects the reaction of rubber with asphalt and the resultant properties of asphalt-rubber binder. Oliver (1981) found rubber morphology (structure) to be the most important factor affecting elastic properties. Porous surfaced rubber particles, of low bulk density, gave asphalt-rubber desirable high elastic recovery; while angular smooth surfaced particles, of high bulk density, resulted in poor elastic properties (Oliver 1981). Cryogenic grinding produces angular, smooth-faced particles.

Past studies indicate that the rate of swelling of the smaller rubber particle sizes is greater than that of the larger particle sizes. This implies that reducing the size of the rubber particles in an asphalt-rubber binder decreases the reaction time required for viscosity to reach a constant level (Roberts et al. 1989).

Chemical composition of rubber also affected the required reaction time for asphalt rubber. Table A.1 gives the typical chemical composition of recycled rubber products for asphalt rubber. Reaction times for asphalt rubber blends were measured using a rotational viscometer which can measure relative changes in fluid viscosity during the mixing and digestion. The digestion levels appropriate for field use of asphalt-rubber binders occurred when the rotational viscometer reading reached a constant level. It was also determined that tire rubber which contained no natural rubber took twice as long to reach a constant level of viscosity (Shuler et al. 1985).

Methods of Incorporating Tire Rubber in Hot Mix

Incorporating crumb rubber into hot mix can be broadly divided into two methods: a wet process and dry process. Heitzman (1992) defines the term wet process as any method that by which the crumb rubber is blended with the asphalt cement before mixing with the aggregate. The wet process was developed in Arizona and is called the

Table 2.1. **Typical Chemical Composition for Recycled Rubber Products for Asphalt-Rubber (Roberts et al. 1989).**

Component	Typical Chemical Composition					
	Auto Tires, Whole	Truck Tires, Whole	Auto Tread	Truck Tread, Mixed	Truck Tread, Precured	Devul. Whole
Acetone Extractables (%)	19.0	12.0	21.0	16.0	18.5	20.0
Ash (%)	5.0	5.0	5.0	4.0	4.0	20.0
Carbon Black (%)	31.0	28.5	32.0	30.0	32.0	20.0
Total Rubber Hydrocarbon (%)	46.0	54.0	42.0	50.0	45.5	40.0
Synthetic Rubber (%)	26.0	21.0	37.0	23.0	40.5	22.0
Natural Rubber (%)	20.0	33.0	5.0	27.0	5.0	18.0

McDonald process. The term dry process is defined as any method by which crumb rubber is added directly to the aggregate before blending with asphalt cement. The original dry process was developed in Sweden in the late 1960's and was patented as PlusRide in United States. The wet process has a variety of applications; whereas, the dry process is limited to HMA applications only.

Wet Process

As mentioned earlier, crumb rubber is blended with asphalt cement prior to mixing with aggregate. Rubber content of this blend is usually in the range of 18 to 26% by total weight of the blend. The blend is formulated at elevated temperatures to promote physical and chemical bonding of the two components. Various petroleum distillates are sometimes added to the blend to reduce viscosity and to promote workability (Stroup-

Gardiner et al. 1993 and Maupin 1992). This process of incorporating tire rubber into asphalt pavements is by far the most common method being used at this time and is backed by extensive research and experience.

One major disadvantage with asphalt rubber at this time is its high cost. Asphalt rubber binder costs at least 2 to 3 times more than a conventional asphalt cement. The reasons for high cost associated with asphalt rubber are unclear; however it is not due to the material cost but more likely to be attributed to royalties associated with development of asphalt rubber technology. The process for producing asphalt rubber is primarily marketed by International Surfacing of Arizona at this time and there is limited competition.

Many states are experimenting with several generic or unpatented wet processes (Maupin 1992 and Page et al. 1992). These processes generally involve preblending a finer gradation of rubber in smaller concentrations with asphalt cement. The Florida Department of Transportation (FDOT) has constructed several demonstration projects to evaluate various CRM quantities and different mixture types. FDOT constructed sections with 3% and 5% crumb rubber (-#80 sieve size) and 10% passing #40 sieve size. Virginia DOT has constructed a test section with 17% CRM (-#10 sieve size). All the above mixtures were of dense gradation. Rubber concentrations used by FDOT are significantly lower than the average 20% currently promoted by International Surfacing Inc.

Dry Process

Adding crumb rubber particles directly to the aggregate before blending with asphalt cement is called the dry process. In this process, the CRM particles act more like an aggregate. A specific gap-gradation of the aggregate is used with a high percentage of fine aggregate (8-12%). One of the major advantages of this process is the ease of incorporation into the mix (no special equipment is required). Several states in United States and some provinces in Canada have used the dry process to incorporate CRM in HMAC. The Alaska Department of Transportation was the first state to use CRM via the dry process (*PlusRide II User's Manual*).

A generic dry process is being used in the United States which was developed by Barry Takallou from his research experience with the PlusRide process (Takallou and Hicks 1988). This process is similar to the PlusRide process except that a finer gradation of rubber is used.

Mixture Design

From the early stages of development of CRMAC, conventional mixture design methods (Hveem and Marshall) were followed. These methods were modified to accommodate the CRM particles as well as the thicker films produced thereof. Adjustments for aggregate gradation for both dense and open-graded gradations were of the greatest importance in terms of voids in the mineral aggregate considerations. Asphalt cements with lower viscosities were selected and compared to a mixture containing no rubber in it. Vallerga (1981) observed that CRMAC requires higher binder contents than a mixture with no rubber. This is observed for both dense and open-gradations. He also suggested that criteria for the upper limit for the Marshall flow value should be more appropriately set to 20.

Jiminez (1982) used the Hveem mixture design method for designing CRM mixtures in the laboratory for both dense and open graded mixtures. Hveem stability values, in general, reduced, and there was an increase in the asphalt content. Swelling was observed for CRMAC samples. It was suggested that the samples be left in the mold for three days at ambient temperature before extrusion to eliminate swelling.

Roberts et al. (1986) suggested a mixture design procedure for CRM mixtures. The dense aggregate gradation was modified to permit space for rubber particles. Rubber particles were treated as additional aggregate. Mixture design then followed standard Marshall design procedure. Samples were to be cooled to room temperature before being extruded from the molds. A higher mixing temperature was also suggested.

Chehovits (1989) suggested mixture designs for CRMAC for both dense and open aggregate gradations. He proposed modifications to the existing Marshall and Hveem design methods for CRMAC. It was recommended that for dense gradations, upper values of the gradation limits were to be considered to create space for the additional amount of the binder. Gradation limits are shown in Table A.2. This method considers the CRM as an integral part of the overall binder. Mechanical mixers are to be used for the blending the aggregate and the binder. A curing time of 1 to 2 hours at 138°C (280±10° F) should be allowed before compaction. This is applicable for both Marshall and Hveem methods. Samples are allowed to cool in the molds for 4 hours to prevent elastic rebound of the rubber particles.

Two modifications were suggested for the Marshall design method. Due to the increased viscosity and other properties, CRMAC mixtures experience less compaction and densification from the traffic after construction. Therefore, the target air void range should be changed to 3 to 4% instead to 3 to 5%. The second modification was to increase the maximum flow value to determine the optimum binder content.

For the Hveem design procedure, he suggested checking the aggregates for minimum Hveem Stability requirements without rubber in the procedure and, using the same aggregates, lowering the minimum stability values to 20. It is also suggested that target air voids for the design should be 3 to 4% instead of a minimum 4%.

For open-graded mixtures using CRM added via dry process, the Marshall mixture design has been used with a reasonable degree of success (Roberts et al. 1989, Takallou et al. 1986, and Takallou et al. 1989). Similar results were obtained with the wet process: increase in Marshall flow values, increase in binder contents, and a decrease in voids.

Table A.2. **Suggested Gradation Specifications for Dense-Graded Asphalt Rubber Concrete (Percent Passing) (Chehovits 1989)**

Sieve Size	Mix Designation		
	9.5 mm (3/8")	12.5 mm (1/2")	19.0 mm (3/4")
25.0 mm (1")	100	100	100
19.0 mm (3/4")	100	100	90-100
12.5 mm (1/2")	100	90-100	70-90
9.5 mm (3/8")	90-100	75-95	60-80
#4	60-80	50-70	40-60
#8	40-60	35-50	30-45
#30	18-30	15-25	12-22
#50	8-18	6-16	5-14
#200	2-8	2-8	2-6

Canadian researchers developed a mixture design method for CRM mixtures (Svec an Veizer 1994). This mixture design method is based on the concepts of stone matrix asphalt (SMA) concrete mixtures. Basic characteristics of SMA mixtures are (1) high binder contents, (2) higher percent of fines and (3) gap-graded aggregates. The Marshall mixture design method is followed for SMA mixtures also.

Laboratory Evaluation and Performance of CRM Asphalt Concrete Mixes

Most of the initial laboratory studies for CRMAC concentrated on factors relating to reduction of reflection cracking. Only in the past decade has more research been directed towards characterizing the CRMAC mixtures in terms of engineering properties and their application as a structural layer. For a material to perform adequately over a design period, it should have certain characteristics: resistance to fatigue cracking, resistance to permanent deformation, resistance to thermal cracking, and resistance to moisture damage.

Shuler et al. (1985) studied the structural properties of CRMAC at the Texas Transportation Institute. Resilient modulus and moisture susceptibility tests were performed. Observed resilient modulus values for CRM mixtures were lower than mixtures containing no rubber. There was no considerable difference in indirect tensile strain for CRM and control mixtures. This study also suggests that CRM has no observable effect on performance after moisture exposure.

Further testing by several researchers (Takallou et al. 1986 and Hoyt et al. 1987) indicate that the resilient modulus for CRM mixtures is lower than for mixtures with no rubber at low temperatures, and equal or higher at higher temperatures (Figure A.2). This implies that CRM mixtures have lower temperature susceptibility than mixtures containing no rubber. Resilient modulus values depend also on the gradation of rubber, gradation of aggregates, and percent rubber in the mixture. Resilient modulus decreases with increasing coarseness of the rubber particles (Figure A.3). Higher percentages of rubber contents generally result in lower modulus for a given gradation (Figure A.4).

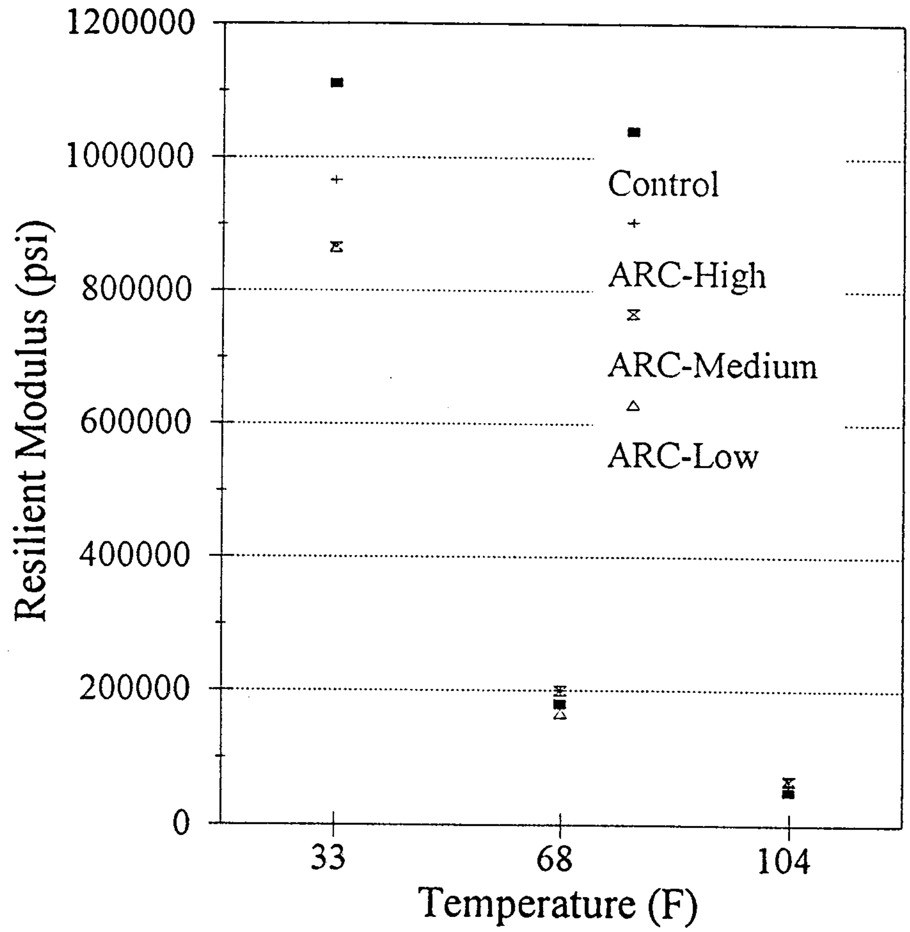

Figure A.2. **Resilient Modulus vs. Temperature for Control and CRM Mixtures (Hoyt et al. 1987).**

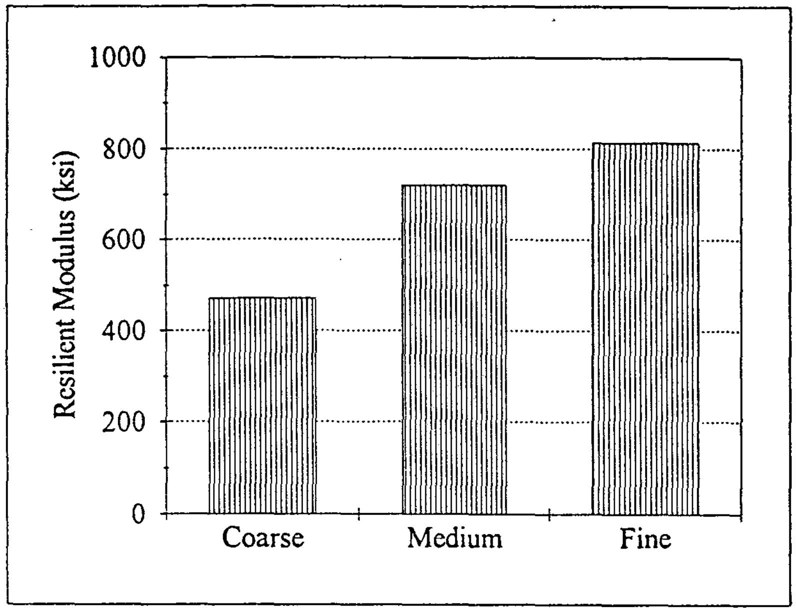

Figure A.3. **Effect of CRM Gradation on Resilient Modulus @10°C (Gap-Graded Aggregate) (Takallou et al. 1986).**

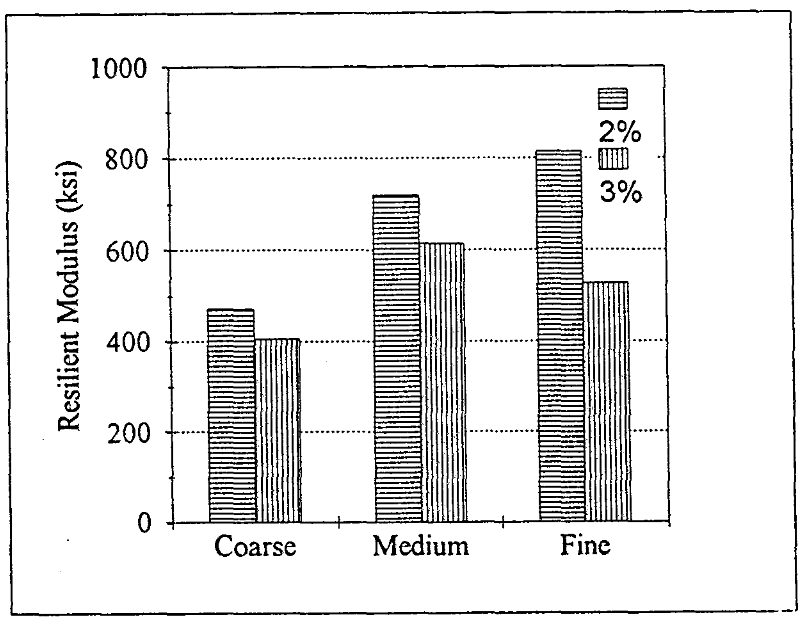

Figure A.4. **Effect of Rubber Content on Resilient Modulus @10°C (Gap-Graded Aggregate) (Takallou et al. 1986).**

Fatigue performance or the number of load repetitions to failure were performed on CRM mixtures in the past. Fatigue lives for CRM mixtures are generally higher than for a conventional mixture with no rubber (Takallou et al. 1986 and Hoyt et al. 1987). This is particularly true at higher temperatures (Table A.3). There was no significant effect from rubber content on the fatigue performance of CRM mixtures. Fatigue tests performed on cylindrical samples show that there is no significant difference in fatigue lives for different percentages of CRM (Figure A.5) (Svec and Veizer 1994).

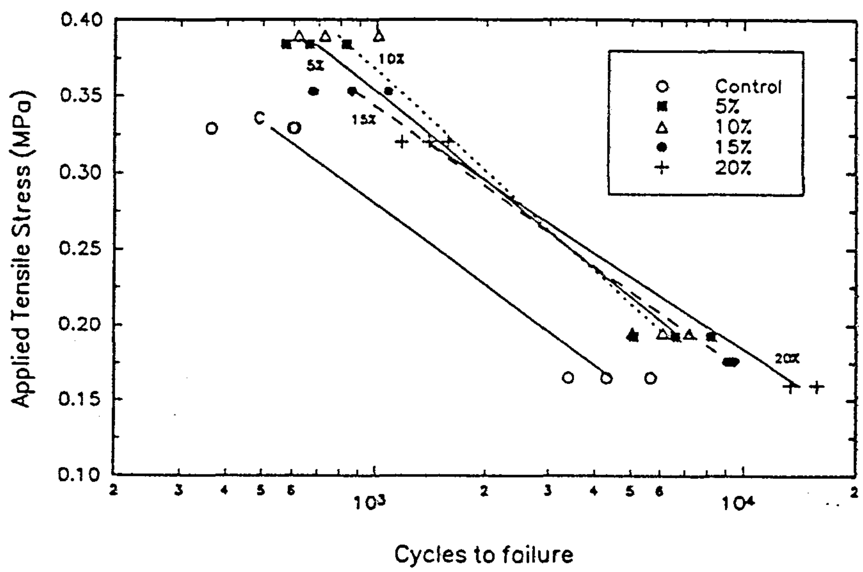

Figure A.5. **Number of Load Repetitions to Failure for Control and CRM Mixtures (Svec and Veizer 1994).**

Very limited testing has been done on CRM mixtures in terms of permanent deformation or creep tests. CRM mixtures, in general, have higher resistance to permanent deformation than a conventional mixture without rubber (Svec and Veizer 1994, Hoyt et al. 1987, and Krutz and Stroup-Gardiner 1992). It appears that slopes of

Table A.3. **Number of Repetitions to Failure for Control and CRM Mixtures at Different Temperatures (Hoyt et al. 1985).**

Material	Temperature °C	# of Repetitions to Failure
AC-10 Control	2	2130
	10	2530
	24	3520
	32	4060
	41	4710
ARC-Low	2	190
	10	940
	24	5780
	32	13200
	41	26100
ARC-Medium	2	2770
	10	5790
	24	14500
	32	20100
	41	29600
ARC-High	2	2950
	10	1990
	24	18900
	32	29800
	41	42900

creep curves for CRM mixtures are much flatter than conventional mixtures. It was suggested that permanent deformation testing in the laboratory should incorporate a repeated loading test instead of a static load test.

Appendix B

Results from TFPS Analysis

PERFORMANCE OF CRM MIXTURES USING TFPS

STRUCTURE - I

DISTRICT	: 20
COUNTY	: JEFFERSON
SURFACE LAYER	: THIN (38 - 64 mm)
BASE	: BLACK BASE ONLY (305 mm)
SUBGRADE	: SOFT
DESIGN ESAL'S	: 7 MILLION

PERFORMANCE OF CRM MIXTURES FOR STRUCTURE-I

Mixture Type	Thickness (mm)			Time for 1st Overlay (years)	Failure Mode	# of EASL's to first overlay (Millions)	Meets design requirements
	Black Base	Surface	Overlay				
Control	305	38	51	9.6	Rutting	2.99	Yes
	305	38	51	8.1	Rutting	2.48	Yes
DGF	305	38	51	9.3	Rutting	2.9	Yes
DGC	305	38	51	8.8	Rutting	2.73	Yes
10FW	305	38	0	7.8	Rutting	2.39	No
10CW	305	38	0	7.8	Rutting	2.39	No
18FW	305	38	51	8.1	Rutting	2.48	Yes
18CW	305	38	51	8.6	Rutting	2.26	yes
18FD	305	38	51	7.8	Rutting	2.39	no
18CD	305	38	51	8.75	Rutting	2.65	Yes
AFM	305	38	51	9.3	Rutting	2.90	yes

PERFORMANCE OF CRM MIXTURES USING TFPS

STRUCTURE - II

DISTRICT	: 20
COUNTY	: JEFFERSON
SURFACE LAYER	: THIN (38 - 64 mm)
BASE	: BLACK BASE (305 mm) + Granular Base (203 mm)
SUBGRADE	: SOFT
DESIGN ESAL'S	: 7 MILLION

PERFORMANCE OF CRM MIXTURES FOR STRUCTURE-II

Mixture Type	Thickness (mm)				Time for 1st Overlay (years)	Failure Mode	# of EASL's to first overlay (Millions)	Meets design requirements
	Black Base	Granular Base	Surface	Overlay				
Control	279	203	64	51	8.75	Rutting	2.65	Yes
DGF	279	203	64	51	8.75	Rutting	2.65	Yes
DGC	279	203	64	51	8.75	Rutting	2.65	Yes
10FW	279	203	64	51	9.75	Rutting	2.99	Yes
10CW	279	203	64	51	9.75	Rutting	2.99	Yes
18FW	279	203	64	51	9.5	Rutting	2.90	Yes
18CW	279	203	64	51	9.5	Rutting	2.90	yes
18FD	279	203	64	51	9.75	Rutting	2.99	Yes
18CD	279	203	64	51	9.25	Rutting	2.82	Yes
AFM	279	203	64	51	8.6	Rutting	2.65	Yes

STRUCTURE - III

DISTRICT	: 20
COUNTY	: JEFFERSON
SURFACE LAYER	: THICK (> 102 mm)
BASE	: BLACK BASE ONLY (279 mm)
SUBGRADE	: SOFT
DESIGN ESAL'S	: 7 MILLION

PERFORMANCE OF CRM MIXTURES FOR STRUCTURE-III

Mixture Type	Thickness (mm)			Time for 1st Overlay (years)	Failure Mode	# of EASL's to first overlay (Millions)	Meets design requirements
	Black Base	Surface	Overlay				
Control	279	102	51	11.75	Rutting	3.70	Yes
DGF	279	102	51	11.5	Rutting	3.61	Yes
DGC	279	102	51	10.75	Rutting	3.34	Yes
10FW	279	102	51	9.5	Rutting	2.90	Yes
10CW	279	102	51	9.75	Rutting	2.99	Yes
18FW	279	102	51	9.75	Rutting	2.99	Yes
18CW	279	102	51	10.25	Rutting	3.16	Yes
18FD	279	102	51	9.5	Rutting	2.90	Yes
18CD	279	102	51	10.5	Rutting	3.25	Yes
AFM	279	102	51	11.5	Rutting	3.61	Yes

PERFORMANCE OF CRM MIXTURES USING TFPS
STRUCTURE - IV

DISTRICT	: 20
COUNTY	: JEFFERSON
SURFACE LAYER	: THICK (> 102 mm)
BASE	: BLACK BASE (254 mm) + Granular Base (152 mm)
SUBGRADE	: SOFT
DESIGN ESAL'S	: 7 MILLION

PERFORMANCE OF CRM MIXTURES FOR STRUCTURE-IV

Mixture Type	Thickness (mm)				Time for 1st Overlay (years)	Failure Mode	# of EASL's to first overlay (Millions)	Meets design requirements
	Black Base	Granular Base	Surface	Overlay				
Control	254	152	102	51	9.75	Rutting	2.99	Yes
DGF	254	152	102	51	9.75	Rutting	2.99	Yes
DGC	254	152	102	51	9.75	Rutting	2.99	Yes
10FW	254	152	102	51	10.75	Rutting	3.34	Yes
10CW	254	152	102	51	10.75	Rutting	3.34	Yes
18FW	254	152	102	51	10.50	Rutting	3.25	Yes
18CW	254	152	102	51	10.50	Rutting	3.25	yes
18FD	254	152	102	51	10.75	Rutting	3.34	Yes
18CD	254	152	102	51	10.25	Rutting	3.16	Yes
AFM	254	152	102	51	9.75	Rutting	2.99	Yes

PERFORMANCE OF CRM MIXTURES USING TFPS
STRUCTURE - V

DISTRICT	: 20
COUNTY	: JEFFERSON
SURFACE LAYER	: THICK (> 102 mm)
BASE	: Granular Base Only (305 mm)
SUBGRADE	: SOFT
DESIGN ESAL'S	: 7 MILLION

PERFORMANCE OF CRM MIXTURES FOR STRUCTURE-V

Mixture Type	Thickness (mm)			Time for 1st Overlay (years)	Failure Mode	# of EASL's to first overlay (Millions)	Meets design requirements
	Granular Base	Surface	Overlay				
Control	305	203	0	> 18.0		7	Yes
DGF	305	203	0	> 18.0		7	Yes
DGC	305	203	0	> 18.0		7	Yes
10FW	305	203	51	15.25	Fatigue Cracking	5.03	Yes
10CW	305	203	51	15.25	Fatigue Cracking	5.13	Yes
18FW	305	203	51	16.25	Fatigue Cracking	5.43	Yes
18CW	305	203	51	16.25	Fatigue Cracking	5.43	Yes
18FD	305	203	51	15.25	Fatigue Cracking	5.03	Yes
18CD	305	203	51	17.25	Fatigue Cracking	6.04	Yes
AFM	305	203	51	> 18.0	Fatigue Cracking	7	Yes

PERFORMANCE OF CRM MIXTURES USING TFPS
STRUCTURE - I

DISTRICT	: 4
COUNTY	: ARMSTRONG
SURFACE LAYER	: THIN (38 - 64 mm)
BASE	: BLACK BASE ONLY (254 mm)
SUBGRADE	: SOFT
DESIGN ESAL'S	: 7 MILLION

PERFORMANCE OF CRM MIXTURES FOR STRUCTURE-I

Mixture Type	Thickness (mm)			Time for 1st Overlay (years)	Failure Mode	# of EASL's to first overlay (Millions)	Meets design requirements
	Black Base	Surface	Overlay				
Control	254	64	51	8.75	Rutting	2.65	Yes
DGF	254	64	51	8.75	Rutting	2.65	Yes
DGC	254	64	51	9.25	Rutting	2.82	Yes
10FW	254	64	51	9.75	Rutting	2.99	Yes
10CW	254	64	51	9.5	Rutting	2.90	Yes
18FW	254	64	51	9.5	Rutting	2.90	Yes
18CW	254	64	51	9.5	Rutting	2.90	Yes
18FD	254	64	51	9.75	Rutting	2.99	Yes
18CD	254	64	51	9.25	Rutting	2.82	Yes
AFM	254	64	51	9.25	Rutting	2.82	Yes

PERFORMANCE OF CRM MIXTURES USING TFPS
STRUCTURE - II

DISTRICT	: 4
COUNTY	: ARMSTRONG
SURFACE LAYER	: THIN (38 - 64 mm)
BASE	: BLACK BASE (279 mm) + Granular Base (152 mm)
SUBGRADE	: SOFT
DESIGN ESAL'S	: 7 MILLION

PERFORMANCE OF CRM MIXTURES FOR STRUCTURE-II

Mixture Type	Thickness (mm)				Time for 1st Overlay (years)	Failure Mode	# of EASL's to first overlay (Millions)	Meets design requirements
	Black Base	Granular Base	Surface	Overlay				
Control	279	152	51	51	10.5	Rutting	3.25	Yes
DGF	279	152	51	51	10.5	Rutting	3.25	Yes
DGC	279	152	51	51	10.75	Rutting	3.34	Yes
10FW	279	152	51	51	13.5	Rutting	4.35	Yes
10CW	279	152	51	51	12.5	Rutting	3.98	Yes
18FW	279	152	51	51	12.25	Rutting	3.88	Yes
18CW	279	152	51	51	11.75	Rutting	3.70	Yes
18FD	279	152	51	51	13.25	Rutting	4.26	Yes
18CD	279	152	51	51	11.25	Rutting	3.52	Yes
AFM	279	152	51	51	12.25	Rutting	3.98	Yes

STRUCTURE - III

DISTRICT	: 4
COUNTY	: ARMSTRONG
SURFACE LAYER	: THICK (> 102 mm)
BASE	: BLACK BASE ONLY (229 mm)
SUBGRADE	: SOFT
DESIGN ESAL'S	: 7 MILLION

PERFORMANCE OF CRM MIXTURES FOR STRUCTURE-III

Mixture Type	Thickness (mm)			Time for 1st Overlay (years)	Failure Mode	# of EASL's to first overlay (Millions)	Meets design requirements
	Black Base	Surface	Overlay				
Control	229	102	51	9.50	Rutting	2.90	Yes
DGF	229	102	51	9.75	Rutting	2.99	Yes
DGC	229	102	51	10.00	Rutting	3.08	Yes
10FW	229	102	51	11.25	Rutting	3.52	Yes
10CW	229	102	51	10.75	Rutting	3.34	Yes
18FW	229	102	51	10.50	Rutting	3.25	Yes
18CW	229	102	51	10.50	Rutting	3.25	Yes
18FD	229	102	51	10.75	Rutting	3.34	Yes
18CD	229	102	51	10.25	Rutting	3.16	Yes
AFM	229	102	51	9.75	Rutting	2.99	Yes

PERFORMANCE OF CRM MIXTURES USING TFPS
STRUCTURE - IV

DISTRICT	: 4
COUNTY	: ARMSTRONG
SURFACE LAYER	: THICK (> 102 mm)
BASE	: BLACK BASE (203 mm) + Granular Base (152 mm)
SUBGRADE	: SOFT
DESIGN ESAL'S	: 7 MILLION

PERFORMANCE OF CRM MIXTURES FOR STRUCTURE-IV

Mixture Type	Thickness (mm)				Time for 1st Overlay (years)	Failure Mode	# of EASL's to first overlay (Millions)	Meets design requirements
	Black Base	Granular Base	Surface	Overlay				
Control	203	152	102	51	8.25	Rutting	2.48	Yes
DGF	203	152	102	51	8.50	Rutting	2.56	Yes
DGC	203	152	102	51	8.50	Rutting	2.56	Yes
10FW	203	152	102	51	11.25	Rutting	3.52	Yes
10CW	203	152	102	51	10.50	Rutting	3.25	Yes
18FW	203	152	102	51	10.25	Rutting	3.16	Yes
18CW	203	152	102	51	9.75	Rutting	2.99	Yes
18FD	203	152	102	51	11.00	Rutting	3.43	Yes
18CD	203	152	102	51	9.00	Rutting	2.73	Yes
AFM	203	152	102	51	8.5	Rutting	2.56	Yes

PERFORMANCE OF CRM MIXTURES USING TFPS
STRUCTURE - V

DISTRICT	: 4
COUNTY	: ARMSTRONG
SURFACE LAYER	: THICK (> 102 mm)
BASE	: Granular Base Only (305 mm)
SUBGRADE	: SOFT
DESIGN ESAL'S	: 7 MILLION

PERFORMANCE OF CRM MIXTURES FOR STRUCTURE-V

Mixture Type	Thickness (mm)			Time for 1st Overlay (years)	Failure Mode	# of EASL's to first overlay (Millions)	Meets design requirements
	Granular Base	Surface	Overlay				
Control	305	178	0	> 18.0		7	Yes
DGF	305	178	0	> 18.0		7	Yes
DGC	305	178	51	16.75	Fatigue Cracking	5.63	Yes
10FW	305	178	51	13.00	Fatigue Cracking	4.16	Yes
10CW	305	178	51	13.60	Fatigue Cracking	4.35	Yes
18FW	305	178	51	14.25	Fatigue Cracking	4.64	Yes
18CW	305	178	51	14.50	Fatigue Cracking	4.74	Yes
18FD	305	178	51	13.25	Fatigue Cracking	4.26	Yes
18CD	305	178	51	16.25	Fatigue Cracking	5.43	Yes
AFM	305	178	51	> 18.0	Fatigue Cracking	7.0	Yes

Appendix C

Viscosity and "m" Values for Control and CRM Binders

Cooper Elastomer Technology Analytical Lab BOHLIN CS SYSTEM

Texaco AC 10/10000/25 C/Run 1 Constant Stress test

1994-09-12 11:50:06

P25DSR gap 1.00 mm
Â 1.00E+03 Pa
Viscosity 7.66E+04 Pas dln(J)/dln(t) 9.83E-01

Joc 4.55E-05 1/Pa Jor 8.45E-05 1/Pa

"Fit int. 1.26E+02 - 3.55E+02 s, 83 - 92"

Temperature 25.0 - 25.0 C
File name TXCT21

Cooper Elastomer Technology Analytical Lab BOHLIN CS SYSTEM

Texaco AC 10/ 2000/25 C/Run 1 Constant Stress test

1994-09-12 12:08:09

P25DSR gap 1.00 mm
Â 6.00E+02 Pa
Viscosity 7.78E+04 Pas dln(J)/dln(t) 9.81E-01

Joc 8.22E-05 1/Pa Jor 1.09E-04 1/Pa

"Fit int. 2.00E+02 - 5.62E+02 s, 87 - 96"

Temperature 24.9 - 25.0 C
File name TXCT31

Cooper Elastomer Technology Analytical Lab BOHLIN CS SYSTEM

Texaco AC 10/ 500/60 C/Run 1 Constant Stress test

1994-09-12 13:16:55

P25DSR gap 1.00 mm
Â 5.00E+02 Pa
Viscosity 1.39E+02 Pas dln(J)/dln(t) 9.80E-01

Joc 2.42E-03 1/Pa Jor -4.94E-04 1/Pa

"Fit int. 1.00E+01 - 2.82E+01 s, 61 - 70"

Temperature 60.0 - 60.0 C
File name TXCS11

Cooper Elastomer Technology Analytical Lab BOHLIN CS SYSTEM

Texaco AC 10/ 1000/60 C/Run 1 Constant Stress test

1994-09-12 13:04:33

P25DSR gap 1.00 mm
Â 1.00E+03 Pa
Viscosity 1.35E+02 Pas dln(J)/dln(t) 9.81E-01

Joc 1.55E-03 1/Pa Jor -1.41E-04 1/Pa

"Fit int. 6.31E+00 - 1.78E+01 s, 57 - 66"

Temperature 59.9 - 60.0 C
File name TXCS21

Cooper Elastomer Technology Analytical Lab BOHLIN CS SYSTEM

Texaco AC 10 PAV/1000/25 C/Run 1 Constant Stress test

1994-09-12 14:14:50

P25DSR gap 1.00 mm
Â 1.00E+03 Pa
Viscosity 1.66E+06 Pas dln(J)/dln(t) 8.32E-01

Joc 4.65E-05 1/Pa Jor 5.17E-05 1/Pa

"Fit int. 2.24E+02 - 6.31E+02 s, 88 - 97"

Temperature 25.0 - 25.0 C
File name TXPCT11

Cooper Elastomer Technology Analytical Lab BOHLIN CS SYSTEM

Texaco AC 10 PAV/2000/25 C/Run 1 Constant Stress test

1994-09-12 14:38:08

P25DSR gap 1.00 mm
Â 2.00E+03 Pa
Viscosity 1.83E+06 Pas dln(J)/dln(t) 8.52E-01

Joc 3.63E-05 1/Pa Jor 5.01E-05 1/Pa

"Fit int. 2.24E+02 - 6.31E+02 s, 88 - 97"

Temperature 25.0 - 25.1 C
File name TXPCT21

Cooper Elastomer Technology Analytical Lab — BOHLIN CS SYSTEM

Texaco AC 10 PAV/200 /60 C/Run 1 — Constant Stress test

1994-09-12 15:24:19

P25DSR gap 1.00 mm
Â 2.00E+02 Pa
Viscosity 1.32E+03 Pas — dln(J)/dln(t) 9.81E-01

Joc 2.65E-04 1/Pa — Jor 3.41E-04 1/Pa

"Fit int. 1.12E+01 - 3.16E+01 s, 62 - 71"

Temperature 60.0 - 60.0 C
File name TXPCS11

Cooper Elastomer Technology Analytical Lab — BOHLIN CS SYSTEM

Texaco AC 10 PAV/500 /60 C/Run 1 — Constant Stress test

1994-09-12 15:12:25

P25DSR gap 1.00 mm
Â 5.00E+02 Pa
Viscosity 1.16E+03 Pas — dln(J)/dln(t) 9.81E-01

Joc 1.17E-04 1/Pa — Jor 2.05E-04 1/Pa

"Fit int. 4.47E+00 - 1.26E+01 s, 54 - 63"

Temperature 60.7 - 60.0 C
File name TXPCS21

Cooper Elastomer Technology Analytical Lab | BOHLIN CS SYSTEM

Texaco AC 10- 4%Rouse/1000/25 C/Run 1 | Constant Stress test

1994-09-12 18:51:55

P25DSR gap 2.00 mm
Â 1.00E+03 Pa
Viscosity 1.60E+05 Pas | dln(J)/dln(t) 9.85E-01

Joc 3.55E-05 1/Pa | Jor 1.54E-04 1/Pa

"Fit int. 2.82E+02 - 7.94E+02 s, 90 - 99"

Temperature 25.0 - 25.0 C
File name RFCT11

Cooper Elastomer Technology Analytical Lab | BOHLIN CS SYSTEM

Texaco AC 10- 4%Rouse/2000/25 C/Run 1 | Constant Stress test

1994-09-12 19:21:21

P25DSR gap 2.00 mm
Â 2.00E+03 Pa
Viscosity 1.52E+05 Pas | dln(J)/dln(t) 9.82E-01

Joc 4.90E-05 1/Pa | Jor 1.06E-04 1/Pa

"Fit int. 2.51E+02 - 7.08E+02 s, 89 - 98"

Temperature 25.0 - 25.0 C
File name RFCT21

Cooper Elastomer Technology Analytical Lab BOHLIN CS SYSTEM

Texaco AC 10- 4%Rouse/ 200/60 C/Run 1 Constant Stress test

1994-09-12 20:04:24

P25DSR gap 2.00 mm
Â 2.00E+02 Pa
Viscosity 2.89E+02 Pas dln(J)/dln(t) 9.80E-01

Joc 2.90E-03 1/Pa Jor -3.58E-04 1/Pa

"Fit int. 2.51E+01 - 7.08E+01 s, 69 - 78"

Temperature 59.8 - 59.9 C
File name RFCS11

Cooper Elastomer Technology Analytical Lab BOHLIN CS SYSTEM

Texaco AC 10- 4%Rouse/ 500/60 C/Run 1 Constant Stress test

1994-09-12 20:19:53

P25DSR gap 2.00 mm
Â 5.00E+02 Pa
Viscosity 2.66E+02 Pas dln(J)/dln(t) 9.81E-01

Joc 1.33E-03 1/Pa Jor -5.59E-06 1/Pa

"Fit int. 1.12E+01 - 3.16E+01 s, 62 - 71"

Temperature 60.0 - 60.0 C
File name RFCS21

Cooper Elastomer Technology Analytical Lab BOHLIN CS SYSTEM

Texaco AC 10-18%Rouse/1000/25 C/Run 1 Constant Stress test

1994-09-12 15:58:52

P25DSR gap 2.00 mm
Â 1.00E+03 Pa
Viscosity 1.90E+06 Pas dln(J)/dln(t) 7.46E-01

Joc 1.08E-04 1/Pa Jor 1.43E-04 1/Pa

"Fit int. 3.55E+02 - 1.00E+03 s, 92 - 101"

Temperature 24.8 - 25.0 C
File name RECT11

Cooper Elastomer Technology Analytical Lab BOHLIN CS SYSTEM

Texaco AC 10-18%Rouse/2400/25 C/Run 1 Constant Stress test

1994-09-12 16:32:43

P25DSR gap 2.00 mm
Â 2.40E+03 Pa
Viscosity 1.76E+06 Pas dln(J)/dln(t) 8.18E-01

Joc 7.50E-05 1/Pa Jor 1.21E-04 1/Pa

"Fit int. 3.55E+02 - 1.00E+03 s, 92 - 101"

Temperature 25.0 - 25.0 C
File name RECT21

Cooper Elastomer Technology Analytical Lab BOHLIN CS SYSTEM

Texaco AC 10-18%Rouse/ 200/60 C/Run 1 Constant Stress test

1994-09-12 17:22:03

P25DSR gap 2.00 mm
Â 2.00E+02 Pa
Viscosity 3.43E+03 Pas dln(J)/dln(t) 9.80E-01

Joc 1.28E-03 1/Pa Jor 1.33E-03 1/Pa

"Fit int. 1.41E+02 - 3.98E+02 s, 84 - 93"

Temperature 60.0 - 60.1 C
File name RECS11

Cooper Elastomer Technology Analytical Lab BOHLIN CS SYSTEM

Texaco AC 10-18%Rouse/ 500/60 C/Run 1 Constant Stress test

1994-09-12 17:44:42

P25DSR gap 2.00 mm
Â 5.00E+02 Pa
Viscosity 2.79E+03 Pas dln(J)/dln(t) 9.82E-01

Joc 4.02E-04 1/Pa Jor 6.40E-04 1/Pa

"Fit int. 3.98E+01 - 1.12E+02 s, 73 - 82"

Temperature 60.0 - 60.0 C
File name RECS21

Appendix D

Laboratory Data for Abilene Field Mixture

ABILENE FIELD MIXTURE

Table D1. Summary Of The Static Creep Test Data For Abilene Field Mix.

Sample#	Stress Level @ 69 kpa	Stress @ 4140 kpa		
	2	1	4	5
Air Voids %	2.5	2.5	2.5	2.7
Binder Content, %	6.8%	6.8%	6.8%	6.8%
Permanent Strain mm/mm	6.3×10^{-4}	2.85×10^{-4}	13.4×10^{-4}	4.2×10^{-4}
Slope mm/mm sec	4.6×10^{-8}	2.8×10^{-8}	35.2×10^{-8}	4.1×10^{-8}
Creep Stiffness kpa	43645	165186	26986	163378

Table D2. AAMAS Test Results For *Unconditioned Specimens @5°C.*

Rice Specific Gravity	2.38		
Sample#	1	3	Average
Bulk Specific Gravity	2.238	2.248	2.243
Air Voids,%	5.9	5.5	5.8
Total Resilient Modulus[1], kpa	$103.6x10^5$	$95.5x10^5$	$99.6x10^5$
Indirect Tensile Strength, kpa	840.2	809.4	824.8
Indirect Tensile Strain @Failure, mm/mm	1.75	2.35	2.05

[1] - Average of the two axes.

Table D3. AAMAS Test Results For *Unconditioned Specimens @25°C*

Rice Specific Gravity	2.38			
Sample#	4	12	18	Average
Bulk Specific Gravity	2.238	2.252	2.246	2.245
Air Voids,%	6.0	5.4	5.6	5.7
Total Resilient Modulus[1], kpa	$28.65x10^5$	$26.84x10^5$	$30.77x10^5$	$4.17x10^5$
Indirect Tensile Strength, kpa	585.3	611.7	611.6	87.37
Indirect Tensile Strain @Failure, mm/mm	7.77	9.57	8.97	8.77

[1] - Average of the two axes.

Table D4. AAMAS Test Results For *Unconditioned Specimens @40°C.*

Rice Specific Gravity	2.38			
Sample#	8	9	16	Average
Bulk Specific Gravity	2.244	2.31	2.259	2.271
Air Voids,%	5.7	6.3	5.1	5.7
Total Resilient Modulus[1], kpa	857981	669735	744600	109774
Indirect Tensile Strength, kpa	154.63	142.9	179.1	23.03
Indirect Tensile Strain @Failure, mm/mm	15.22	24.75	20.1	20.02

[1] - Average of the two axes.

Table D5. AAMAS Test Results For *Moisture Conditioned* Specimens Tested @25°C

Rice Specific Gravity	2.38			
Sample#	2	10	13	Average
Bulk Specific Gravity	2.243	2.232	2.259	2.244
Air Voids,%	5.8	6.2	5.0	5.7
Degree Of Saturation, %	73.3	75.9	66.9	72.02
Total Resilient Modulus[1], kpa	$20.8x10^5$	$23.2x10^5$	$27.7x10^5$	$3.47x10^5$
Indirect Tensile Strength, kpa	465.8	537.5	523.0	73.73
Indirect Tensile Strain @Failure, mm/mm	19.75	18.74	13.53	17.34

[1] - Average of the two axes.

Table D6. AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @5°C For Set-1.

Rice Specific Gravity	2.38			
Sample#	5	15	17	Average
Bulk Specific Gravity	2.244	2.245	2.246	2.245
Air Voids,%	5.7	5.7	5.6	5.7
Total Resilient Modulus[1], kpa	$211.3x10^5$	$211.4x10^5$	$224.9x10^5$	$214.9x10^5$
Log-Log Slope of the Indirect Tensile Creep Curve	0.4158	0.4489	0.4428	0.4358
Indirect Tensile Creep Modulus @3600sec, kpa	499698		500740	500216

[1] - Average of the two axes.

Table D7. AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @5°C For Set-2.

Rice Specific Gravity	2.38			
Sample#	6	7	14	Average
Bulk Specific Gravity	2.241	2.260	2.233	2.244
Air Voids,%	5.8	5.0	6.2	5.7
Total Resilient Modulus[1], kpa	$209.3x10^5$	$204.0x10^5$	$189.8x10^5$	$201.0x10^5$
Indirect Tensile Strength, kpa	836.7	756.6	677.02	756.7
Indirect Tensile Strain @Failure, mm/mm (10^{3-})	2.06	2.15	2.66	2.29

[1] - Average of the two axes.

Table D8. AAMAS Test Results For *Traffic Densified Samples* Tested @40°C For Set-1.

Rice Specific Gravity	2.38			
Sample#	5	6	7	Average
Bulk Specific Gravity	2.322	2.311	2.322	2.318
Air Voids,%	2.4	2.9	2.4	2.6
Slope Of Compressive Creep Test Curve, b	0.07475	0.22504	0.11027	0.13669
Intercept Of Compressive Creep Test Curve, a	0.00284	0.00156	0.00423	0.00288
Total Permanent Strain @3600sec, mm/mm	0.00355	0.00739	0.00877	0.00657
Compressive Creep Modulus @3600sec, kpa	78867	41697	39356	53309

[1] - One of the LVDTs was off the range, so data was discarded.

Table D9. AAMAS Test Results For *Traffic Densified Samples* Tested @40°F For Set-2.

Rice Specific Gravity	2.38			
Sample#	8	9	10	Average
Bulk Specific Gravity	2.318	2.315	2.321	2.318
Air Voids,%	2.6	2.7	2.5	2.6
Unconfined Compressive Strength, kpa	1158.5	1021.5	1137.1	1105.4
Compressive Strain @Failure, mm/mm (10^{-3})	24.69	26.25	24.69	25.21

[1] - Average of the two axes.

Table D10. AAMAS Test Results For *Traffic Densified Samples* Tested @40°C For Set-3.

Rice Specific Gravity	2.380		
Sample#	3	4	Average
Bulk Specific Gravity	2.319	2.318	2.318
Air Voids,%	2.6	2.6	2.6
Dynamic Resilient Modulus @200th cycle, kpa	1127460	1101240	1114350
Slope Of Repetitive Creep Test Curve, b	0.61427	0.483034	0.548652
Intercept Of Repetitive Creep Test Curve, a	0.00081	0.00021	0.00051
Total Permanent Strain @10000sec, mm/mm	0.025804	0.018742	0.022273

[1] - One of the LVDTs was off the range, so data was discarded.

Appendix E

Laboratory Data for Lufkin Field Mixture

LUFKIN FIELD MIXTURE

Table E1. Summary Of The Static Creep Test Data For Lufkin Field Mix.

Sample#	Stress Level @ 69 kpa		Stress Level @ 414 kPa	
	1	3	260	460
Air Voids, %				
Binder Content, %	7.3	7.3	7.3	7.3
Permanent Strain mm/mm	4.4×10^{-4}	7.3×10^{-4}	8.4×10^{-4}	15.9×10^{-4}
Slope (mm/mm sec)	4.6×10^{-8}	3.5×10^{-8}	2.7×10^{-8}	5.4×10^{-8}
Creep Stiffness, kpa	47052	35316	145029	113333

Table E2. AAMAS Test Results For *Unconditioned Specimens @5°C.*

Rice Specific Gravity	2.376			
Sample#	10	17	18	Average
Bulk Specific Gravity		2.22	2.24	2.23
Air Voids,%		6.6	5.7	6.2
Total Resilient Modulus[1], kpa	6.77×10^6	7.14×10^6	7.71×10^6	7.21×10^6
Indirect Tensile Strength, kpa	[a]	557.6	554.2	555.9
Indirect Tensile Strain @Failure, cm/cm (10^{-3})	[a]	5.32	6.67	5.99

[1] - Average of the two Axes
[a] - one of the LVDTs was off the range, so data was discarded.

Table E3. AAMAS Test Results For *Unconditioned Specimens @25°C.*

Rice Specific Gravity	2.376			
Sample#	11	14	15	Average
Bulk Specific Gravity	2.228	2.242	2.222	2.231
Air Voids,%	6.2	5.6	6.5	6.1
Total Resilient Modulus[1], kpa	1.84×10^6	1.62×10^6	1.62×10^6	1.69×10^6
Indirect Tensile Strength, kpa	410.1	479.6	442.6	444.1
Indirect Tensile Strain @Failure, cm/cm (10^{-3})	18.5	12.5	9.92	13.6

[1] - Average of the two axes.

Table E4. AAMAS Test Results For *Unconditioned Specimens @40°C.*

Rice Specific Gravity	2.376			
Sample#	1	7	9	Average
Bulk Specific Gravity		2.232	2.22	2.226
Air Voids,%		6.1	6.6	6.4
Total Resilient Modulus[1], kpa	382129	420134	453082	418451
Indirect Tensile Strength, kpa	[a]	129.7	114.8	122.5
Indirect Tensile Strain @Failure, cm/cm (10^{-3})	[a]	18.80	22.65	20.73

[1] - Average of the two Axes
[a] - One of the LVDTs was off the range, so data was discarded.

Table E5. AAMAS Test Results For *Moisture Conditioned* Specimens Tested @25°C.

Rice Specific Gravity	2.376			
Sample#	2	5	6	Average
Bulk Specific Gravity	2.216	2.254	2.223	2.231
Air Voids,%	6.7	5.1	6.4	6.1
Degree Of Saturation, %	68.0	70.1	72.9	70.3
Total Resilient Modulus[1], kpa	726818	1235252	1025740	995939
Indirect Tensile Strength, kpa	290.6	347.9	328.2	322.2
Indirect Tensile Strain @Failure, cm/cm (10^{-3})	23.93	14.92	22.57	20.47

[1] - Average of the two axes.

Table E6. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested at 5°C For Set-1.**

Rice Specific Gravity	2.376			
Sample#	8	13	16	Average
Bulk Specific Gravity		2.213	2.261	2.237
Air Voids,%		6.9	4.8	5.9
Total Resilient Modulus[1], kpa	9.77×10^6	9.56×10^6	10.31×10^6	9.88×10^6
Log-Log Slope of the Indirect Tensile Creep Curve	[a]	0.41469	0.432578	0.423634
Indirect Tensile Creep Modulus @ 3600sec, kpa	[a]	94468	108082	101275

[1] - Average of the two axes.
[a] - Loss of data due to software problem in MTS machine.

Table E7. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @5°C For Set-2.**

Rice Specific Gravity	2.38		
Sample#	3	12	Average
Bulk Specific Gravity	2.210	2.210	2.210
Air Voids,%	7.0	7.0	7.0
Total Resilient Modulus[1], kpa	8.35×10^6	9.11×10^6	8.73×10^6
Indirect Tensile Strength, kpa	654.7	645.9	650.3
Indirect Tensile Strain @Failure, cm/cm (10^{-3})	2.06	2.11	2.08

[1] - Average of the two Axes

Table E8. AAMAS Test Results For *Traffic Densified Samples* Tested @40°C For Set-1.

Rice Specific Gravity	2.376		
Sample#	4	6	Average
Bulk Specific Gravity	2.321	2.314	2.318
Air Voids,%	2.3	2.6	2.5
Slope Of Compressive Creep Test Curve, b	0.05946	0.13322	0.09634
Intercept Of Compressive Creep Test Curve, a	0.003638	0.001959	0.00288
Total Permanent Strain @ 3600sec, cm/cm	0.00593	0.005846	0.005888
Compressive Creep Modulus @ 3600sec, kpa	69593	70463	70028

Table E9. AAMAS Test Results For *Traffic Densified Samples* Tested @40°C For Set-2.

Rice Specific Gravity	2.376		
Sample#	3	5	Average
Bulk Specific Gravity	2.316	2.309	2.313
Air Voids,%	2.5	2.8	2.7
Dynamic Resilient Modulus @200thcycle, kpa		705870	705870
Slope Of Repetitive Creep Test Curve, b		0.69797	0.69797
Intercept Of Repetitive Creep Test Curve, a		0.000398	0.000398
Total Permanent Strain @ 10000 sec, cm/cm		0.02695	0.02695

[1] - One of the LVDTs was off the range, so data was discarded.

Appendix F

Guidelines on the Use of Crumb Rubber Modifier

in Asphalt Concrete Pavements

GUIDELINES ON THE USE OF CRUMB RUBBER MODIFIER IN ASPHALT CONCRETE PAVEMENTS

Crumb rubber modifier (CRM) has been used in asphalt binders for chip seals and interlayers for many years in all parts of Texas. Only in the past few years, however, has the Texas Department of Transportation (TxDOT) used CRM in hot-mix asphalt concrete (HMAC). This is the case throughout the United States, as well. While the technology is well established and documented on the use of CRM in chip seals, technology for using CRM in HMAC is still in an experimental stage of development.

These guidelines are based on the results of a 2-year TxDOT Research Study (0-1332) performed by Texas Transportation Institute. This document is intended to provide short-term guidelines to TxDOT engineers to aid them in immediate implementation of new federal legislation requiring the use of CRM in bituminous pavements. These guidelines are based primarily upon results from literature reviews, a laboratory investigation, performance prediction modeling, and a limited field investigation. Modifications to the guidelines will be needed as TxDOT personnel advance the state-of-the-art through field experimentation and as further research is performed.

Uses of CRM in Asphalt Concrete Pavements

One of two methods, wet or dry, are most commonly used to incorporate crumb rubber into asphalt paving mixtures. The wet process defines any method where CRM is added to the asphalt cement prior to incorporating the binder into the asphalt paving project. The dry process defines any method of adding the CRM directly into the hot mix asphalt mixture process, typically pre-blending the CRM with the heated aggregate prior to charging the mix with asphalt. While these guidelines are intended to partially address both wet and dry processes, it should be noted that only the wet process was evaluated in the field.

Based on the laboratory results of study 1332 and the use of the Texas Flexible Pavement System (TFPS), the following tables can be used as general guidelines regarding the predicted performance of crumb rubber mixtures. Chapter 3 of this report presents the predicted performance of a number of different types of mixtures; however, the following tables represent only one mix: a CMHB CRM mixture which was actually place in the field and tested in the laboratory. Table F1 predicts performance for a hot-wet Texas climate and Table F2 presents results for a cold-dry Texas climate. Note that for similar pavement life, a reduction in base thickness can be achieved in the cold-dry climates.

Table F1. Predicted Performance of CRM Mixtures for Hot-Wet Texas Climate.

Pavement Structure*	Failure Mode	Time to First Overlay, years
I	Rutting	9.3
II	Rutting	8.6
III	Rutting	11.5
IV	Rutting	9.8
V	Fatigue Cracking	> 18

* See notes below.

Structure I: Thin CRM Surface Layer (64 mm or 2.5 in)
305 mm (12 in) Black Base

Structure II: Thin CRM Surface Layer (64 mm or 2.5 in)
305 mm (12 in) Black Base
203 mm (8 in) Granular Base

Structure III: Thick CRM Surface Layer (102 mm or 4 in)
279 mm (11 in) Black Base

Structure IV: Thick CRM Surface Layer (102 mm or 4 in)
254 mm (10 in) Black Base
152 mm (6 in) Granular Base

Structure V: Thick CRM Surface Layer (203 mm or 8 in)
305 mm (12 in) Granular Base

Table F2. Predicted Performance of CRM Mixtures for Cold-Dry Texas Climate.

Pavement Structure*	Failure Mode	Time to First Overlay, years
I	Rutting	9.3
II	Rutting	12.3
III	Rutting	9.8
IV	Rutting	8.5
V	Fatigue Cracking	> 18

* See notes below.

Structure I: Thin CRM Surface Layer (64 mm or 2.5 in)
254 mm (10 in) Black Base

Structure II: Thin CRM Surface Layer (51 mm or 2 in)
279 mm (11 in) Black Base
152 mm (6 in) Granular Base

Structure III: Thick CRM Surface Layer (102 mm or 4 in)
229 mm (9 in) Black Base

Structure IV: Thick CRM Surface Layer (102 mm or 4 in)
203 mm (8 in) Black Base
152 mm (6 in) Granular Base

Structure V: Thick CRM Surface Layer (178 mm or 7 in)
305 mm (12 in) Granular Base

Mixture Design

It should be noted here that the current thinking of most asphalt technologists is that crumb rubber should be used in mixtures that are open- or gap-graded, such as TxDOT's Coarse-Matrix, High Binder (CMHB) mixture or Stone Matrix Asphalt (SMA) Mixtures. These types of mixtures allow room for the crumb rubber particles while maintaining stone-on-stone contact under load. If dense-graded asphalt concrete mixtures are used, the CRM concentration should be significantly less than what is typically used (no more than 10% by weight of binder). Further recommendations follow.

CRM Concentrations ≥ 10% by Weight of Binder

Crumb rubber modified asphalt concrete mixtures containing 10% CRM (or more) by weight of the binder should be designed according to Tex-232-F, *Mixture Design Procedure for Crumb Rubber Modified Asphalt Concrete*. This procedure is based upon the following design criteria and can be used for either wet or dry processes of CRM incorporation:

- Minimum Voids in the Mineral Aggregate (VMA) = 20%,
- Optimum laboratory molded density = 97.0%,
- Minimum volume of binder = 17%
- Minimum volume of coarse aggregate (retained on No. 10 sieve) = optimum volume of coarse aggregate plus 5.0% (as determined from density versus volume of coarse aggregate curve), and
- Percent aggregate passing No. 200 sieve = 6.0%.

Target gradation values are as follows:

Sieve Sizes	Fine Surface, % Passing (limits)	Coarse Surface, % Passing (limits)
22.4 mm (7/8 in)	-	100
16.0 mm (5/8 in)	100	98-100
9.5 mm (3/8 in)	98-100	55-65
4.75 mm (No.4)	40-50	35-45
2.00mm (No.10)	15-25	15-25
75μm (No. 200)	4-8	4-8

CRM Concentrations < 10% by Weight of Binder

Tex-232-F

Crumb rubber modified asphalt concrete mixtures containing less than 10% CRM by weight of the binder should be designed according to Tex-232-F, *Mixture Design Procedure for Crumb Rubber Modified Asphalt Concrete*. In addition, however, the mixture's draindown potential should be evaluated in the laboratory. Draindown is considered to be that portion of the asphalt binder which separates itself from the sample as a whole.

Bulletin C-14

If a dense-graded mixture is to be used, conventional mixture design requirements as specified in Construction Bulletin C-14 should be used; however, a laboratory investigation should be performed to determine the maximum quantity of CRM which can be incorporated into the mix. Typically, as increasing quantities of CRM are added to a dense-graded mixture, Hveem stability values decrease and air voids increase. *Do not alter specification requirements to allow lower Hveem stability and lower densities.* Rather, use lower quantities of CRM such that acceptable Hveem stability and density is maintained. Figure F1 below shows the effect of CRM on a dense-graded mixture (Type D) as measured in the laboratory.

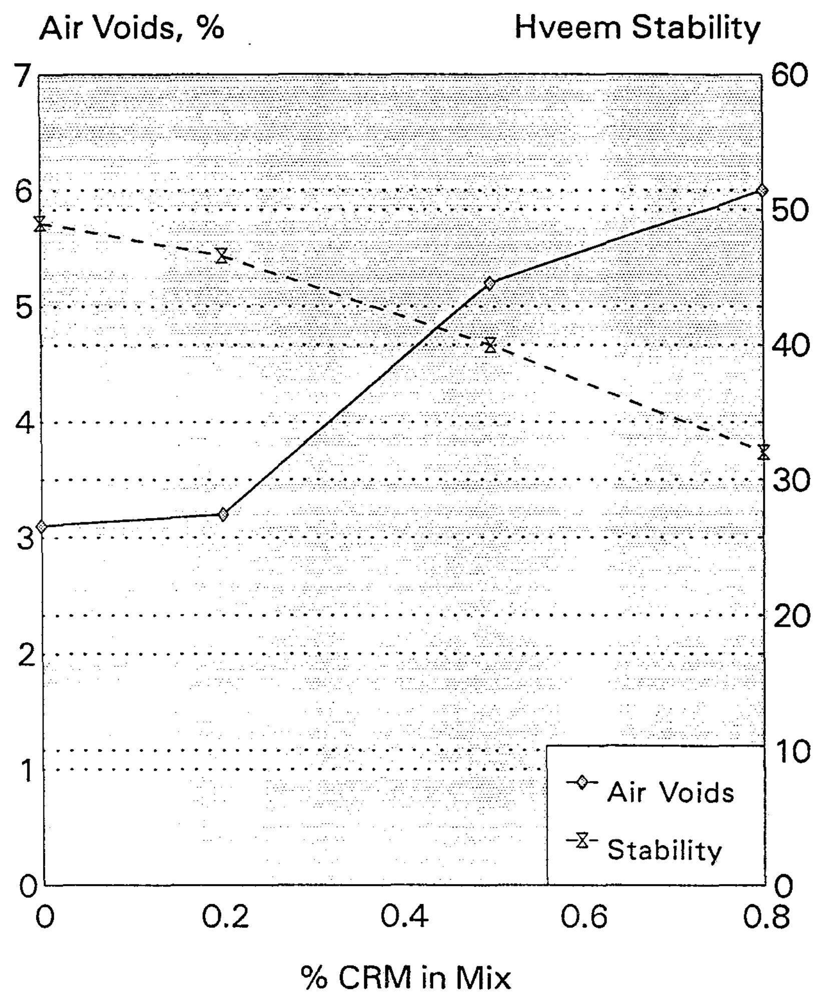

Figure F1. Effect of CRM on Hveem Stability and Density for Dense-Graded Mixtures.

Plant Production, Laydown, and Compaction Procedures

There are several components in the hot-mix asphalt construction process that are affected by crumb rubber modified asphalt cement. Much of what is addressed here was based on the construction of three crumb-rubber paving jobs conducted during research study 1332. All of these jobs made use of the wet process with about 17 to 18 percent rubber in the binder and the gradation of the mix was that of a CMHB. Many department and contracting personnel were interviewed regarding their perception of the operation and it should be noted that for most people the CMHB was as "unique" as working with CRM. Therefore, some of the elements of the construction process which were affected in these jobs may be due to the gradation of the mixture as much as the addition of CRM.

Elements which may be affected by the use of crumb rubber-modified asphalt binder include

- Binder Handling,
- Binder-Aggregate Mixing,
- Surge Storage,
- Transportation,
- Placement, and
- Compaction.

Binder Handling

Crumb rubber modified binders are produced by blending asphalt and ground tire rubber in a mixing tank. During the blending process, an interaction between the CRM and the asphalt binder occurs. For typical crumb rubber-modified binders, this reaction takes about 45 minutes; however, there are other variables which can alter the amount of time needed for reaction. The reaction can be affected by the temperature at which the blending occurs, the length of time the temperature remains elevated, the type and amount of mechanical mixing energy, the size and texture of the CRM, and the aromatic content of the asphalt cement (Heitzman 1992). The following guidelines regarding binder handling are offered below.

Crumb Rubber Modified Binder Mixing and Reaction Procedure
International Surfacing, Inc. recommends the following procedure.

> *Asphalt Cement Temperature:* The temperature of the asphalt cement shall be between 190°C (375°F) and 220°C (425°F) at the addition of the granulated rubber.
>
> *Blending and Reacting:* The asphalt and CRM shall be combined and mixed

together in a blender unit, pumped into the agitated storage tank, and then reacted for a minimum of 45 minutes from the time the CRM is added to the asphalt cement. Temperature of the crumb rubber blend should be maintained between 165°C (325°F) and 190°C (375°F) during the reaction period.

Transfer: After the material has reacted for at least 45 minutes, the crumb rubber blend shall be metered into the mixing chamber of the asphalt concrete production plant at the percentage required.

Delays: When a delay occurs in binder use after its full reaction, the blend shall be allowed to cool. The blend shall be reheated slowly just prior to use to a temperature between 165°C (325°F) and 190°C (375°F), and shall be thoroughly mixed before pumping and metering into the hot plant for combination with the aggregate. The viscosity of the blend shall be checked by the supplier. If the viscosity is out of range, the blend shall be adjusted by the addition of either asphalt cement or CRM as required to produce a material with the appropriate viscosity.

Blending of the crumb rubber binder can be accomplished away from the plant but provisions must be made for continuous heating of the binder so that temperatures do not fall below 165°C (325°F).

As mentioned previously, blending will generally take a minimum of 45 minutes. Because of this batching requirement, a bottle-neck can easily develop during HMA production. Therefore, provision should be made at the plant to store an adequate quantity of binder so that disruption of the HMA production does not occur (Roberts et al. 1989).

The density of CRM is greater than asphalt cement; therefore, settlement of the CRM can occur without continuous, adequate mixing. This should be noted by inspectors and samples of the reacted binder should be taken at the top and bottom of the tank to determine if the viscosity is uniform.

Binder-Aggregate Mixing

After the blending of asphalt and CRM is completed, mixing with aggregates can proceed as with conventional HMA; however, temperature of the binder should remain above 165°C (325°F). Either a drum or batch plant can be used.

In the field projects evaluated in study 1332, the mixing plants were operated anywhere from 150°C (300°F) to 160°C (325°F). In one project, the plant started out the job operating at 165°C (325°F) to 171°C (340°F); however, the mat appeared to be flushing. Therefore, the plant temperature was dropped to 155°C (310°F) to correct this problem.

Surge Storage

No experience has been reported with heated silo storage of crumb rubber modified hot mix. Although no difficulties should be encountered if temperatures are maintained, the high viscosity of these binders, and consequently mixtures, makes long-term storage in silos somewhat impractical, and potentially risky (Roberts et al. 1989). It is recommended to store mixtures for surge purposes only when using drum mixing plants.

One plant operator reported that acceptable surge storage time for conventional mixtures is 1½ to 2 hours; however, he suggested a maximum of ½ hour for the crumb rubber mixture. The crumb rubber mixture has higher binder contents and tends to be "stickier" and can build up on the gates to the storage tank.

Transportation

Conventional rear or bottom dump hauling equipment can be used to transport mix to the roadway. Consideration should be given to the use of coverings on the trucks to prevent rapid mixture heat loss during cooler weather or long haul distances.

For the field projects evaluated in study 1332, haul distances were relatively short and ambient temperatures were higher than 27°C (80°F). A CRM mixture placed in Lufkin in July and August lost 3 to 6°C (5 to 10°F) on a 30-mile haul. A CRM mixture placed in September near Abilene lost about 11°C (20°F) on a 10-mile haul.

After discharge of the mixture and prior to reloading, truck beds should not be sprayed with diesel fuel or other petroleum distillates to prevent sticking. In fact, it has been reported that these diluents cause a reaction with the crumb rubber binder and actually promote adhesion between the mix and truck bed (Roberts et al. 1989). Instead, a mixture of lime water, soap solution or silicone emulsion is recommended.

Placement

Crumb rubber-modified hot mix should be placed using conventional self-propelled laydown equipment equipped with a heated screed. The temperature of the mix at discharge from the screed should be not less than 138°C (280°F). The mixture temperature shall be measured in the truck just prior to dumping in the spreader.

On the CMHB CRM jobs constructed during study 1332, the mix compacted about 6 mm (1/4 inch) per 25 mm (1 inch) of compacted pavement. Therefore, to have a 50 mm (2 inch) thick compacted pavement, the mixture behind the screed should be 62 mm (2 ½inches) thick.

Project inspectors report that this type of mixture does not lend itself to raking and that joints cannot be raked or feathered. The leading edge of the screed should be right on the joint such that no raking is required.

The laydown machine and haul trucks should be coordinated to insure a steady consistent speed. Project inspectors reported that it was essential for the laydown machine to travel at a uniform speed.

If the laydown machine is stopped for more than 5 minutes, the laydown machine should be moved forward and the mix compacted.

Compaction

Crumb rubber modified asphalt paving mixtures should be compacted using conventional steel wheel rollers with a minimum weight of 9 metric tons (10 tons). The hot rubber binder in the mixture tends to stick to the tires of pneumatic rollers causing pick-up of the mix. Rollers should operate at a uniform speed of no more than 5 kmph (3 mph).

The nuclear density gauge should be used to aid in establish the necessary rolling pattern. It is recommended that the density be checked after each pass. All breakdown and intermediate compaction rolling should be performed with vibratory steel-wheel rollers operating at high frequency and low amplitude. One to two static passes may be needed to take out roller marks in pavement.

When placing CMHB CRM mixtures in relatively thin mats of 38 to 50 mm (1½ to 2 inches), the mat tends to cool quickly. This is due, in part, to the water used during the rolling process. Rolling should be completed at a mat temperature of 105°C (220°F) or more. This may be difficult to accomplish as inspectors reported that the mat loses as much as 14°C (25°F) per roller pass.

If necessary, the steel-wheel rollers can be wetted with plain water or soapy water to prevent pick-up of the mix.

Temperature of the pavement surface should be less than 52°C (125°F) before traffic is allowed to prevent pick-up of the mix. If this temperature needs to be increased to allow traffic sooner, it may be necessary to use blotter sand at an application rate of 0.5 to 1 kg/m^2 (1 to 2 lbs/yd^2).

Trial Section

It is recommended that a trial section of at least 90 metric tons (100 tons) of mix be constructed off site to examine the mixing plant process, control, placement procedures, compaction patterns, and to calibrate the nuclear density device.

Quality Control

Blending Time and Temperature

The contractor shall maintain records indicating for each batch of crumb rubber binder produced the quantity of asphalt cement, the temperatures of the asphalt cement and crumb rubber blend, the amount of other additives used and the quantity of rubber used. As a minimum, the following information should be recorded for each batch of crumb rubber binder:

Asphalt Cement Temperature
Asphalt Cement Specific Gravity
Specific Gravity of Asphalt Cement Corrected for Temperature
Quantity of Asphalt Cement Used in Batch
Quantity of CRM Used in Batch
Time CRM First Added to Batch
Time All CRM Added to Batch
Batch Completion Time
Batch Holding Temperature
End Holding Time

Viscosity of the Binder

Samples of the binder should be sent to the Materials and Tests Division for viscosity verification as directed. Field viscosity should be monitored for each batch of crumb rubber binder by either the contractor or TxDOT personnel. Field viscosity should be measured using the Haake viscometer or equivalent rotational viscometer using the protocol described in this report. Viscosity should be measured for each batch at the time of the batch completion and again if the binder is held over more than 2 hours prior to plant use.

Binder Content in the Mix

Conventional extraction procedures cannot be used to verify crumb rubber binder in a mixture. Binder content should be monitored using the nuclear asphalt content gauge which has been properly calibrated to the job mix. It should be noted that a California study (Dotz 1988; Epps 1994) concluded that nuclear gauges should not be used to determine total binder content in crumb rubber mixtures. However, for the field projects evaluated in study 1332, the nuclear asphalt content gauge appeared to be adequate.

Frequency of testing should be as on any conventional paving job.

Rubber Content in Mix

At this time, the best way of monitoring the quantity of rubber in a mix is to monitor the quantity of rubber which is used in the production of the binder. The contractor shall maintain records indicating for each batch of crumb rubber binder produced the quantity of asphalt cement, the amount of anti-strip or other additives used and the quantity of crumb rubber used. This information should be provided to the department on a daily basis.

Moisture Content in Aggregate

Use standard procedures.

Gradation of Aggregate

Use standard procedures.

Gradation of Rubber

The gradation of the crumb rubber should be checked at the time it is submitted for mixture design purposes. All of the CRM to be used for a particular job should be on-site at the time construction begins. The gradation of the shipment should be checked once at the beginning of the job and again on any additional shipments.

It was discovered, in this study, that it would be advisable to reserve a sample of the CRM at the time of the mixture design such that it can be visually compared with the CRM which is delivered for use in construction.

Please see the test protocol presented in this report for sieve analyses of crumb rubber modifier.

Mixing Temperature

Use standard procedures.

Laydown Temperature

The mixture temperature shall be measured in the truck just prior to dumping into the spreader.

Theoretical Maximum Specific Gravity

In this study, it was determined that accurate measurements of maximum theoretical specific gravity are difficult to obtain. Because this is a very important laboratory result for controlling mixture density, it is recommended that this test be performed with more frequency than normal. If possible, it is recommended that each reported test value of theoretical maximum specific gravity be an average of two tests. If the difference in the two test results range more than 0.011, a third test should be performed.

Care should be taken to ensure separation of the particles prior to testing so that the particles of the fine aggregate portion are not larger than 6 mm (1/4 in). These crumb rubber mixtures tend to be much stickier than conventional mixtures; therefore, this process may take as long as 30 minutes.

In-Place Density

Use standard procedures.

Additional Notes

Laboratory handling of this type of mixture can be more cumbersome due to the sticky nature of the mix. Some tips offered by laboratory personnel follow:

- Prior to molding samples, spray a light coating of vegetable oil on laboratory molds then wipe clean with paper towel.
- For shipment of roadway cores or laboratory molded samples, wrap samples in plastic wrap rather than newspaper to prevent sample from sticking to the packaging.
- When molding laboratory samples, do not put a paper gasket in the bottom of the mold but put 2 gaskets on top of the sample and then remove while sample is still hot.

Appendix G
Recommended Testing Protocol for CRM and CRM Binders

BROOKFIELD VISCOSITY TEST FOR CRUMB-RUBBER MODIFIED BINDERS

Scope

This method describes the determination of the viscosity of crumb rubber modified binder at 175°C using a Brookfield rotational viscometer. This method is a modification of ASTM D2994 and can be used for crumb rubber modifier contents up to 18% by weight of the binder.

Apparatus

1 A Brookfield rotational viscometer, LV model with spindles RV3 and RV6.
2 A glass thermometer graduated in 0.5°C subdivisions.
3 Forced draft oven capable of maintaining a temperature of 200°C plus or minus 2°C.
4 600 milliliter glass beaker.
5 A stirring rod or spatula capable of mixing and stirring the viscous binder.

Method

Fill the 600 ml beaker with about 500 ml of pre-blended crumb rubber binder. Heat the sample to about 5°C above the test temperature in a forced draft oven. Hand-stir sample 2 to 3 times throughout the heating process. When sample reaches about 180°C, remove from oven. Stir sample vigorously and insert the selected spindle as shown in the tabulation. Rotate the spindle at 12 revolutions per minute (rpm) while allowing the temperature of the sample to drop to the test temperature of 175°C.

When the sample reaches 175°C, stop the viscometer. Hand-stir the sample briefly, actuate spindle and take a reading 30 seconds later.

Crumb Rubber Content, % by weight of binder	Spindle No.	Revolutions per Minute
10% or less	RV3	12
more than 10%	RV6	12

Reporting

Viscosity readings should be taken and recorded accurately to 0.1 Pa s. Report the results along with date, time and binder temperature.

PROTOCOL FOR MEASURING THE ZERO SHEAR VISCOSITY USING A SHEAR RHEOMETER

This method described the determination of the zero shear viscosity (η_o) for unaged, aged and crumb rubber modified (CRM) asphalt. The material is subjected to a constant stress and maintained for a period of time during which period the creep flow behavior is observed. η_o is calculated from the deformation data.

Description of Terms

$\gamma(t)$ is the deformation as a function of time
σ_0 is the constant stress applied at time t=o
$J(t)$ is the creep compliance as a function of time
J_o is the instantaneous elastic compliance
J_e is the steady state compliance
η_o is the zero shear viscosity or steady state viscosity of the material.

Test Method

1 Test is done in the shear mode with material between two parallel plates.
2 η_o is obtained between 25°C and 60°C.
3 25 mm plates are used. Gap is set at 1mm for unaged or unfilled asphalt and 2 mm of aged or filled asphalt
4 Test temperature is maintained within 0.1°C of the set temperature.
5 Appropriate constant stress is chosen to conduct the test within linear viscoelastic region.

Apparatus

1 A dynamic shear rheometer (DSR) system consisting of parallel plates is used in the shear mode. Accessories include an environmental chamber, a loading device and data acquisition system.

2 Constant stress is chosen to perform the creep test. Stress imposed is dependent on the machine capability which varies according to the gap set between the parallel plates. To perform the test within the linear viscoelastic range, the stress is chosen closer to the lower limit.

3 The test is complete (steady state is reached) when m=dln(J)/dln(t)=1. In practice this is not always possible within a reasonable time period. Hence m can be chosen (0.7 to 0.9) in the instrument and a conventional method is used to extrapolate the data with fairly reasonable accuracy.

Preparation of Apparatus

1 The test plates are mounted on the fixture and firmly tightened.

2 Test temperature is selected at which the test is to be performed to find η_o. Allow DSR to reach within 0.1°C of the set temperature.

3 At the test temperature, zero gap level is set by manually spinning the moveable plate and closing the gap until the removable plate touches the fixed plate. The zero gap is reached when the plate stops spinning completely.

4 The plates are moved apart and the gap is set (1 mm or 2 mm)

5 The sample (previously poured out in a silicone mold) is applied in between the plates.

6 If testing at 25°C, the sample is warmed to about 40°C to remove any preload in the sample. The sample is brought down to the test temperature.

Interpretation of Results

The value of m is continuously calculated by the instrument and is shown on the computer screen. For m=0.7 to 0.9, which is far removed from steady state, a conventional method can be used here to calculate η_o. Creep compliance in shear at the steady state, denoted by J(t) can be written in the form,

$$J(t) = J_e + t/\eta_o$$

where t is the time. This equation can be written in the alternate form,

$$J(t)/t = J_e/t + 1/\eta_o$$

Therefore when J(t) is plotted against 1/t, the intercept at the ordinate gives the value of $1/\eta_o$. An example calculation for finding η_o is shown in Figures 1 and 2. The creep data given in Figure 1 is plotted as J(t) vs. 1/t in Figure 2 where a straight line is fit for data with 1/t tending to zero. The η_o is found to be 7.78E4 Pa s.

Report

The report should include the following:

1 Temperature of test.

2 Gap size between the parallel plates.

3 Constant stress imposed.

4 m at which the test was stopped.

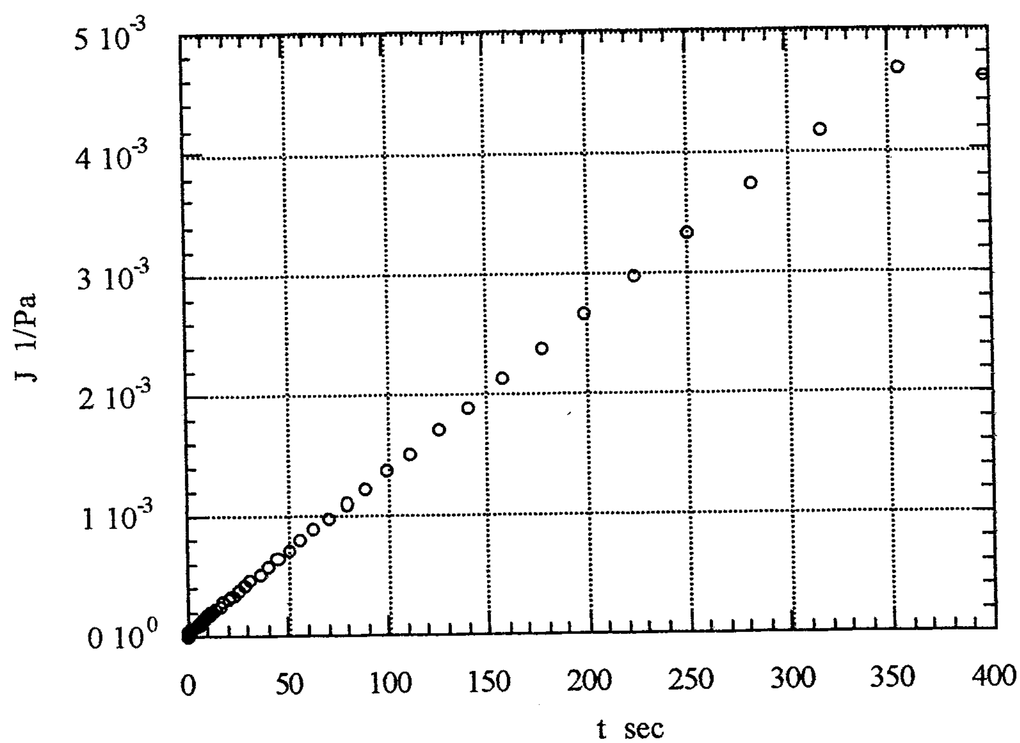

Figure 1. Creep data for unaged asphalt at 25°C for a constant stress of 10000 Pa.

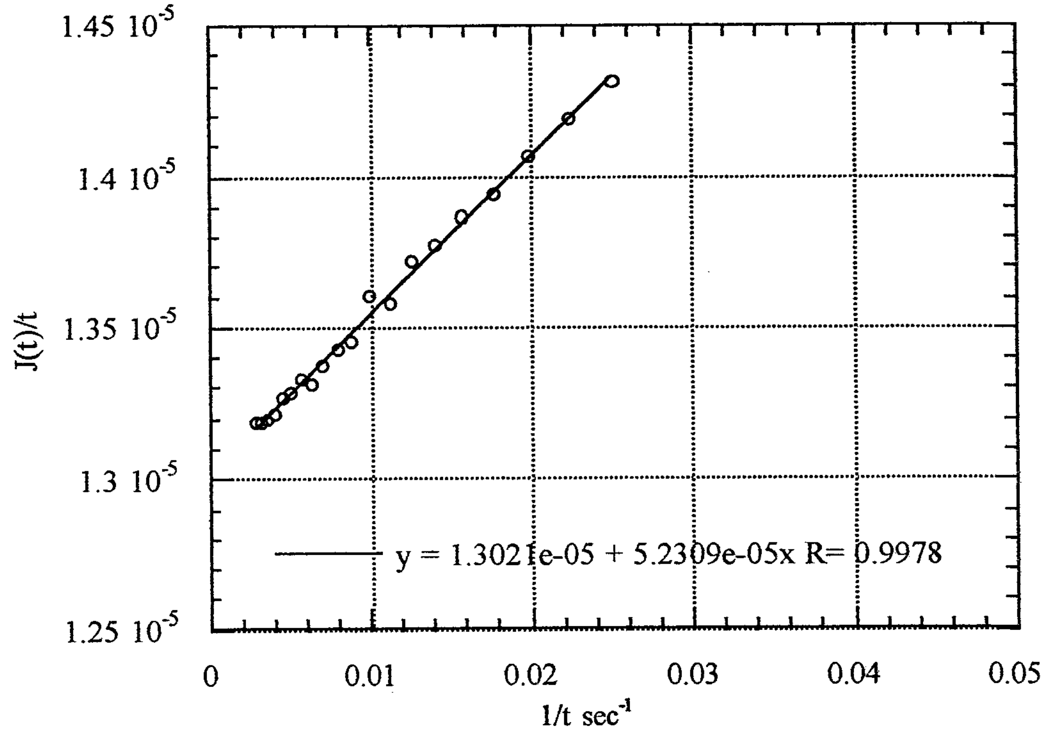

Figure 2. An example curve of J(t)/t vs. t at 25°C (for unaged asphalt) closer to the ordinate axis to find the intercept.

FIELD VISCOSITY OF CRUMB RUBBER MODIFIED BINDER USING HAND-HELD ROTARY VISCOMETER (HAAKE VISCOSITY)

Scope

This method describes the determination of the field dynamic viscosity of crumb rubber binders using a hand-held, battery operated, rotary viscometer. This test method is adapted from Method BR5 T, Manual 3, Revised Edition 1992, Southern African Bitumen and Tar Association, Roggebaai, South Africa.

Apparatus

1 A hand-held rotary viscometer with a rotor-cup to measure dynamic viscosity in the range of 0.5 to 10 Pa s, accurate to 0.1 Pa s.

2 A 0 - 300°C thermometer accurate to 1°C.

3 Asbestos gloves.

4 A stirring rod or spatula capable of mixing and stirring the viscous crumb rubber binder.

5 Metal tins or glass beakers with capacity of approximately one liter and with a diameter of at least 100 mm allowing a sample depth of at least 80 mm. If the tins are also used as sample containers, they should have tightly fitting lids.

Method

Take a representative sample of the crumb rubber binder. Stir sample well with suitable stirring rod. Measure and record the temperature.

Attach the correct viscosity cup-rotor to the viscometer such that viscosity measurements will be between 0.5 and 10 Pa s. Ensure that the vent hole on top of the rotor is open. Place the sampling tin or beaker on a firm non-heat absorbing base and immerse the viscometer rotor in the center of the sample up to the depth mark on the stem. After approximately 30 seconds, start the rotation of the rotor while holding the instrument absolutely horizontal. Take and record the viscosity reading to an accuracy of 0.1 Pa s approximately 10 seconds after the rotor is set in motion. Remove and clean the rotor.

Reporting

The viscosity readings should be taken and recorded accurately to 0.1 Pa s. Report the results to the same accuracy together with the date, time, sampling position and binder temperature.

Notes

To prevent damage to the instrument some manufacturers stipulate that the viscometer is set in motion before the cup is immersed. In order to comply with this requirement, while still allowing for the cup to heat up to the temperature of the crumb rubber binder, it is suggested that the cup is immersed for 30 seconds, pulled more than halfway out of the binder, the motion started and the cup immersed to the mark. The viscosity reading should then be taken approximately 10 seconds after this second immersion.

Clean the rotor-cup as soon as possible after the test, while it is still hot, by first wiping excess material off with a dry cloth or paper and then cleaning it in a suitable solvent.

SIEVE ANALYSIS AND LOOSE FIBER CONTENT OF CRUMB RUBBER

Scope

In this method a dry sieve analysis is carried out on the crumb rubber intended for use in crumb rubber binder. The loose fibers are collected during the sieving operation as a rough indicator of fiber content. This test method is adapted from Method BR6 T, Manual 3, Revised Edition 1992, Southern African Bitumen and Tar Association, Roggebaai, South Africa.

Apparatus

1 Appropriate test sieves of the sizes called for in specifications.

2 A suitable nylon or bristle sieve brush.

3 A balance accurate to 0.01 g to weigh up to at least 200 g.

Method

Obtain a representative sample of approximately 500 g of crumb rubber either from randomly selected individual bags as delivered on site or from the rubber crumbs container at the blending or mixing plant at random intervals.

Mix the sample thoroughly and break down any lumps that it may contain. Scoop out duplicate test samples of more or less 50 g each and test each sample as follows:

> Nest the sieves on the receiver in descending order of size and transfer the sample to the top sieve. Place the lid in position and hand sieve the sample for approximately two minutes by rocking and tapping the sieves. Mechanical sieving can be used for this initial operation especially when the presence of fibers is suspected, but the final sieving should be done by hand.

Remove the lid and gently rub the rubber crumbs in the uppermost sieve. Hand sieve the material until nothing more passes that specific sieve. If fiber is present it can be clearly seen in the sample as short light colored hairs. These will form a ball during the sieving operation. Collect the bier balls carefully and place them in a clean receptacle. Remove the sieve and repeat the operation for each sieve in the series. For 1.18 mm sieves and finer, the fibers collected on a specific sieve are placed back on the next coarser sieve and resieved to remove rubber particles sticking to the fibers. The fibers are then carefully collected and placed in the fiber receptacle.

Weigh the material retained on each sieve accurately to 0.01 g. Also determine the mass of the fibers collected from the sieves.

Reporting

Calculate the percentages passing the sieves. Calculate the fiber content as a percentage of the total sample by mass and report together with the grading results.