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16. Abstract

CRM binders were fabricated in the laboratory and evaluated according to ten different binder tests. Some of the test procedures routinely used for CRM binders were determined to have no apparent relationship to mixture properties or field performance. The SHRP binder tests were also used to characterize CRM binders. The SHRP direct tension test and, to a lesser degree, the force-ductility test appear to measure CRM binder characteristics, which are attributed to improved cracking performance in CRM mixtures.

Nine CRM mixtures were evaluated using AAMAS characterization procedures: four wet-process mixtures, four dry-process mixtures, and one control mix. It was determined that CRM has the potential to significantly improve the fatigue and thermal cracking performance of asphalt concrete pavements, but only when the wet method is used and the binder is properly designed. The dry process should produce mixtures with reduced propensity for rutting, but may have an adverse effect on cracking.

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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LABORATORY EVALUATION OF CRUMB-RUBBER MODIFIED (CRM) BINDERS AND MIXTURES

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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.

Further recommendations regarding guidelines and specifications will be provided in this study's subsequent research report: 1332-2F.

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

This report summarizes the laboratory results of the first 15 months' research effort for this study. The report contains three major sections: evaluation of crumb rubber modified (CRM) binders, CRM mixture design, and CRM mixture performance evaluation.

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).

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.

Nine CRM asphalt concrete mixtures were evaluated in the laboratory: four wet-process mixtures, four dry-process mixtures, and one control mix. Three of these mixtures were designed according to the standard TxDOT (C-14 Bulletin) design procedure. A control (Type D) mixture was designed according to this method and two mixtures incorporating crumb rubber (added dry as part of the aggregate): dense-graded with fine CRM (DGF) and dense graded with coarse CRM (DGC). These two mixtures contain the maximum amount of crumb rubber that could be added while still conforming to standard mixture design criteria.

A more detailed summary of this report is contained in Chapter 1 on Page 1.

1

Summary

This report summarizes the laboratory results of the first year's research effort for this study. The report is divided into three major sections: evaluation of crumb rubber modified (CRM) binders, CRM mixture design, and CRM mixture performance evaluation.

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. Both of these types of methods have been included in this study. 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).

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. Some of the preliminary conclusions regarding the binder study are as follows:

- At concentrations of fine (-#80) CRM above 10 percent, something happens to markedly enhance the failure strain (in the direct tension test), and thus ostensibly increase resistance to pavement cracking. We believe that, at a certain concentration of rubber particles in the wet process, a three-dimensional network of rubber is created within the CRM 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 CRM asphalt system. It is this close proximity of the soft swollen particles that promotes the formation of the three dimensional network.
- Some of the specification values which are typically used for CRM binders tend to be product-oriented, in that not all binders from wet-processes or technologies which incorporate different concentrations or gradations of CRM meet these specifications. Furthermore, the specified criteria are not necessarily related to good pavement performance.
- The following tests are routinely performed on CRM binders with a reasonable degree of repeatability; however, these test results have no apparent relationship to mixture properties or field performance. Some of these tests may best be used to qualify or specify a particular type of binder or technology.

  Penetration at 77°F (25°C), 100 g, 5 sec.;
  Penetration at 39.2°F (4°C), 100 g, 5 sec.;
  Penetration at 39.2°F (4°C), 200 g, 60 sec.;
  Ductility;
  Resiliency; and
  Softening Point.
- SHRP bending beam, direct tension, and SHRP Pressure Aging Vessel (PAV) tests were successfully performed on the CRM binders without modification. However, repeatable results for the dynamic shear rheometer test have not yet been obtained. This test may require modification.
- The SHRP direct tension test and, to a lesser degree, the force-ductility test

appear to measure CRM binder characteristics which are attributed to improved cracking performance in CRM mixtures.

- Viscosity of CRM binders may be measured with the Brookfield viscometer with reliability; however, a strict adherence to a specified test protocol is required. This test may also be used as a quality control test for monitoring CRM concentration.

Nine CRM asphalt concrete mixtures were evaluated in the laboratory: four wet-process mixtures, four dry-process mixtures, and one control mix. Three of these mixtures were designed according to the standard TxDOT (C-14 Bulletin) design procedure. A control (Type D) mixture was designed according to this method and two mixtures incorporating crumb rubber (added dry as part of the aggregate): dense-graded with fine CRM (DGF) and dense graded with coarse CRM (DGC). These two mixtures contain 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.

Six of the nine mixtures evaluated in the laboratory were designed according to TxDOT's recently developed mixture design procedure for asphalt-rubber mixtures. These mixtures are gap-graded and similar in gradation to a stone matrix-type mixture. These six different mixtures include both wet and dry processes for adding the crumb rubber to the mix. Rubber concentration was varied, and for comparison purposes all rubber concentrations are expressed as a percent by weight of the asphalt, whether a dry or wet process was used. Another variable included here is the size of the rubber: fine (-#80 mesh) or coarse (-#10 mesh). These six mixtures are 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).

Some of the preliminary conclusions regarding the mixture study are listed below.

- Field performance of dense-graded mixtures containing CRM has generally been poor; however, the concentration of rubber in these mixtures has been at levels of 18% (by weight of asphalt) or greater. Results from this laboratory study indicate that acceptable performance may be obtained with CRM in dense-graded mixtures at lower concentrations of rubber (no more than 10% by weight of asphalt). The dense-graded laboratory mixtures evaluated in this study contained CRM added dry, as part of the aggregate.
- TxDOT's (volumetric) mixture design procedure for asphalt-rubber mixes generally produces mixtures that can be considered very rut resistant. This method can be used to incorporate rubber of any size or process (wet or dry). It is particularly effective for larger concentrations of rubber (10% or more, by weight of asphalt).
- CRM can be incorporated into hot-mix asphalt concrete in any way (dry or wet, coarse or fine) without having a detrimental effect on rutting performance as long as the mixture is designed properly.
- CRM has the potential to significantly improve the fatigue and thermal cracking performance of asphalt concrete pavements, but only when the wet method is used and the binder is properly designed. A significant improvement in fatigue and thermal cracking characteristics was observed with one particular mixture: 18%FW. This data is also strongly supported by some of the binder properties for this blend, in particular, the direct tension test on the binder at 14°F (-10°C). It is believed that this improved performance is related to the concentration and distribution of the rubber particles in the binder. It is expected that the optimum concentration of rubber particles occurs at a lower level for the fine rubber than

for the coarser rubber. This improved performance may be observed with the coarser CRM but would occur at a concentration level higher than 18% which was the maximum level evaluated thus far in this study.

The wet process should produce asphalt mixtures (if properly designed) that inhibit cracking and *may* inhibit rutting. The dry process, on the other hand, should produce mixtures with reduced propensity for rutting but may have adverse effects on cracking. In the dry process, the rubber exists as discrete particles. Discrete particles in asphalt will normally intensify the propensity for cracking but may enhance rutting resistance.

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. That is, if a mixture normally performs satisfactorily, additional funds should not be expended on unnecessary additives. When polymer additives, including tire rubber, are used to reduce cracking, they should be blended with asphalts softer than the usual grade. On the contrary, when these materials are used to reduce rutting, they should most likely be used to stiffen the asphalt grade that normally produces the rutting mix.

2

Laboratory Evaluation of Binders

The objectives of the binder laboratory study were: (1) to prepare crumb rubber modified binders for use in the mixture study; (2) to characterize these crumb rubber modified binders; (3) to determine what test procedures are appropriate for CRM binders (in particular as related to mixture properties); and (4) to develop test protocol. Final recommendations regarding all of these objectives will be made in the final report; however, this chapter discusses preliminary analyses.

Binders were fabricated in the laboratory using a torque-fork mixer. A laboratory mixer of this type was first used for crumb rubber binder blending in 1977 (Pavlovich et al. 1979). The system consists of a constant speed motor with stirrer assembly which is capable of recording torque changes as load varies on the stirrer. The resulting apparatus is a rotational viscometer which can measure relative changes in fluid viscosity during mixing. Also, this device uses a mixing propeller for agitation and is primarily intended to be a mixer.

Two sources of CRM and three CRM concentrations were used to fabricate CRM binders in the laboratory for a total of six different binders as follows:

CRM Concentration, % by weight of Asphalt	-#10 Mesh(Coarse) CRM	-#80 Mesh (Fine) CRM
4%	X	X
10%	X	X
18%	X*	X

* Note: This binder would be of the type commonly marketed by International Surfacing, Inc.

The asphalt used in this study was Texaco AC-10 and the rubber was from two sources: the -#10 mesh CRM was from Granular Products in Mexia, Texas, and the -#80 mesh CRM was from Rouse Rubber in Vicksburg, Mississippi.

The addition of CRM to asphalt can have a significant impact on material properties and both binder and mixture testing procedures. Some of the standard tests typically performed on conventional asphalt cement binders (such as absolute and kinematic viscosity, penetration, ductility, softening point) are either inappropriate for CRM binders or the test results are so different from standard asphalt cement their relevance is unclear.

International Surfacing, Inc. (ISI) of Chandler, Arizona markets the most commonly used "wet" process, also known as the McDonald technology. Most state DOTs do not have adequate data or experience with CRM binders to have developed their own specifications and, therefore, normally use the binder specifications which are recommended by ISI. Recommended specifications for the ISI (Type II) binder are shown below:

•	Apparent Viscosity, 347°F, Spindle 3, 12 RPM, cps (ASTM D2669)	Min Max	1,000 4,000
•	Penetration, 77°F, 100 g, 5 sec, (ASTM D5)	Min Max	50 100
•	Penetration 39.2°F, 200g, 60 sec, (ASTM D5)	Min	25
•	Softening Point, °F	Min	120
•	Resilience, 77°F, % (ASTM D3407)	Min	10
•	Ductility, 39.2°F, 1 cpm: cm	Min	10
•	TFOT Residue, Penetration Retention, 39.2°F, %	Min	75
•	Ductility Retention, 39.2°F, %	Min	50

Some of these specification values tend to be product-oriented, in that other materials from wet processes or technologies which incorporate different concentrations or gradations of CRM may not meet these specifications. That is, the specified criteria are not necessarily related to good pavement performance.

The following tests were performed on the six different CRM binders fabricated in the laboratory as well as the control binder (Texaco AC-10):

- Penetration at 77°F (25°C), 100 g, 5 sec.,
- Penetration at 39.2°F (4°C), 100 g 5 sec.,
- Penetration at 39.2°F (4°C), 200g, 60 sec.,
- Ductility at 77°F (25°C) and 39.2°F (4°C),
- Force-Ductility,
- Brookfield Viscosity,
- Elastic Recovery,
- Softening Point,
- SHRP Bending Beam, and
- SHRP Direct Tension.

The above tests were performed in triplicate on each binder and a discussion of these test results follows.

2.1 Penetration

The penetration test is an empirical measure of asphalt consistency. In this test, a container of asphalt cement is brought to a standard test temperature in a temperature-controlled water bath. A prescribed needle is allowed to bear on the surface of the asphalt cement for a specified time period. The distance, in units of 0.1 mm, that the needle penetrates into the asphalt cement is the penetration measurement. Penetration measurements on control and crumb rubber modified binders are shown in Table 2.1.

Table 2.1. Penetration Data for Control and CRM Binders.

Binder	Penetration		
	77°F (25°C) 100 g, 5 sec	39.2°F (4°C) 200g, 60 sec	39.2°F (4°C) 100 g, 5 sec
Texaco AC-10	110	38	12
AC-10 + 4% Fine CRM	94	32	7
AC-10 + 10% Fine CRM	73	28	7
AC-10 + 18% Fine CRM	56	26	4
AC-10 + 4% Coarse CRM	79	29	4
AC-10 + 10% Coarse CRM	70	28	4
AC-10 + 18% Coarse CRM	47	22	3

According to these data, penetration at any of the test temperatures decreases with the addition of CRM and continues to decrease with increasing concentrations of CRM. Penetration at 77°F (25°C) data are plotted in Figure 2.1.

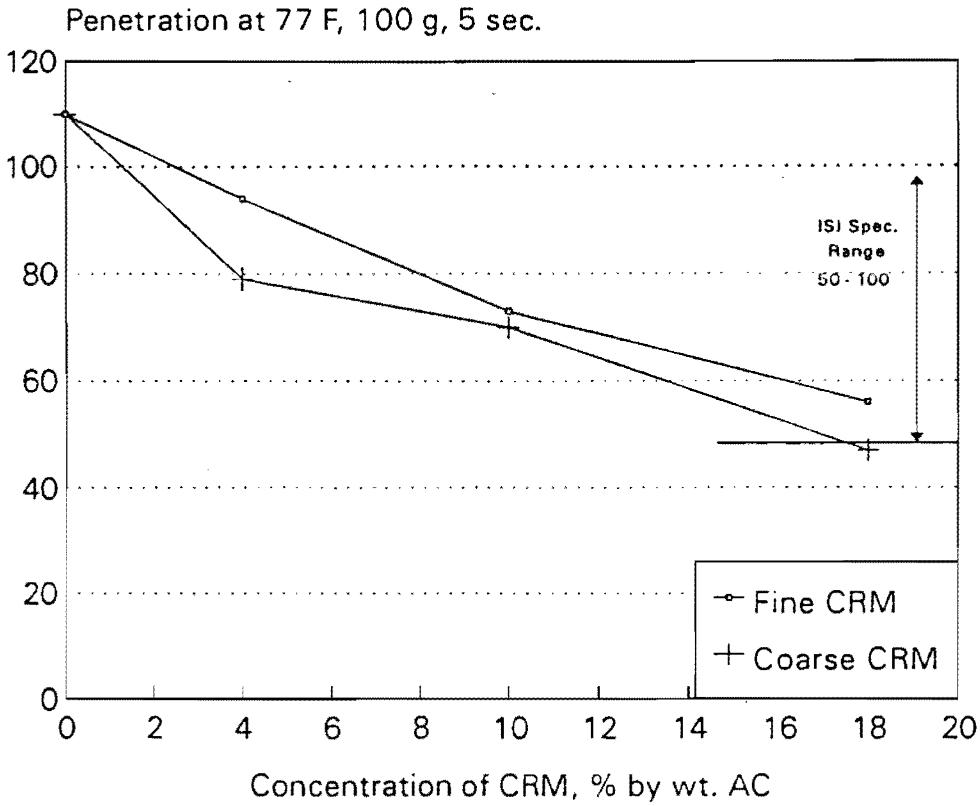


Figure 2.1. Penetration at 77°F (25°C), 100 g, 5 sec. for Control and CRM Binders.

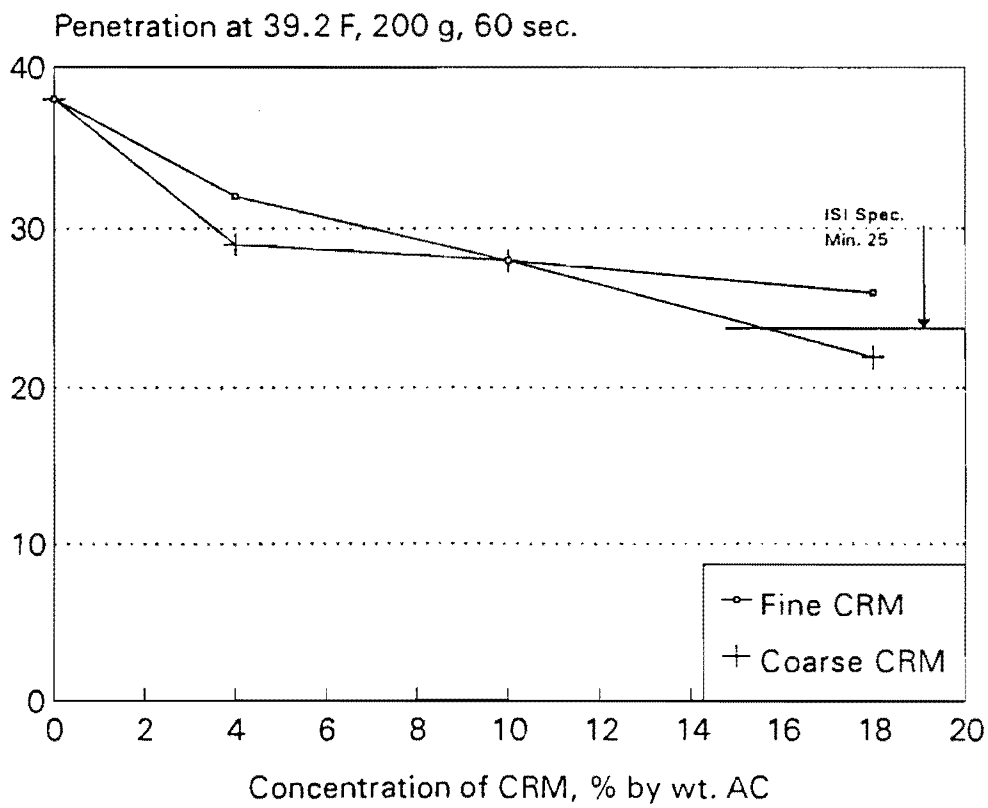


Figure 2.2. Penetration at 39.2°F (4°C), 200 g, 60 sec., for Control and CRM Binders.

According to the ISI Guide Specification (Type II), the penetration at 77°F (25°C) for asphalt-rubber binders which contain 15 to 20 percent rubber by weight of the mix should be between 50 and 100. All of the CRM binders except one (18% Coarse CRM) met this specification.

Penetration at 39.2°F (4°C) data is shown in Figure 2.2 along with the ISI Guide Specification. Again, all of the binders except one (18% Coarse CRM) met the specification.

2.2 Ring and Ball Softening Point

The ring and ball softening point test is used as the basic measurement of consistency for grading blown asphalts. It is also performed on CRM binders as shown previously in the guide specification recommended by ISI.

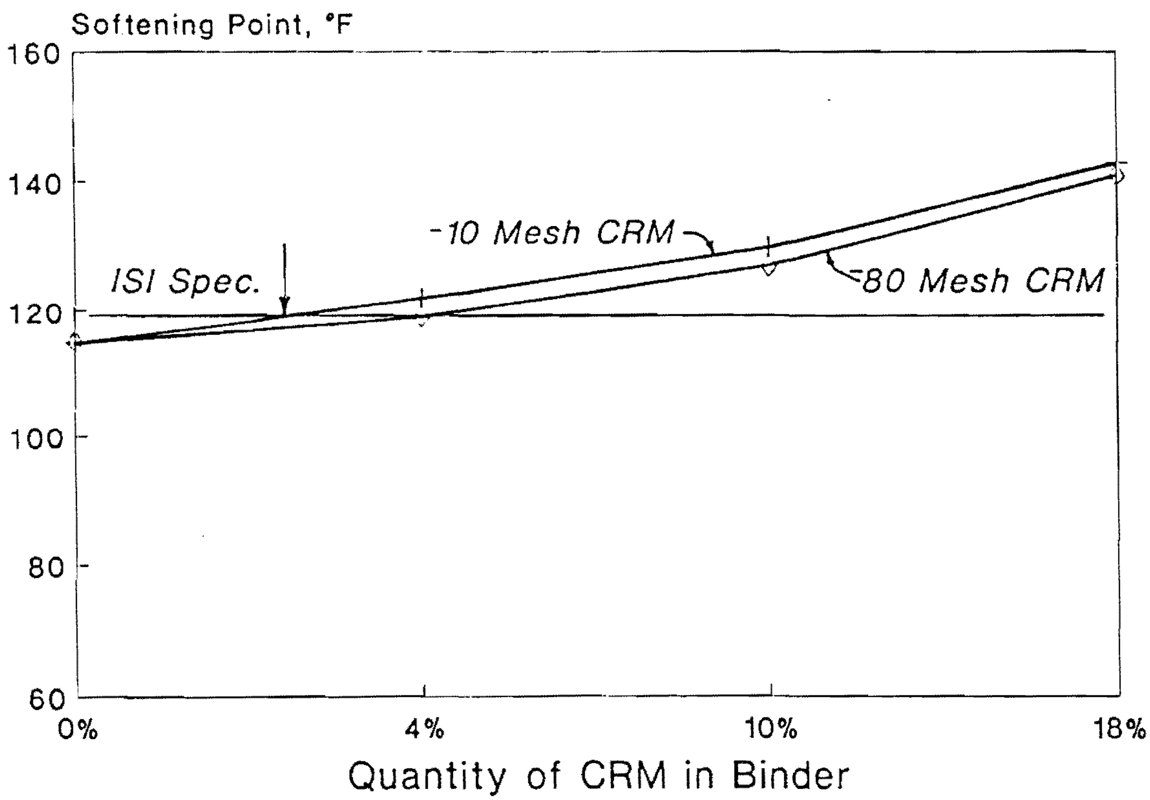


Figure 2.3. Ring and Ball Softening Point of Control and CRM Binders.

The softening point of CRM asphalt binders is higher than that of unmodified binder as shown in Figure 2.3 and is affected by both rubber size and concentration. Note that all of the CRM binders met the ISI specification.

2.3 Ductility

Some engineers consider ductility to be an important characteristic of asphalt cement. It provides a measure of the consistency of asphalt cement. The presence or absence of ductility, however, is usually more significant than the actual degree of ductility (*The Asphalt Handbook* 1989). The ductility of asphalt cement is measured by the distance to which it will elongate before breaking when two ends of a briquette specimen of the material are pulled apart at a specified speed and at a specified test temperature.

The ductility test is typically performed on asphalt cement residue after Thin Film Oven Testing (TFOT) and is measured at a test temperature of 77°F (25°C) and a speed of 5 centimeters per minute. TxDOT (*Standard Specifications* 1993) requires that an unmodified AC-10 have a ductility of at least 100 cm after Thin Film Oven Testing. International Surfacing, Inc. (*Guide Specification)* recommends that the ductility of asphalt-rubber be measured at 39.2°F (4°C) and one cm per minute and that under these conditions the ductility be a minimum of 10 cm.

In this study, ductility measurements were made on the CRM binders and the control binder (AC-10) both before and after TFOT. Measurements were made at 77°F (25°C) and 39.2°F (4°C). These data are shown in Tables 2.2 and 2.3.

From these data, it is observed that asphalt cement modified with CRM has a very low ductility. Figure 2.4 shows that even small quantities of CRM (4%) cause a dramatic decrease in the ductility of asphalt cement. Further increases in the quantity of CRM (up to 18%) do not continue to significantly decrease the ductility. It is also observed from this figure that the fine-graded CRM binders have a slightly higher ductility than the coarse-graded CRM binders.

Table 2.2. Ductility of Binders Measured at 77°F (25°C) and 5 cm Per Minute.

Binder	Ductility, cm.		
	Before TFOT	After TFOT	Retention, %
Texaco AC-10	120+	120+	100
AC-10 + 4% Fine CRM	25	22	96
AC-10 + 10% Fine CRM	21	22	105
AC-10 + 18% Fine CRM	21	20	95
AC-10 + 4% Coarse CRM	14	12	86
AC-10 + 10% Coarse CRM	12	12	100
AC-10 + 18% Coarse CRM	11	10	91

Table 2.3. Ductility of Binders Measured at 39.2°F (4°C) and 1 cm Per Minute.

Binder	Ductility, cm.		
	Before TFOT	After TFOT	Retention, %
AC-10	110	102	93
AC-10 + 4% Fine CRM	11	7	64
AC-10 + 10% Fine CRM	12	7	58
AC-10 + 18% Fine CRM	11	10	91
AC-10 + 4% Coarse CRM	8	6	75
AC-10 + 10% Coarse CRM	8	6	75
AC-10 + 18% Coarse CRM	7	6	86

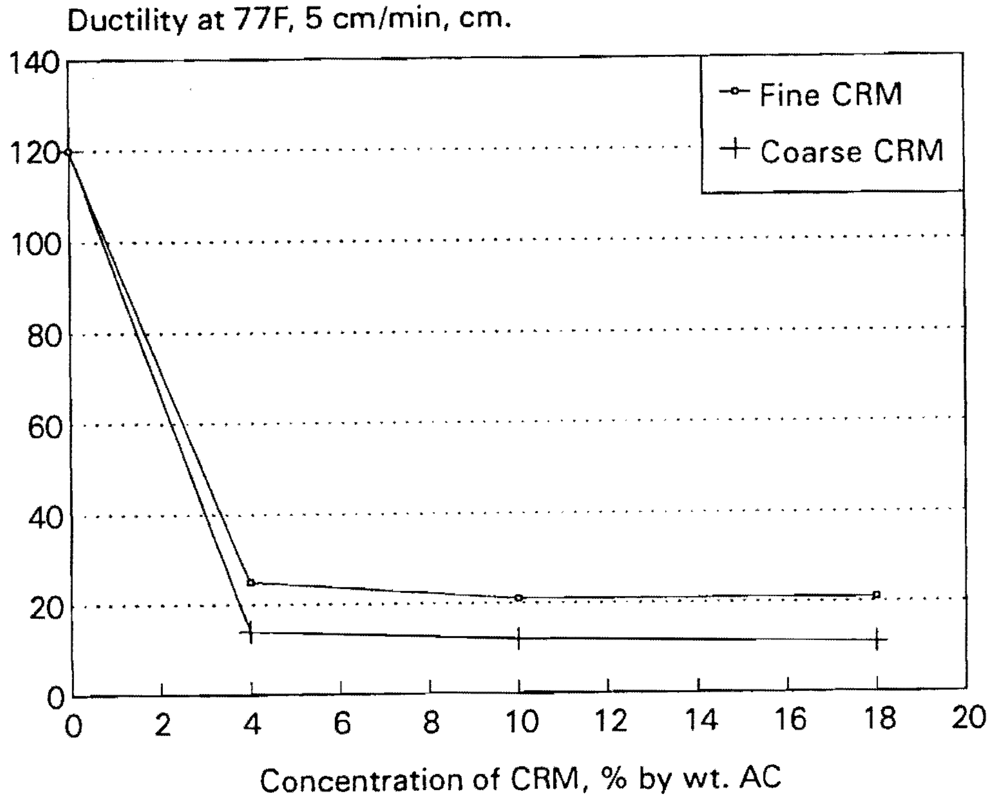
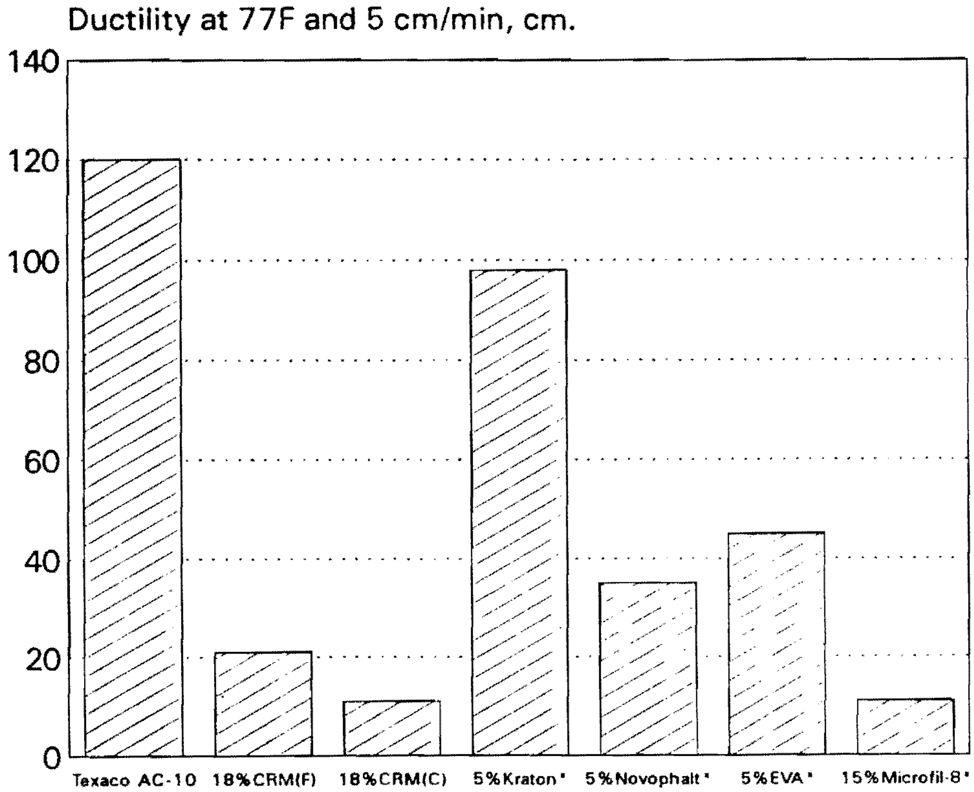


Figure 2.4. **Ductility at 77°F (25°C) of Control Asphalt Cement and Asphalt Cement Modified with Different Concentrations of Fine and Coarse CRM.**

Ductility may be described as the ability of asphalt cement to be drawn into a thin thread; thus, it is logical to assume that any discrete particles (such as CRM) which are blended into the asphalt cement would cause a decrease in ductility. It is, therefore, of interest to see how the ductility of CRM binders compares with other modified binders. Button and Little (1986) performed a comprehensive study of modified binders and some of these data are shown in Figure 2.5. With the exception of Kraton, most of the other modifiers shown here exhibit a relatively low ductility, in particular, the Microfil-8 (or carbon black) which is comparable with the CRM binders.

Ductility tests at 39.2°F (4°C) and one cm per minute are shown in Figure 2.6. As in the ductility tests at 77°F (25°C), increasing concentrations of CRM have little effect on ductility. According to the ISI Guide Specification, the ductility at 39.2°F (4°C) for a Type II CRM binder should be greater than 10 cm. The CRM binder produced with the fine rubber met this specification as shown in Figure 2.6.



* After Button and Little (1986). Note: Original, unmodified binder was Texaco AC-5.

Figure 2.5. Ductility of CRM Binders as Compared with Other Modified Binders.

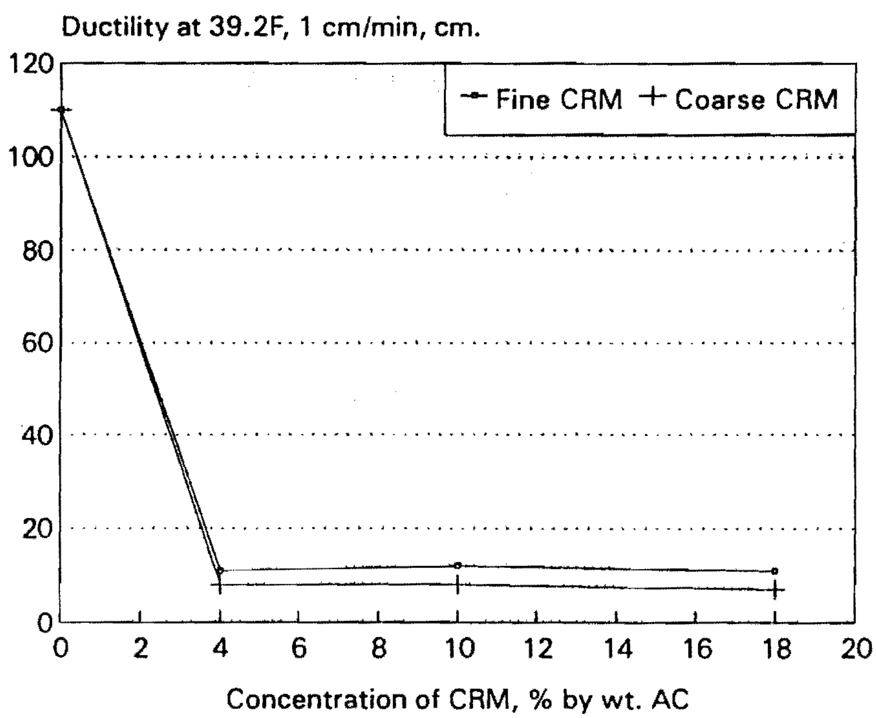


Figure 2.6. Ductility at 39.2°F (4°C) of CRM Binders Modified with Different Concentrations of Fine and Coarse CRM.

Ductility retention is another ISI recommended specification for CRM binders. Ductility retention is simply the ratio of the ductility measurement after TFOT to that before TFOT. These measurements are shown in Figure 2.7. The requirement is that the retention be at least 50 percent and all CRM binders met this requirement.

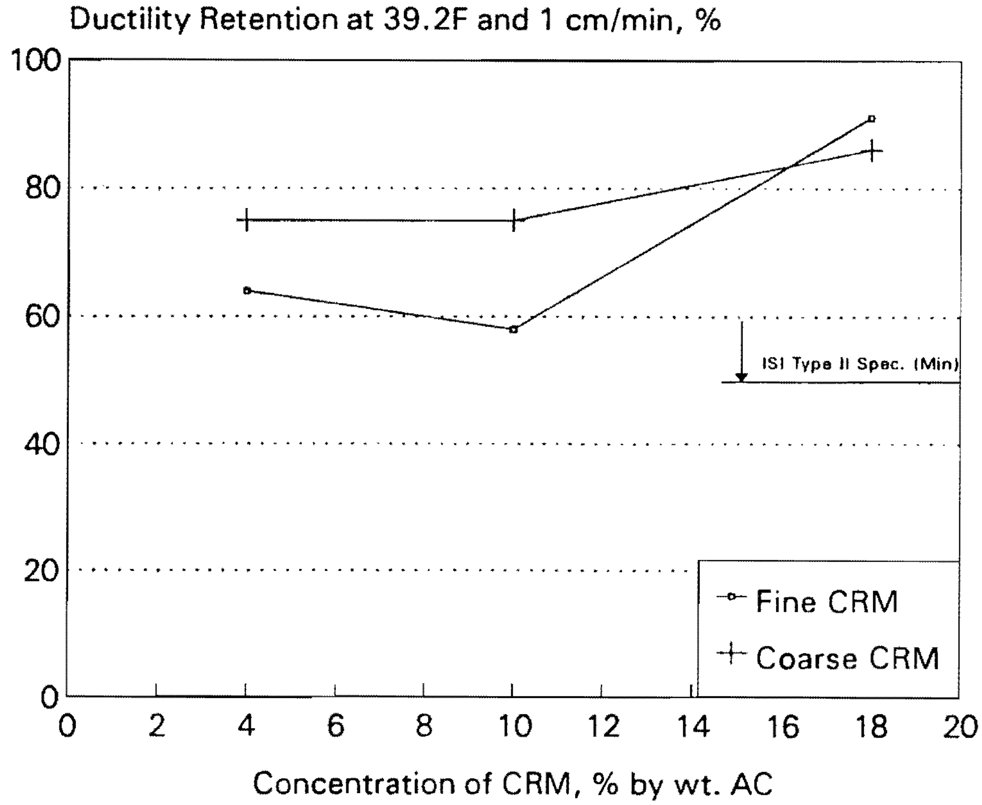


Figure 2.7. Ductility Retention of CRM Binders Modified with Different Concentrations of Fine and Coarse CRM.

2.4 Force Ductility

The force ductility test is a modification of the asphalt ductility test. The test has been described by Shuler et al. (1985) as a means to measure tensile load-deformation characteristics of asphalt and modified-asphalt binders.

The test is performed as described by ASTM D113 with certain changes. The principal alteration of the apparatus consists of adding two force cells in the loading chain. Specimens are maintained at 39.2°F (4°C) by circulating water through the ductility bath during testing. A second major alteration of the standard ASTM procedure involves the test specimen shape. The mold is modified to produce a test specimen with a 1-square centimeter cross-sectional area for a distance of approximately 3 centimeters.

Stress data is calculated using the initial one-square centimeter cross-sectional area. Engineering strain is obtained by dividing the change in gauge length by the original gauge length. Area under the stress-strain curve could be considered total work or energy required to produce failure. These data are presented in Table 2.4.

Table 2.4. Summary of Force-Ductility Data at 39.2°F (4°C) and 5 cm Per Minute for CRM Binders.

Binder Type	Maximum Engr. Stress, psi(kPa)	Maximum Engr. Strain, in/in	Area Under Stress-Strain Curve	Initial Slope of True Stress-Strain Curve
AC-10	33.7(232.5)	9.3	56.6	3.0
AC-10 + 4% Fine CRM	65.2(449.5)	8.1	177.5	2.9
AC-10 + 10% Fine CRM	66.3(457.1)	8.9	313.6	2.4
AC-10 + 18% Fine CRM	72.9(502.6)	7.2	400.0	2.9
AC-10 + 4% Coarse CRM	70.4(485.4)	5.4	168.5	3.0
AC-10 + 10% Coarse CRM	71.4(492.3)	4.7	211.9	2.8
AC-10 + 18% Coarse CRM	73.4(506.1)	4.2	255.5	2.4

Three parameters from the force ductility test are plotted in Figures 2.8, 2.9, and 2.10: maximum stress, maximum strain, and area under the stress-strain curve, respectively. The maximum stress required to cause failure in the binder (Figure 2.8) is slightly higher for the binders produced with fine CRM. The maximum strain at failure is also

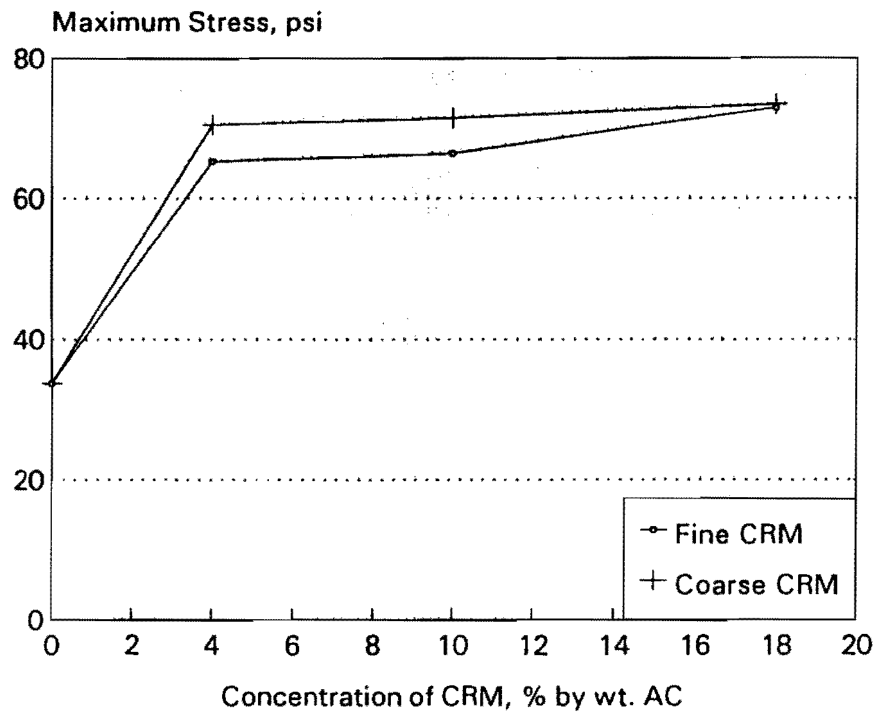


Figure 2.8. **Maximum Stress in the Force Ductility Test for CRM Binders.**

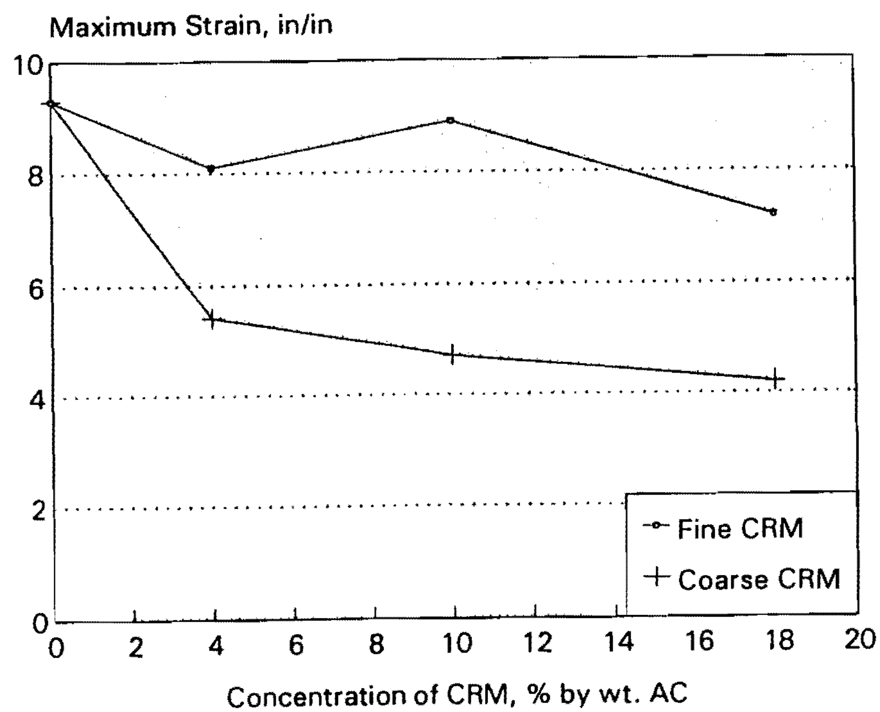


Figure 2.9. **Maximum Strain at Failure in the Force Ductility Test for CRM Binders.**

higher for the fine CRM binders (Figure 2.9), which concurs with the previously described ductility-test data.

The area under the stress-strain curve shown in Figure 2.10 indicates that increasing quantities of rubber causes an increase in the total energy required to fail the sample. There is no difference between the fine and coarse CRM binders at a concentration of four percent CRM; however, the fine CRM binder has a significantly greater energy required to cause failure than the coarse CRM binder at concentration levels above four percent CRM.

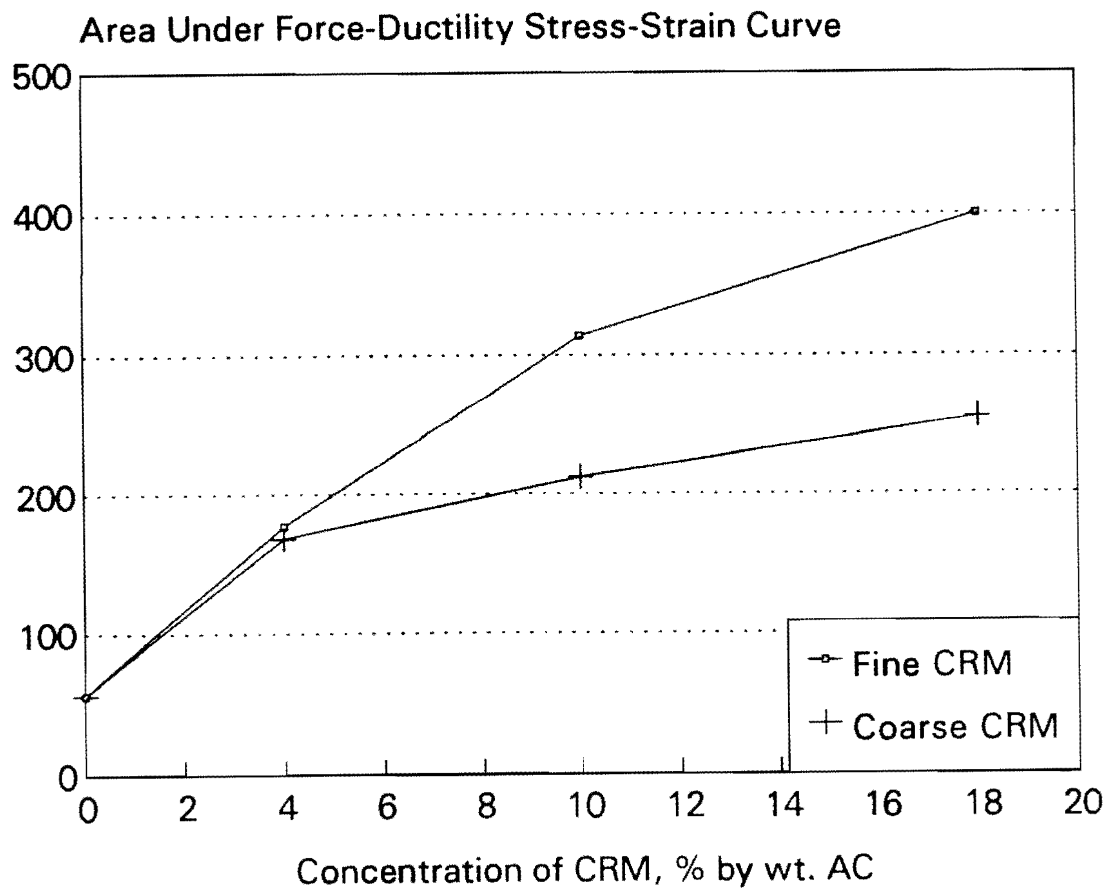


Figure 2.10. Area Under the Stress-Strain Curve in the Force-Ductility Test for CRM Binders (Total Energy Required to Cause Sample Failure).

2.5 Brookfield Viscosity

Viscosity of crumb rubber modified binders is typically performed using a Brookfield rotational viscometer. Standard absolute and kinematic viscosity tests for asphalt cements which employ a capillary viscometer are not appropriate for crumb rubber modified binders since rubber particles interfere with laminar flow through the capillary. Although the Brookfield viscosity test is routinely performed for crumb rubber modified materials, data can be inconsistent without a strict adherence to a proper test protocol. This is illustrated in Figure 2.11 where a viscosity-temperature curve was developed for a particular crumb rubber modified binder. In this figure, 2

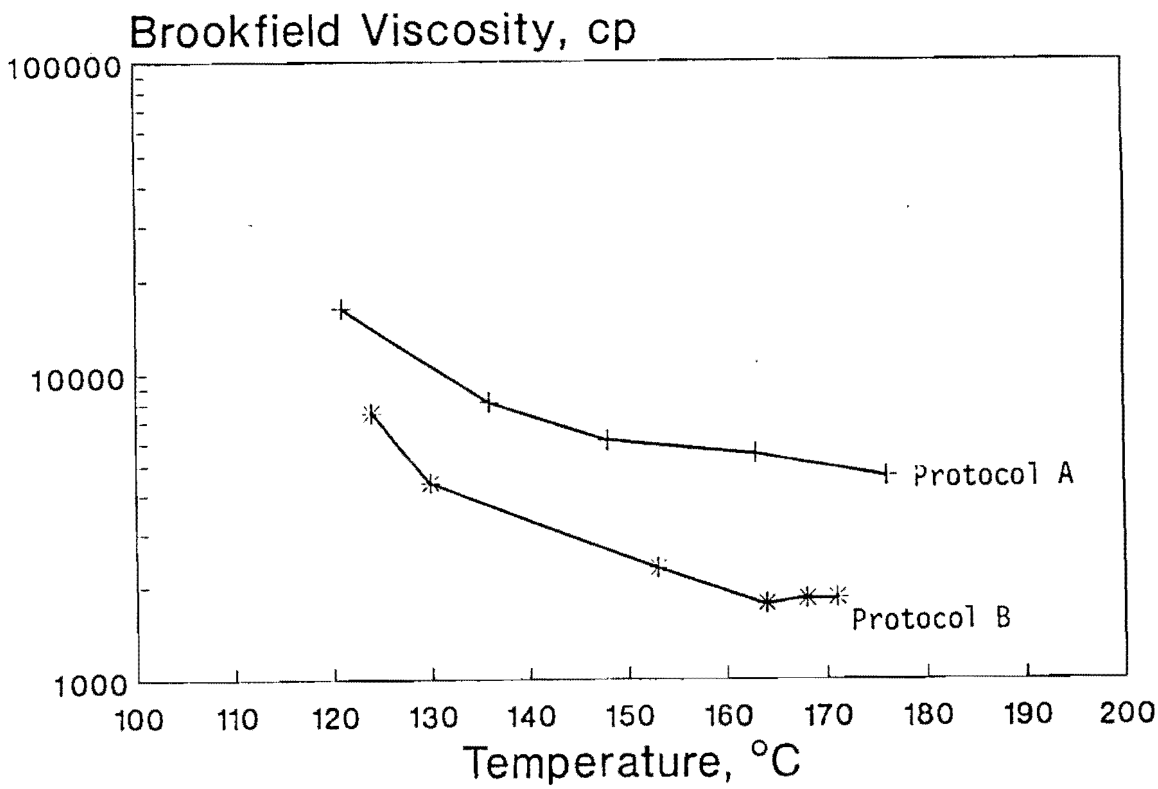


Figure 2.11. Brookfield Viscosity for a CRM Binder (-#80 Mesh CRM at 18 Percent) Tested According to Two Different Protocols.

significantly different curves were obtained for the same binder. The difference between the 2 curves is caused by slight changes in the test protocol for the development of each curve. For example, viscosity measurements were made for 1 curve as the temperature of the binder was increasing, and for the other curve measurements were made as the temperature of binder was decreasing. In any case, it appears that temperature control is a key factor in obtaining consistent Brookfield viscosity results. It is also very important to agitate the binder prior to each measurement, to minimize discrepancies caused from the CRM settling to the bottom of the test beaker.

A Brookfield viscosity test protocol (at 175°C) was developed in this study for TxDOT which produces little variability. Results obtained using this protocol along with its variability are shown in Figure 2.12.

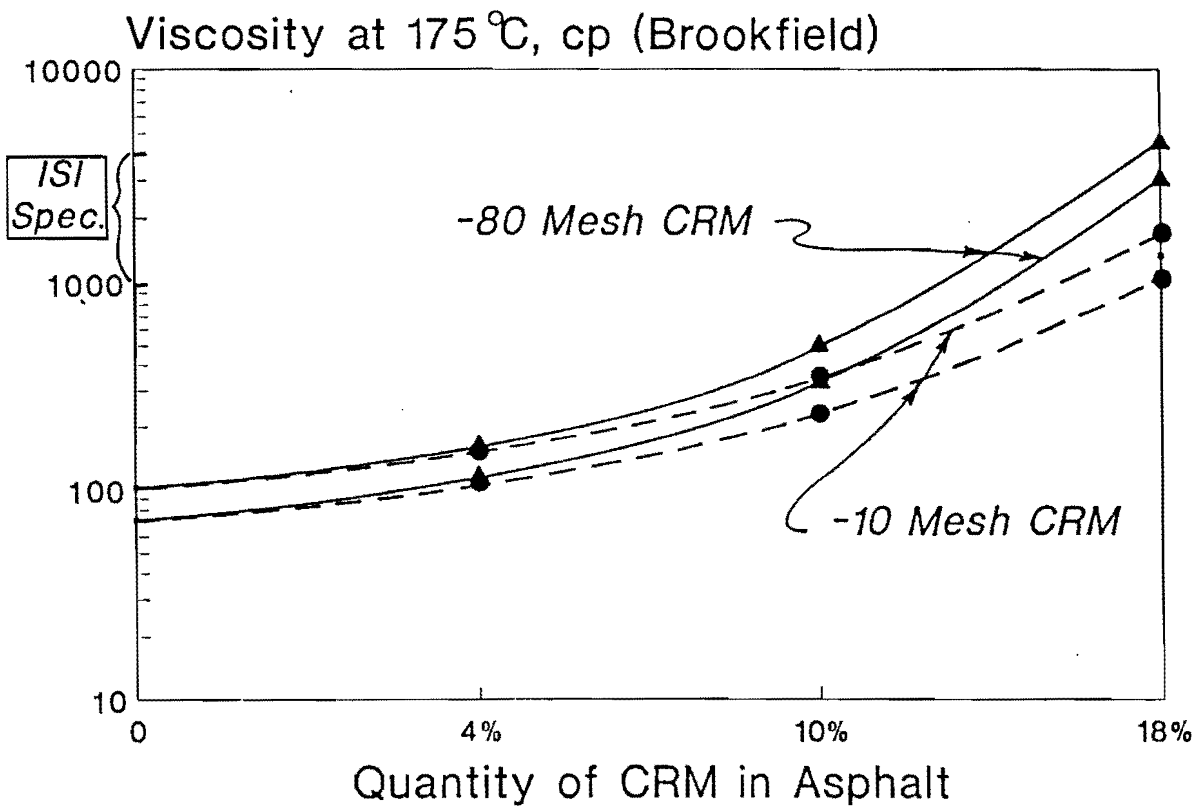


Figure 2.12. **Brookfield Viscosity for Asphalt and CRM Binders of Different Particle Sizes at 175°C Measured According to TTI Protocol.**

2.6. Elastic Recovery

The elastic recovery test was developed for use with polymer-modified binders but is not a widely used or widely accepted test for bituminous binders. The test is performed using the standard ductility test equipment and specimen molds. The test is performed at 50°F (10°C) and at a rate of pull of 5 cm/min. The specimen is elongated to 20 centimeters and maintained here for 5 minutes. The specimen is then cut in half and allowed to recover for one hour. At the end of 1 hour the ductilometer is retracted until the two broken ends of the specimen touch. The total distance the specimen recovers is reported as the elastic recovery.

All of the CRM binders tested (4, 10, and 18% coarse CRM and 4, 10, and 18% fine CRM binders) broke prior to reaching 20 centimeters. Therefore, elastic recovery could not be measured for these CRM binders according to this procedure.

2.7. Resiliency

A resiliency test was performed on control and CRM Binders according to ASTM D1754. This test was included in the testing program because it is incorporated in the guide specifications for asphalt-rubber binders set forth by International Surfacing, Inc. The resiliency test is primarily used for bituminous hot-poured types of joint sealants for portland cement concrete and asphaltic concrete pavements.

The test equipment is the same as for the penetration test except the penetration needle is replaced with a ball penetration tool. The ball penetration tool is allowed to penetrate the bituminous specimen for five seconds and the reading is recorded as P. Without returning the dial pointer to zero, the ball is pressed down an additional 100 (P + 100) at a uniform rate in 10 seconds. The tool is held here for an additional five seconds during which time the dial is re-zeroed. The clutch is then released, and the specimen is allowed to recover for 20 seconds, and the final dial reading is recorded as F. The recovery is computed as follows:

$$\text{Recovery, \%} = P + 100 - F.$$

The average of three determinations is called the resilience. These data are shown below in Table 2.5.

Table 2.5 Resiliency Test Data for Control and CRM Binders at 77°F (25°C).

Binder	Resiliency, (% Recovery)
Texaco AC-10	0
AC-10 + 4% Fine CRM	0
AC-10 + 10% Fine CRM	0
AC-10 + 18% Fine CRM	0
AC-10 + 4% Coarse CRM	1
AC-10 + 10% Coarse CRM	5
AC-10 + 18% Coarse CRM	11

The resiliency for the control binder and for all of the CRM binders produced with the fine crumb rubber was zero. The coarse CRM binders, however, did exhibit some resiliency as shown in Figure 2.13. The resiliency increased with increasing concentrations of rubber. Only the binder with 18% CRM met the ISI specification as shown in this figure. As mentioned previously, this test is typically used for joint sealants and the relevancy for this test for CRM binders used in paving applications is uncertain.

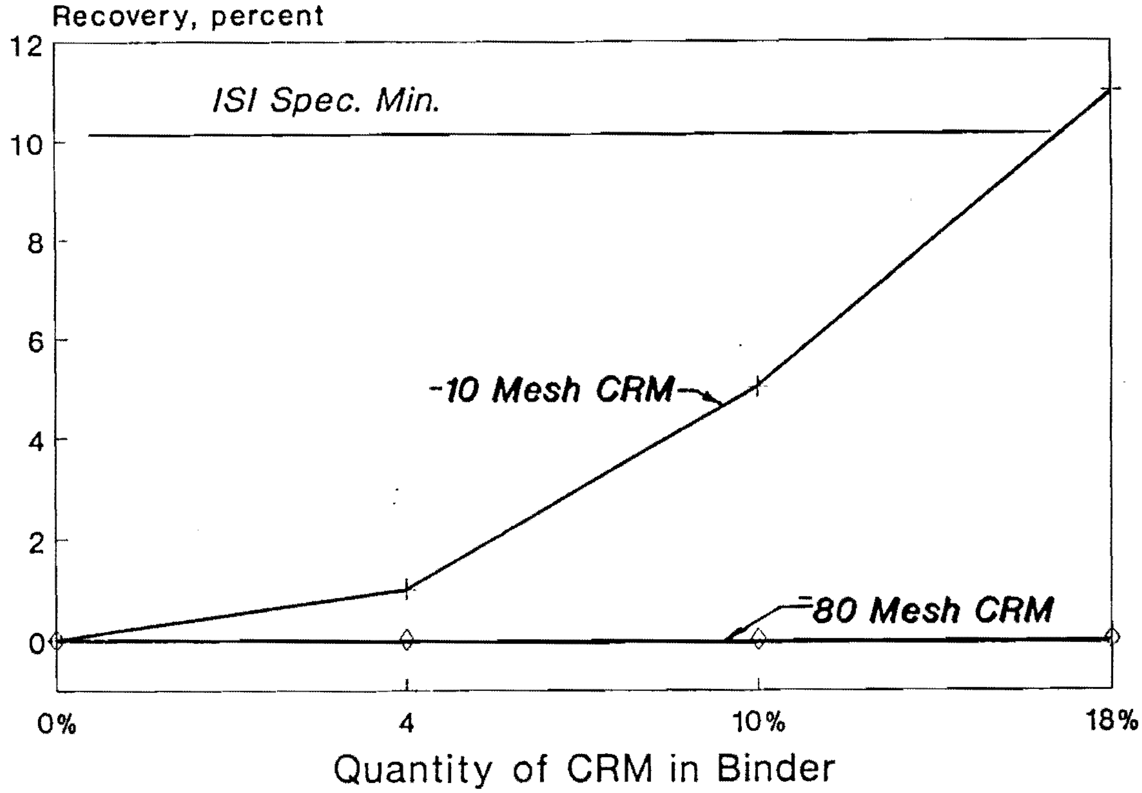


Figure 2.13. Resiliency for Control and CRM Binders.

2.8. SHRP Binder Specification and Binder Tests

The SHRP asphalt binder specification (Kennedy and Moulthrop 1993), as shown in Table 2.6, is based on characterization of the linear-viscoelastic properties as influenced by time of loading and temperature. Kennedy and Moulthrop (1993) state that the specification relies primarily on asphalt binder stiffness of both unaged and aged material measured at a specific combination of load duration and temperature. Selected temperatures are related to the environment in which the asphalt binder must serve. Hence, asphalt binder grades are specified for design pavement temperature.

Environmental conditions are specified as the highest seven-day average of daily maximum pavement temperature and the lowest pavement temperature in a year. Asphalt binders are categorized with the designation PG x-y where

PG	is the Performance Grade,
x	is the design high temperature range, and
y	is the design low temperature range.

Table 2.6. Strategic Highway Research Program (SHRP) Binder Specification.

Performance Grade	PG 52							PG 58					PG 64					PG 70			
	-10	-16	-22	-28	-34	-40	-46	-16	-22	-28	-34	-40	-16	-22	-28	-34	-40	-10	-16	-22	-28
Average 7-day Maximum Pavement Design Temperature, °C [a]	<52							<58					<64					<70			
Minimum Pavement Design Temperature, °C [a]	>-10	>-16	>-22	>-28	>-34	>-40	>-46	>-16	>-22	>-28	>-34	>-40	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28
Original Binder																					
Flash Point Temp, AASHTO T48: Min., °C	230																				
Viscosity, ASTM D 4402: [b] Max, 3 Pa·s (3000 cP) Test Temp, °C	135																				
Dynamic Shear, SHRP B-003: [c] G*/sin δ, Min, 1.0 kPa Test Temp @ 10 rad/s, °C	52							58					64					70			
Physical Hardening Index,[d] h	Report																				
Rolling Thin Film Oven Residue (AASHTO T 240, ASTM D 2872)																					
Mass Loss, Max, percent	1.00																				
Dynamic Shear, SHRP B-003: G*/sin δ, Min, 2.2 kPa Test Temp @ 10 rad/s, °C	52							58					64					70			
Pressure Aging Vessel Residue (SHRP B-005)																					
PAV Aging Temperature, °C	90							100					100					100/(110)[e]			
Dynamic Shear, SHRP B-003: G* sin δ, Max, 5,000 kPa Test Temp @ 10 rad/s, °C	25	22	19	16	13	10	7	25	22	19	16	13	28	25	22	19	16	34	31	28	25
Creep Stiffness, SHRP B-002: [f] S, Max, 300,000 kPa, m - value, Min, 0.30 Test Temp @ 60s, °C	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	-6	-12	-18	-24	-30	0	-6	-12	-18
Direct Tension, SHRP B-006: [f] Failure Strain, Min, 1.0% Test Temp @ 1.0 mm/min, °C	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	-6	-12	-18	-24	-30	0	-6	-12	-18

Notes:

a Pavement temperatures are estimated from air temperatures using an algorithm contained in the SUPERPAVE software program or may be provided by the specifying agency.

b This requirement may be waived at the discretion of the specifying agency if the supplier warrants that the asphalt binder can be adequately pumped and mixed at temperatures that meet all applicable safety standards.

c For quality control of unmodified asphalt cement production, measurement of the viscosity of the original asphalt cement may be substituted for dynamic shear measurements of G*/sin δ at test temperatures where the asphalt is a Newtonian fluid (generally above 55°C). Any suitable standard means of viscosity measurement may be used, including capillary or rotational viscometry.

d The physical hardening index h accounts for physical hardening and is calculated by $h = (S_{24}/S_1)^{m_1/m_{24}}$ where 1 and 24 indicate 1 and 24 hours of conditioning of the tank asphalt. Conditioning and testing is conducted at the designated test temperature. Values should be calculated and reported. S is the creep stiffness after 60 seconds loading time and m is the slope of the log creep stiffness versus log time curve after 60 seconds loading time.

e The PAV aging temperature is 100°C, except in desert climates, where it is 110°C.

f If the creep stiffness is below 300,000 kPa, the direct tension test is not required. If the creep stiffnesses is between 300,000 and 600,000 kPa the direct tension failure strain requirement can be used in lieu of the creep stiffness requirement. The m-value requirement must be satisfied in both cases.

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Hence, an asphalt binder PG 58-34 would meet the specification for a design high pavement temperature up to 58°C (136.4°F) and a design low temperature warmer than -34°C (-29.2°F) (Huber 1993). Required asphalt binder properties are designed to maximize beneficial effects of the binder with respect to permanent deformation, fatigue cracking, and low temperature cracking within the specified environmental conditions (Kennedy and Moulthrop 1993).

The following rheological properties are measured as part of the specification (Anderson and Kennedy 1993):

- *Tenderness* - tank material with a minimum value of $G^*/\sin\delta$ measured at the maximum pavement temperature.
- *Rutting* - RTFOT residue with a minimum value of $G^*/\sin\delta$ measured at the maximum pavement temperature.
- *Thermal cracking* - Pressure Aging Vessel (PAV) residue with a maximum value of stiffness and a minimum value for the slope of the log stiffness versus log time curve measured at the minimum pavement design temperature plus 18°F (10°C). In addition, a lower limit on the strain to failure at the minimum pavement design temperature is specified.
- *Fatigue* - PAV residue with a maximum value of $G^*\sin\delta$ measured at the intermediate pavement temperature.

The G^* and $\sin\delta$ parameters used in evaluating tenderness, rutting and fatigue are measured using a dynamic shear rheometer. In this test, a thin film of asphalt binder (0.04 in to 0.08) (1 to 2 mm) is placed between two parallel plates and tested in torsion.

The plates are oscillated, and the maximum torque, angular deflection, and phase angle are recorded. When the plate deflection is greatest at the maximum applied torque, the material is said to be perfectly elastic, and the phase angle is zero. When the applied torque is zero at the maximum deflection point, the material is a perfect viscous fluid, and the phase angle is 90°.

King (1993) describes the specifications as follows: "To control rutting, the complex modulus (G^*) and the phase angle (δ) are measured at the maximum pavement service temperature. In the SHRP binder specification, the modulus term ($G^*/\sin\delta$) must be greater than 2.2 kPa. This value represents the materials resistance to pure viscous flow, the most relevant binder property relating to permanent deformation. To control fatigue, the complex modulus and the phase angle are measured at the average pavement service temperature. In the SHRP binder specification, $G^*\sin\delta$ must be less than 5.0 MPa."

Efforts to measure properties of asphalt binders modified with crumb rubber using the dynamic oscillatory shear rheometer are unsuccessful thusfar in this study. As described previously, a very thin sample of binder (0.04 to 0.08 in.) (1 to 2 mm) is used in the test procedure. The crumb rubber itself often exceeds this dimension causing erroneous test results. While the spacing between the plates may be increased, efforts to perform this test thus far are producing inconsistent results.

Thermal cracking is evaluated on the binder residue from the pressure aging vessel (PAV). The PAV simulates what happens to the binder during long-term aging. SHRP specifies a maximum stiffness at low temperature after accelerated aging to resist thermal cracking. Stiffness is evaluated using the bending beam rheometer. For this study, all tests were performed at 5°F (-15°C) using the bending beam rheometer. In this test, a beam of asphalt binder is loaded in the center.

The deflection of the beam is measured for 240 seconds. A computer collects the data, calculates the stiffness (S) at various loading times, and calculates the slope (m) of the log stiffness versus log time curve. The stiffness after 60 seconds loading time, and the slope are the reported data.

At low temperatures, asphalt binder becomes very stiff and strain intolerant and researchers have related cracking resulting from a single low-temperature excursion to the stiffness of the asphalt cement at the cracking temperature (Anderson and Kennedy 1993). The temperature at which cracking occurs in this mechanism is referred to as the limiting stiffness temperature. The limiting stiffness temperature is simply the pavement temperature at which a certain stiffness value is reached after a specified loading time; at temperatures below this, the pavement will experience thermal cracking. The low test temperature in the specification is at the critical temperature, in this case, minimum pavement temperature plus 18°F (10°C). Test results from the bending beam rheometer are shown in Table 2.7.

According to the SHRP specification, the bending beam test is to be performed on the PAV residue; however, for research purposes, this test was performed on both the original binders and the PAV residue. Each test value shown in Table 2.7 for original binders represents the average of five tests and after PAV represents an average of 2 tests. SHRP requires that the stiffness after PAV be less than 43,511 psi (300 MPa) and the slope of the log stiffness-log time curve be greater than 0.30.

The bending beam stiffness data are plotted in Figure 2.14. The dashed lines in Figure 2.14 represent binders produced with the fine CRM (-80 mesh) and the solid lines represent binders produced with the coarse CRM (-10 mesh). Intuitively, one might expect the CRM binders to be stiffer than neat asphalt cement; however, in the bending beam test, this is not the case. Generally, as rubber content increases, the stiffness decreases. Recall that this test is performed at 5°F (-15°C) and the temperature susceptibility of crumb rubber is much less than that of asphalt cement.

Table 2.7. **Properties of Control and CRM Binders as Measured with the Bending Beam Rheometer Before and After PAV Aging (All Tests Performed at 5°F (-15°C).**

Binder	Creep Stiffness, MPa		Slope of Log Stiffness-Log Time Curve, m	
	Original	After PAV	Original	After PAV
Texaco AC-10	87.4	137.5	0.49	0.37
AC-10 + 4% Fine CRM	83.5	118.0	0.47	0.38
AC-10 + 10% Fine CRM	75.4	101.0	0.45	0.35
AC-10 + 18% Fine CRM	59.8	79.0	0.41	0.34
AC-10 + 4% Coarse CRM	90.4	154.0	0.45	0.34
AC-10 + 10% Coarse CRM	80.2	121.0	0.43	0.35
AC-10 + 18% Coarse CRM	76.0	81.0	0.40	0.35

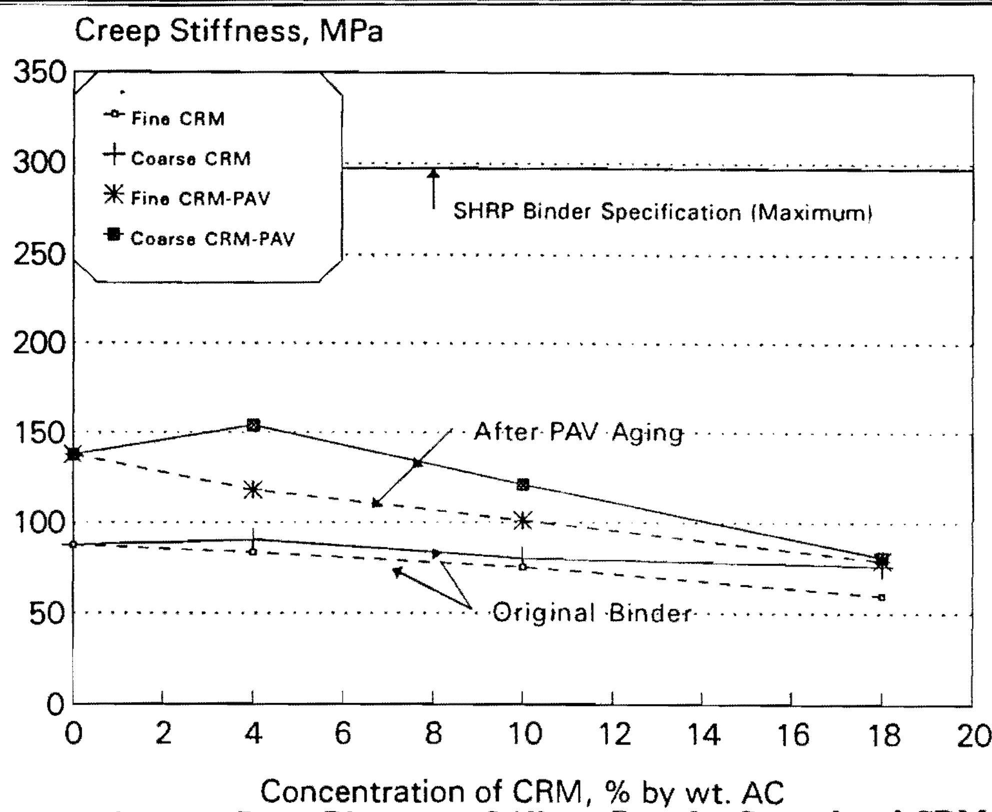


Figure 2.14. **Bending Beam Rheometer Stiffness Data for Control and CRM Binders Before and After PAV Aging (Tests Performed at 5°F [-15°C]).**

Therefore, the higher the concentration of CRM, the lower the temperature susceptibility of the binder at low temperatures.

One might also expect that the finer gradation of crumb rubber would produce a stiffer binder than the coarser CRM. Again, the reverse is true in this test procedure. This may be attributed to the fact that the asphalt film thickness is greater for the coarse CRM than for the fine CRM thereby producing a greater stiffness.

One may assume, from this data, that if the neat asphalt cement meets the SHRP specification, then a binder made with crumb rubber added to that asphalt cement will also meet the specification. Note that the Texaco AC-10 (shown as 0% CRM in Figure 2.14) and, of course, all of the CRM binders have stiffness values well below the specification maximum of 43,511 psi (300 MPa).

If the creep stiffness is below 43,511 psi (300 MPa), the direct tension test is not required on the PAV-aged binder; however, this test was performed on the PAV-aged control and CRM binders for research purposes. The direct tension test measures how much the binder can be stretched before it breaks. A minimum 1% elongation (failure strain) in the direct tension test at the given temperature will qualify a binder for a grade it may not pass with the bending beam.

Direct tension tests were performed on the control and CRM binders after aging in the PAV and these results are shown in Figure 2.15. Each data point represents an average of four tests. According to SHRP specifications, because all of these binders passed the bending beam test, direct tension testing was not required. Surprisingly, however, all of the binders except the 18% fine CRM binder failed the direct tension specification requirement.

Note that at concentrations of fine rubber above 10%, something happens to markedly enhance failure strain (and thus ostensibly increase resistance to pavement cracking).

We believe that, at a certain concentration of rubber particles in the wet process, a three-dimensional network of rubber 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 three dimensional network.

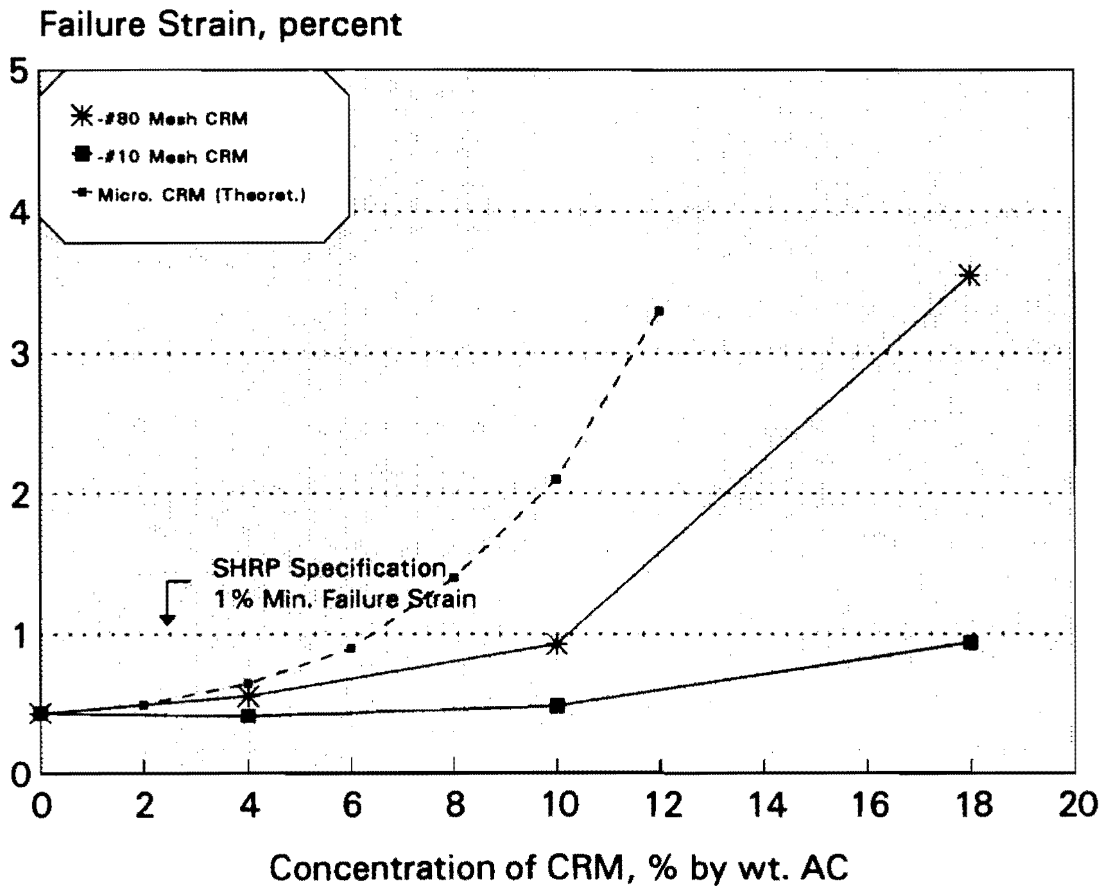


Figure 2.15. Direct Tension Test Results for Control and CRM Binders After PAV Aging (Tests Performed at 5°F [-15°C]).

Shell researchers (Collins and Mikols 1985) reported that, at approximately 5 to 6% neat styrene-butadiene styrene (SBS) rubber in asphalt, a three-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 Figure 2.15, 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 three-dimensional network of tire rubber would be formed at a concentration near 6%.

3

Mixture Design

Two methods were used for designing crumb rubber mixtures for further laboratory characterization: TxDOT's standard method of mixture design (Bulletin C-14) and TxDOT's method for designing crumb rubber mixtures (Tex-232-F).

Current thinking by most asphalt technologists is that asphalt mixtures containing CRM should be gap-graded. The formulation used by TxDOT for crumb rubber mixtures is termed coarse-matrix, high-binder (CMHB). Many of the premature failures which have occurred with crumb rubber mixtures have been associated with dense-graded aggregate systems. It is believed that, in a dense-graded mixture, the CRM interferes with the development of interparticle friction which is needed to transfer the surface loading to the underlying structure. On the other hand, the CMHB mixture is designed such that the CRM fills the available voids thereby maintaining stone-to-stone contact.

One of the simplest and most economical methods of incorporating CRM into asphalt mixtures is by using a generic dry process: adding the CRM to the mixture as part of the aggregate rather than preblending it with the asphalt cement. This process was evaluated along with dense-graded aggregates. It is believed that one of the main problems with using CRM in dense-graded mixtures is that the concentration of CRM has typically been too high for this type of gradation (18% CRM or more, by weight of the binder). Therefore, standard mixture design procedures (C-14) were used to determine how much CRM could be added to a Type D mixture (dry) still maintaining all requirements associated with standard mixture design (acceptable air voids and Hveem stability values).

Dense-graded mixtures were designed using two different gradations of CRM (-#10 mesh and -#80 mesh) and varying the concentration of CRM. Three concentrations of CRM were evaluated: 0.2, 0.5 and 0.8% CRM by weight of the aggregate. Of these 3 concentrations, it was determined that 0.5 percent was the optimum concentration of CRM for these dense-graded mixtures, as will be discussed further. Expressing the CRM concentration in terms of asphalt content, it is about 10% by weight of the asphalt for the -#80 CRM and 7% for the -#10 CRM. These mixtures were then characterized using AAMAS (Von Quintus et al. 1991).

Tex-232-F is a volumetric design procedure for designing coarse-matrix, high-binder (CHMB) crumb rubber mixtures. This procedure was used to design mixtures that corresponded with the binders produced in the binder study as well as two other generic dry mixtures. A discussion of these mixture designs is contained in this chapter.

3.1 Selection of Materials

The aggregates used for the mixture designs included a crushed limestone and field sand. Care was taken to eliminate the influence of aggregate properties in the evaluation process; therefore, aggregate from the same source was used for all the mixes. The crushed limestone was from Gifford Hill in New Braunfels. Limestone screenings were from Texas Crushed Stone in Georgetown and the field sand was from a local source near Hearne, Texas. Specific gravities of the aggregate are as follows:

Bulk specific gravity of coarse limestone	2.554,
Bulk specific gravity of limestone screenings	2.443, and
Bulk specific gravity of field sand	2.551.

The asphalt used for this study was a Texaco AC-10 from Port Neches. This asphalt was used for the design of all the mixtures, including the control.

The rubber used in this study is from ground, whole tires. Two different sizes of CRM were used in this study: -#10 mesh and -#80 mesh. The source of the rubber passing the #10 sieve size was Granular Products of Mexia, Texas. The source of rubber passing the #80 sieve size was Rouse Rubber of Vicksburg, Mississippi. Rubber Passing #10 sieve size from here on referred to as coarse rubber and rubber passing #80 sieve size as fine rubber.

3.2 Mixture Design for Dense-Graded Mixtures

All dense-graded mixtures were designed according to standard TxDOT mixture design procedures (Bulletin C-14). A conventional Type-D mix is the control mix for this study and is a blend of 90% crushed lime stone and 10% field sand. The final gradation is shown in Table A1 of Appendix A. The mix design is based on weight calculations. Asphalt contents were added by weight of the aggregate instead of by the weight of the mix. A step by step mixture design procedure is discussed briefly.

Control Mix (Type D)

(a) A cumulative weight of 4000g was weighed according to the gradation established. The aggregate was kept in a forced-draft oven at 320°F for 5 hours. Asphalt AC-10 was kept in the oven for 1 hour till it reached the appropriate viscosity or temperature. The mix was prepared in accordance with Test Method Tex-205-F. Samples were mixed with varying percentages of asphalt contents: 4.0, 4.5, 5.0, 5.5, 6.0, and 6.5% by weight of the aggregate.

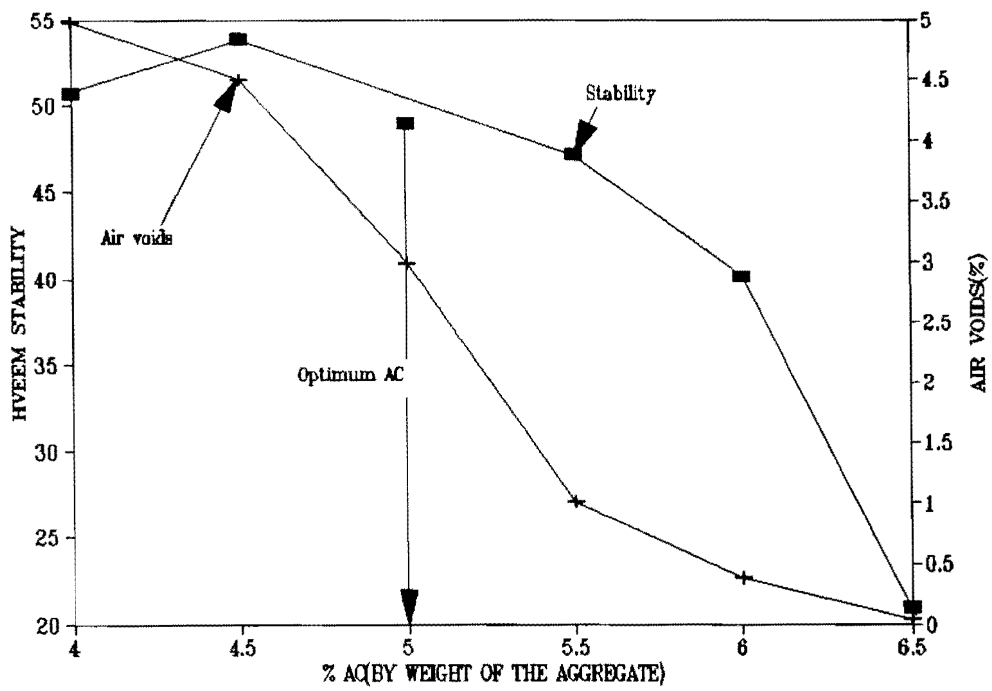
(b) Three samples were molded for each of the asphalt content. The mixture was compacted using the Texas Gyratory Compactor. The samples were molded according to Test Method Tex-206-F.

(c) The bulk specific gravity of the samples was determined using Test Method Tex-207F and theoretical maximum specific gravity (Rice) was obtained using Test Method Tex-227-F. All the values are tabulated and documented in Appendix-A Table 2.

(e) Stability tests were performed on the samples prepared above using Hveem stabilometer in accordance with Test Method Tex-208-F. All the values are tabulated in Appendix A, Table A2.

A graph plotting asphalt content versus stability and air voids on y-axis is shown in Figure 3.1. The optimum asphalt content is that asphalt content at which the mix has a 97% density. The criteria for the acceptance of the mix are at 97% density, and the minimum stability should be 35. From Figure 3.1 for the control mix, the optimum asphalt content was chosen to be 5% by weight of the aggregate.

Figure 3.1. Tex-C-14 Mix Design for Type-D Control Mix.



Addition of CRM to Dense-Graded Mixtures

As described earlier, the task here was to determine the optimum rubber content that can be added dry in the dense graded mix with little or no changes to the gradation. In order to achieve that, it seemed logical to replace the sand with the equivalent volume amount of CRM. For the coarse rubber, equivalent volume of field sand retained on #80 sieve was replaced and for the fine rubber, equivalent volume of #200 was replaced. Initially 0.2% CRM, by weight of aggregate, was added to the mix. Conventional mix design procedures were then followed, as previously described, for both fine and coarse CRM mixtures. The results are tabulated in Appendix B and C respectively. There was a very little change in stability and optimum asphalt content between these CRM mixtures and the control mix.

The next step was to increase the rubber content in the mix: 0.8% dry rubber by weight of the mix was then added and the mix design procedure was repeated. Both fine and coarse rubber mixes failed to meet the requirements i.e., minimum stability of 35 at optimum density of 97%. Also, significant "swelling" of the samples was observed at this concentration and this was substantiated by high air voids. Results are tabulated in Appendices B and C. The relationships between stability, density and % AC contents are plotted in Figures 3.2 and 3.3.

The above procedure was repeated with 0.5% rubber by weight of the aggregate. These 2 mixes (with coarse and fine rubber) meet the requirements for stability and density as illustrated in Figures 3.4 and 3.5.

Figure 3.2. **Tex-C-14 Mix Design for Dense Graded Mix With 0.8% Fine Rubber by Weight of the Aggregate.**

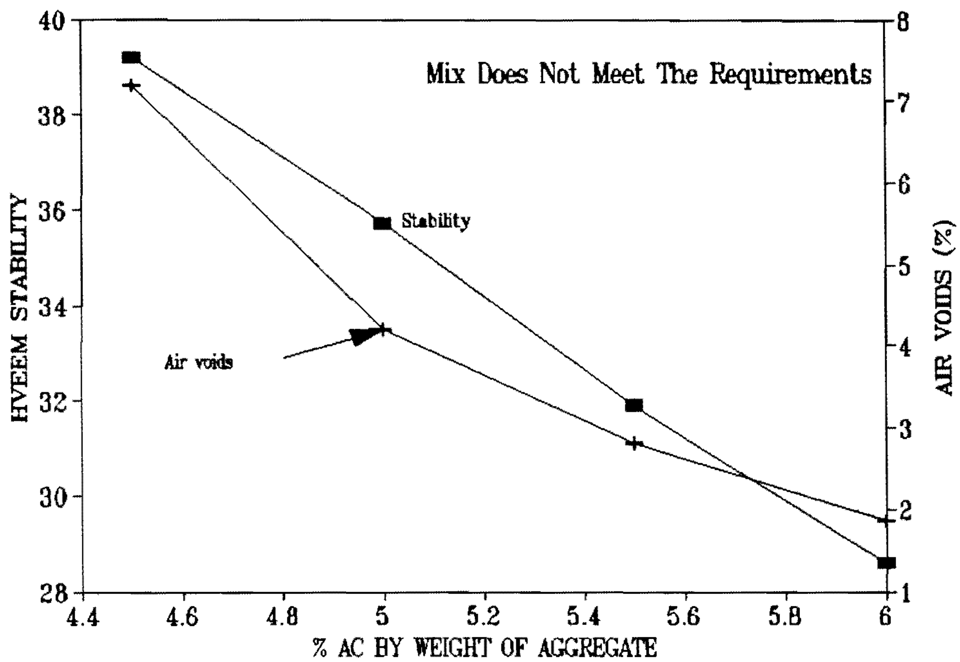


Figure 3.3. **Tex-C-14 Mix Design for Dense Graded Mix With 0.8% Coarse Rubber by Weight of the Aggregate.**

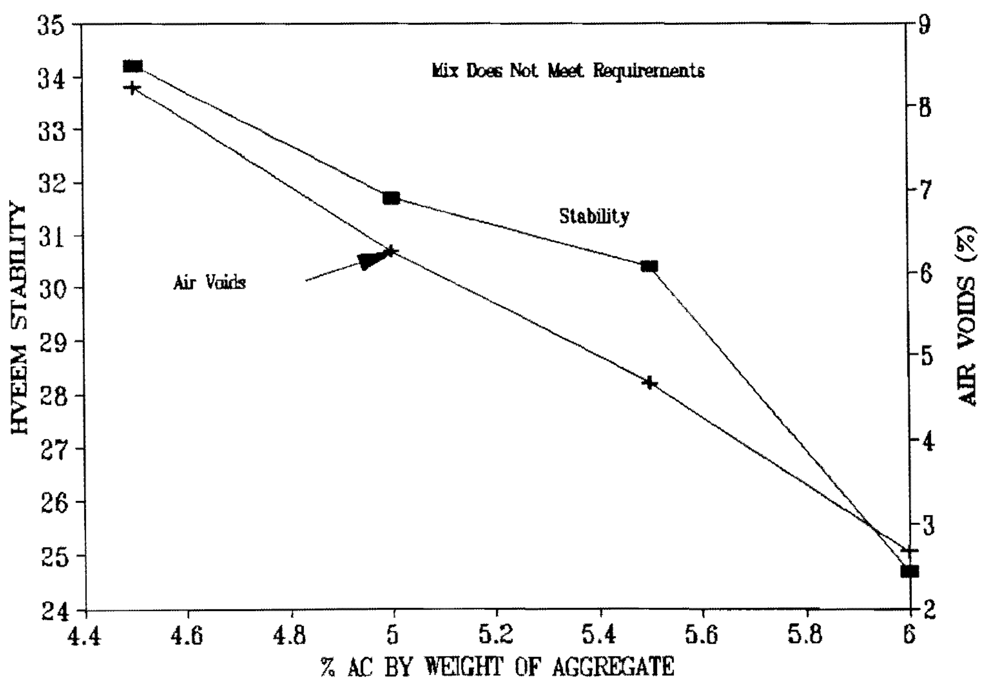


Figure 3.4 Mix Design for Dense Graded Mix With 0.5% Fine Rubber by Weight of the Aggregate.

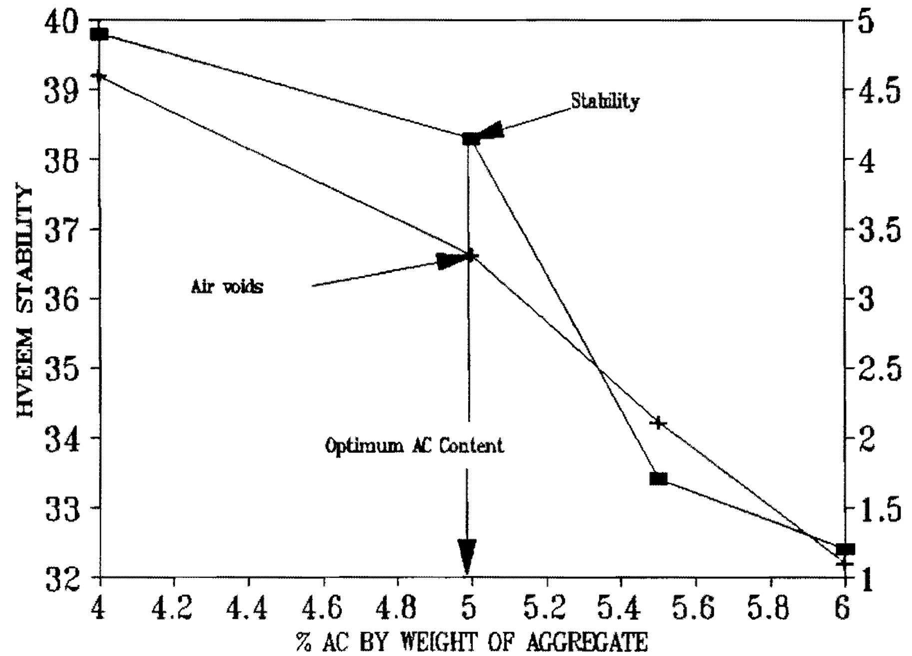
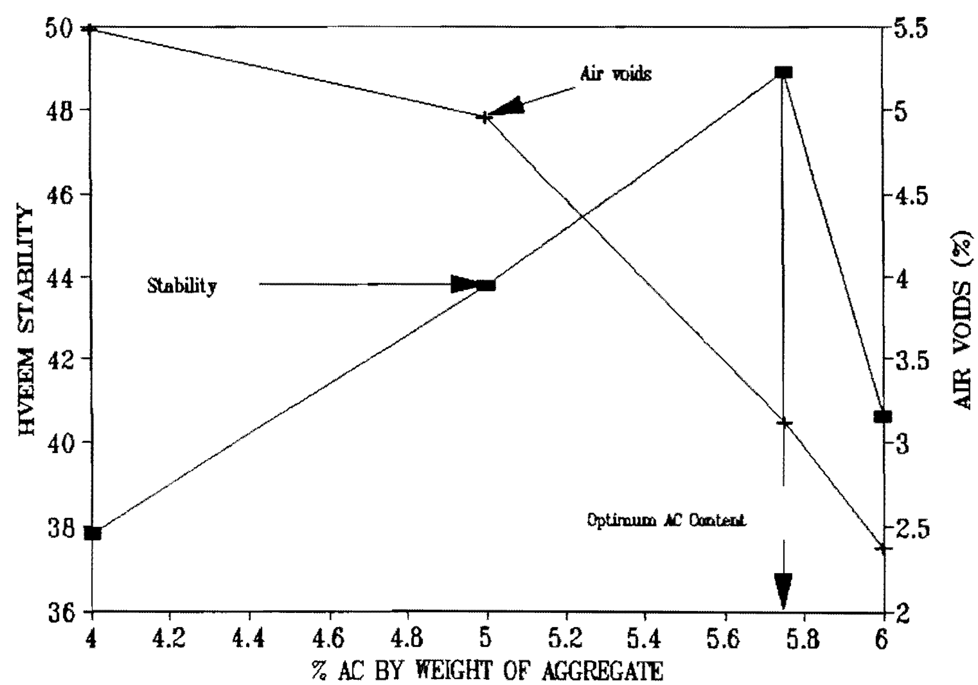


Figure 3.5. Mix Design for Dense Graded Mix With 0.5% Coarse Rubber by Weight of the Aggregate.



Mixtures were also made with 0.6% and 0.7% rubber contents by weight of the aggregate and checked for stability and density. The mix with fine rubber failed to meet the stability and optimum density requirement at 0.7% rubber content. The mix with coarse rubber does not meet both the criteria for stability and density at both 0.6% and 0.7% rubber contents. All the results are tabulated in Appendix B for fine rubber and Appendix C for coarse rubber. A summary of all the stability values and density are tabulated in Table 3.1.

Table 3.1. Type-D Mix With Varying Dry Rubber Contents.

Mix Type (% Rubber By Weight Of Aggregate)	Optimum AC (%)[1]	Stability	Air Voids (%)
Control (0% Rubber)	5.00	49.9	3.1
0.2% Fine Rubber	5.00	50.6	3.4
0.5% Fine Rubber	5.00	38.3	3.3
0.7% Fine Rubber	5.25	32.6	3.0
0.8% Fine Rubber	5.25	34.0	3.4
0.2% Coarse Rubber	5.00	46.6	3.2
0.5% Coarse Rubber	5.75	48.9	3.1
0.6% Coarse Rubber	5.75	25.7	4.2
0.7% Coarse Rubber	5.75	26.3	4.3
0.8% Coarse Rubber	5.75	28.0	3.8

[1] - % By weight of aggregate

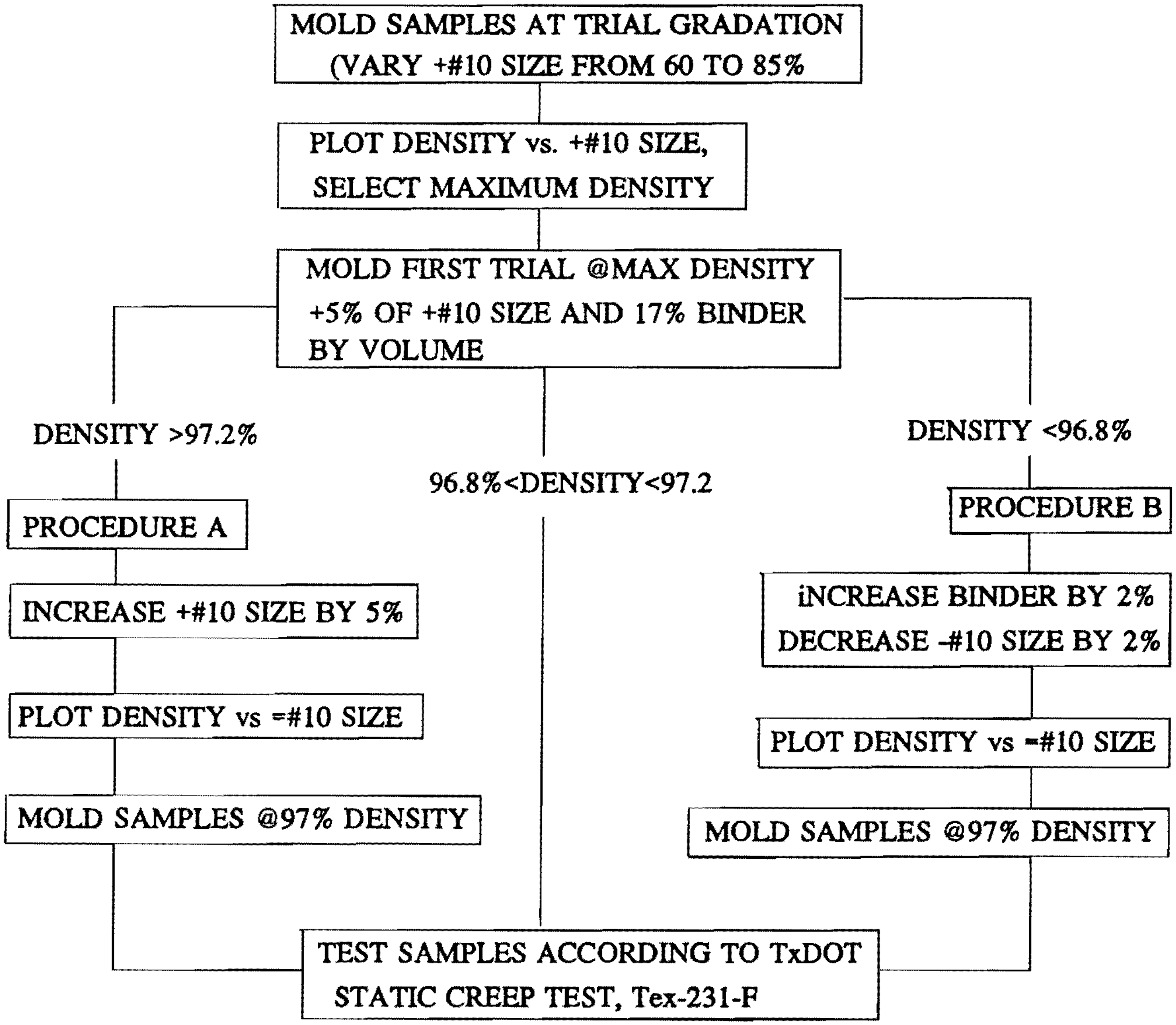
From this table, we may conclude that the mixes containing rubber contents of 0.5% by weight of aggregate were the optimum rubber contents that could be incorporated in this dense-graded mix. Therefore, these two mixtures of fine and coarse CRM at 0.5% CRM (by weight of aggregate) were considered for further evaluation using AAMAS. If these rubber contents were to be expressed in terms of percent AC content, 0.5% fine rubber is equivalent to 10% by weight of the optimum asphalt content and 0.5% coarse rubber is equivalent to 7.3% by weight of the optimum asphalt content. The final gradations for fine and coarse mixes to be evaluated using AAMAS are given in the Appendices B and C, respectively.

3.3 Tex-232-F Mix Design for CMHB CRM Mixtures

Preblending CRM and asphalt prior to incorporation into the hot-mix is known as a "wet" process. The blending process is explained in detailed in the second chapter. If rubber is added directly to the aggregate it is called as dry process. Initially it was decided to design the mixes with rubber contents of 4%, 10% and 18% by weight of the asphalt content; however, draindown with the 4% CRM binder was excessive, and this binder was eliminated from the mixture experiment. Two generic dry mixtures were also designed according to this procedure: 18% fine CRM and 18% coarse CRM (by weight of the asphalt). For comparison purposes, the dry CRM concentrations are expressed herein as a percentage of the asphalt content. These dry mixes are also designed according to Test Method Tex-232-F and evaluated using AAMAS. Mixture design using Tex-232-F is explained briefly in the following paragraphs.

This mixture design procedure may be broadly divided into two steps. The first step is to blend the aggregates and, using varying ratios of coarse to fine aggregates, to obtain the volume of aggregate retained on #10 sieve size that yields the maximum density. The second step is to achieve a 97±0.2% density by varying the volume of +#10 sieve size material and/or varying binder content. A flow chart of the mix design procedure is shown in Figure 3.6.

Figure 3.6 Flow Chart For TxDOT CRM Mix Design, Tex-232-F



The design criteria for this mix design are listed below:

(a) Minimum voids in the mineral aggregate (VMA) = 20%;

(b) Optimum laboratory molded density = 97%;

(c) Minimum volume of the binder = 17%;

(d) Minimum volume of coarse aggregate (retained on #10 sieve) = optimum volume of coarse aggregate + 5% (as determined from the density verses volume of coarse aggregate curve);

(e) Percent aggregate passing #200 sieve = 6%.

STEP 1. To Achieve A Volume Of +#10 Aggregate at Maximum Density

- A sieve analysis was performed and grading factors were determined. Using these grading factors, trial gradations were found for all six fractions of +#10 to -#10 size material. The fractions are 60/40; 65/35; 70/30; 75/25; 80/20 and 85/15 by weight of the aggregate.
- Each gradation consisting of the above fractions was mixed with 5% binder. For the sample preparation, the aggregate and the binder were heated to a temperature of 325±5°F in a forced-draft oven. The binder was stirred well before mixing with the aggregate in order to a have a uniform distribution of the rubber particles. Then the aggregate and the binder were mixed using a mechanical mixer according to Test Method Tex-205-F.
- After blending, the mix was kept in a forced-draft oven at 250±5°F till it reached this temperature. Then 3 samples were molded using the Texas gyratory compactor in accordance with Test Method Tex-206-F. The samples were left in the molds for 2 hours to eliminate any expansion of the sample due to the rebound of the rubber particles.
- Theoretical maximum specific gravity was determined in accordance with Tex-227-F. Also bulk specific gravity and relative density were determined in accordance with Tex-207-F. A graph was plotted between the volume of aggregate retained on #10 sieve and the relative density. From this plot, the volume and the gradation are found for the mix that has maximum relative density. All the values are tabulated and plotted. These tables and graphs are presented in Appendices D through I.

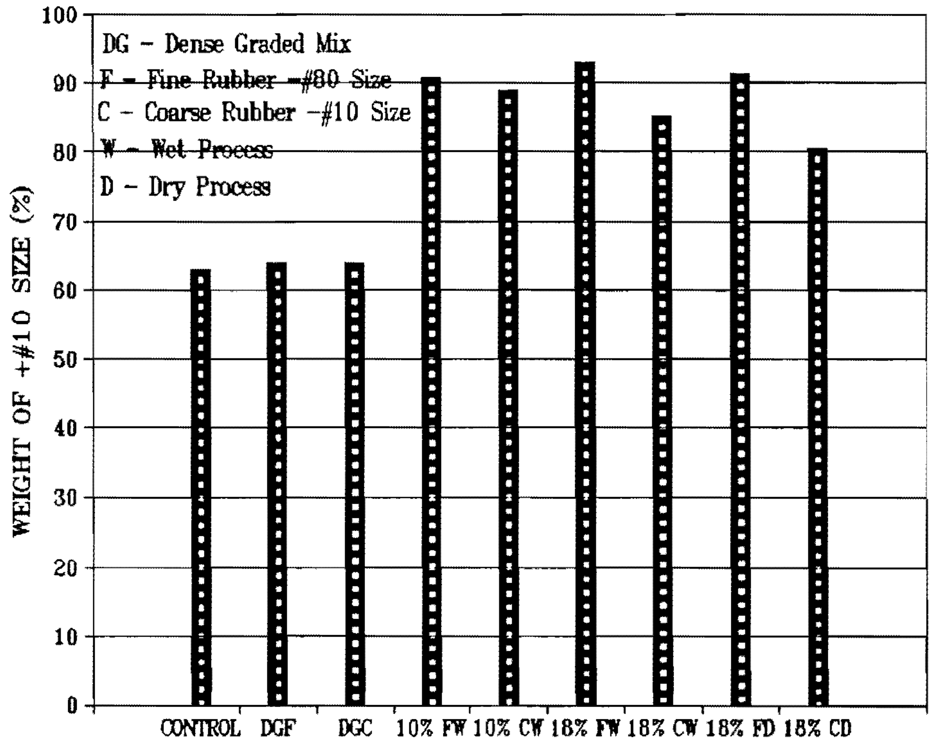
STEP 2. To Achieve A Gradation That Has An Optimum Density Of 97±0.2%.

- To the volume fraction determined in step one, 5% of the volume was added to the +#10 sieve size and reduced correspondingly for the -#10 sieve size. The volume of the binder was increased to 17%, and then samples were molded as previously described.
- If the density is between 97±0.2%, the gradation is acceptable.
- If the density is more than 97±0.2%, 5% (volume) of the +#10 size material is added.
- If the density is less than 97%, 2% binder is added to the mix and the -#10 size material is reduced by 2%.

The above steps should be followed until acceptable density is achieved. All the trials for each mix are presented in the Appendices D through I. The final gradations for all the mixtures are shown here, as well. These mixtures were then evaluated using AAMAS.

For the dry CRM mixtures, step 1 of this procedure was skipped. This was based on an assumption that the final aggregate gradation would not be very different whether the rubber was added dry or wet. However, it was observed that there was some difference in the gradation as shown in Figure 3.7. This figure shows the amount of material retained on #10 sieve (by weight) for each mix. All the samples that satisfied the density requirement of 97±0.2% were tested for static creep in accordance with Test Method Tex-231-F. The test results are tabulated in the appendices.

Figure 3.7. Volume Of +#10 Size Fraction For All The Nine Mixes.



4

Mixture Performance Evaluation

There are four pavement distresses, resulting from load or environmental conditions, which are believed to be the most important with respect to reductions in serviceability and in asphalt pavement performance: fatigue cracking, thermal cracking, permanent deformation, and moisture damage (Von Quintus et al. 1991). Laboratory evaluations were performed in accordance with AAMAS test procedures. Appendix J contains a description of AAMAS as well as a discussion on the preparation of samples. The following is a discussion of individual test results as well as how these results are related to these four types of distresses. Individual test results are discussed herein and all of the data are tabulated in the appendices.

4.1 Rutting

Two types of rutting can occur in asphaltic concrete pavements. These are (1) one-dimensional densification and (2) the lateral movement or plastic flow of asphalt concrete. The more severe premature rutting failures and distortion of asphaltic concrete materials are related to lateral flow and loss of shear strength in the mix, rather than densification (Von Quintus et al. 1991).

One method of evaluating rutting potential was developed by Mahboub and Little (1988) and is recommended for use by AAMAS as a "rough" guideline for mixture evaluation. This method is a graphical solution whereby uniaxial creep data can be compared to criteria for predicting rutting potential. The uniaxial creep test was performed as described earlier on 4-inch (10.2 cm) high by 4-inch (10.2 cm) diameter samples which were molded to air void contents less than 3% to simulate traffic densification. The samples were loaded under static conditions at 60 psi (414 kPa) for one hour with a one hour recovery period. The creep modulus data is shown in Figure 4.1 along with AAMAS criteria.

According to Figure 4.1, the creep moduli of all of the mixtures tested are considered to have low to moderate rutting potential. The dense-graded mixture with fine (dry) CRM seems to be the most rut resistant while the CMHB, 18% coarse CRM (wet) mixture appears to be the least rut resistant.

AAMAS Static Creep Test

In addition to the creep modulus criteria discussed above, there are other parameters measured in this creep test worthy of discussion. Mitchell (1976) presents Figure 4.2 as a schematic representation of the influence of creep stress intensity on creep rate at some selected time after stress application. To provide resistance to thermal cracking, a mixture should have a low tensile creep modulus, but a relatively high tensile strength. The creep modulus of the control mix is significantly higher than all of the crumb rubber modified mixtures (2 to 5 times higher). The indirect tensile strengths at 41°F (5°C) and at a loading rate of 0.05 in/min (0.13 cm/min) for some of the crumb rubber mixtures were about the same as (in one case higher than) the control mixture. Thus, some of the crumb rubber mixtures (18%FW, DGC, and DGF) appear to offer greater resistance to thermal cracking than the control mixture.

At low stresses, creep rates are small and of little practical importance (Little and Youssef 1992). In the midrange of stresses, a nearly linear relationship is found between the log of strain rate and stress. At stresses approaching the strength of the

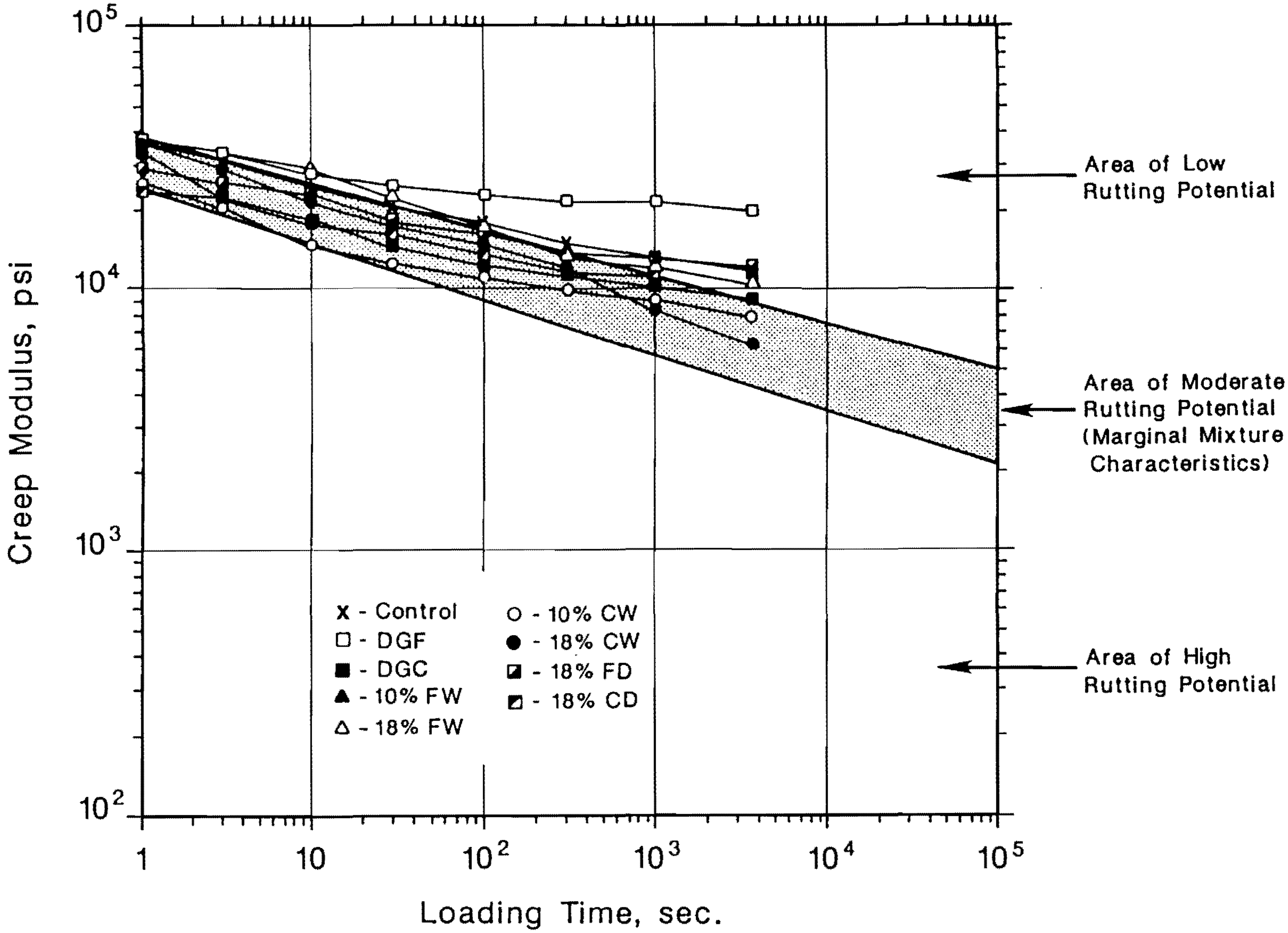


Figure 4.1. Control and CRM Mixtures - AAMAS Chart for Asphaltic Concrete Mixture Rutting Potential for Surface Layers of Asphaltic Concrete Pavements.

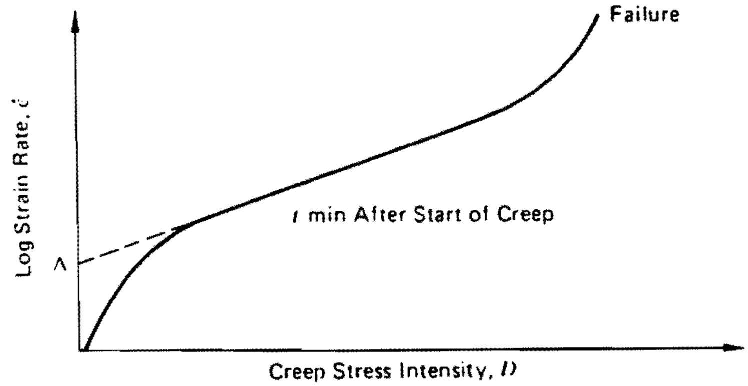


Figure 4.2. Influence of Creep Stress Intensity on Creep Rate (Mitchell 1976).

material, the strain rates become very large and represents the onset of failure. If the stress state in the field (creep stress intensity) is one that pushes the log strain rate into the region near failure (beyond the steady state region), then assumptions of linearity are most certainly not appropriate. This point is very important because in the past linear viscoelastic response of asphalt mixtures under field loading conditions has been assumed. This has largely been because such an assumption is convenient, and creep data from laboratory tests at relatively low stress levels are simply shifted to higher stress states in the field by employing principles of linear viscoelastic superposition. Such an approach is clearly incorrect in the highly non-linear region of Figure 4.2. The importance of selecting a realistic stress state for laboratory testing is then essential.

Another popular generalized form used to illustrate the various stages of creep is illustrated in Figure 4.3. In this figure, creep strain, for a given stress level, is plotted versus time, and the creep strain is divided into three stages. In the first or primary stage the rate of deformation increases rapidly. In the second or "steady state" region, the deformation rate is constant as is the angle of slope (rate of deformation). The third region is the failure stage (tertiary), in which the deformation again increases rapidly.

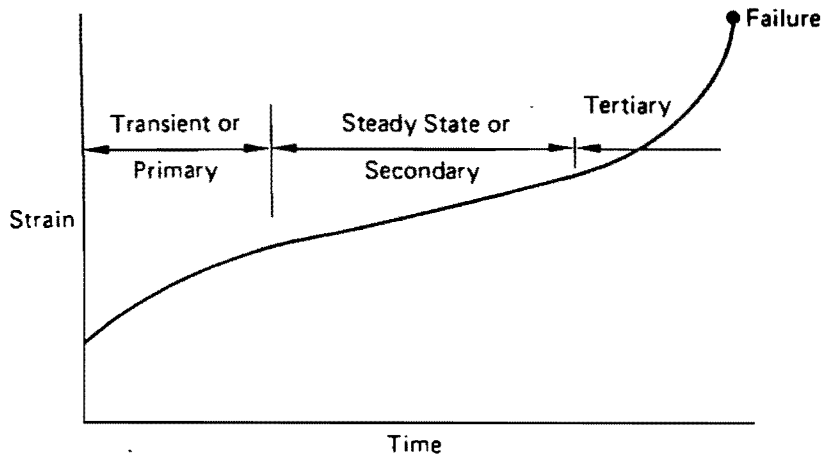


Figure 4.3. Stages of Creep (Mitchell 1976).

The relationship between creep strain and logarithm of time may actually be linear, concave upward, or concave downward. A linear relationship is often assumed for engineering applications because of its simplicity in analysis. However, there is no fundamental "law" of behavior to dictate one form or another.

Use of the uniaxial creep test to define the 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 actual field conditions (Little and

Youssef 1992).

The AAMAS uniaxial creep curves for all the mixtures are presented in Figures 4.4 through 4.12. None of the mixtures appear to reach the tertiary creep region within the one-hour loading period. Uniaxial creep data for all the mixtures are shown below in Table 4.1.

Table 4.1. Uniaxial Static Creep Data (σ_1 = 60 psi [414 kPa]) for Control and Crumb-Rubber Mixtures.

Mixture Type	Log-Log Slope of Steady-State Creep Curve	Strain at End of 3600 Seconds, in/in	Strain Recovery, percent
Control	0.078	0.004153	13
Dense-Graded with Fine CRM (DGF)	0.030	0.003145	21
Dense-Graded with Coarse CRM (DGC)	0.067	0.005227	34
10% Fine CRM - Wet Method (10%FW)	0.111	0.005033	9
10% Coarse CRM - Wet Method (10%CW)	0.098	0.007767	4
18% Fine CRM - Wet Method (18%FW)	0.165	0.005931	20
18% Coarse CRM - Wet Method (18%CW)	0.180	0.009015	3
18% Fine CRM - Dry Method (18%FD)	0.072	0.003383	29
18% Coarse CRM - Dry Method (18%CD)	0.049	0.005010	15

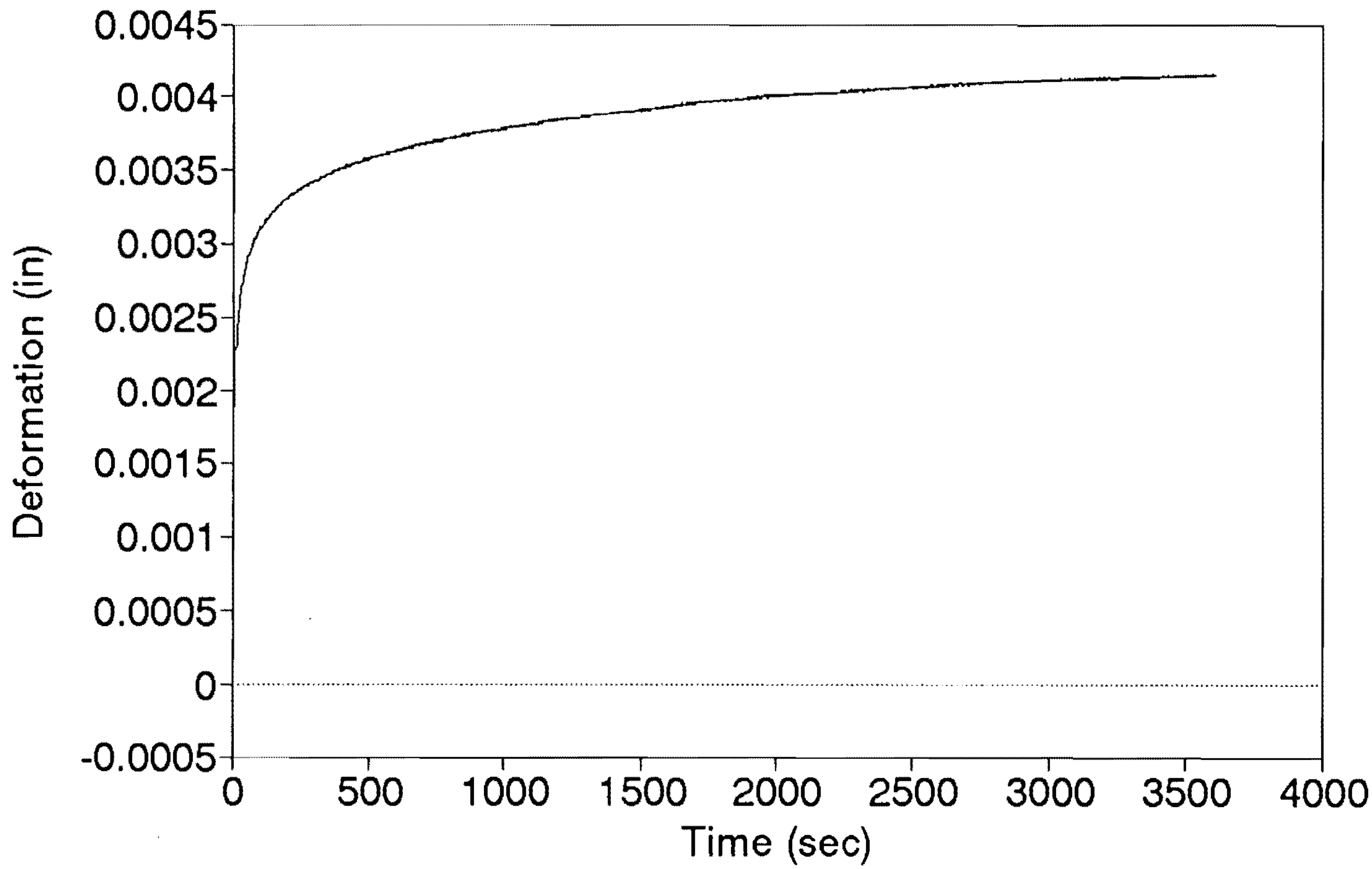


Figure 4.4. Static Creep Deformation Versus Time of Loading for Type D, Control Mix (no rubber).

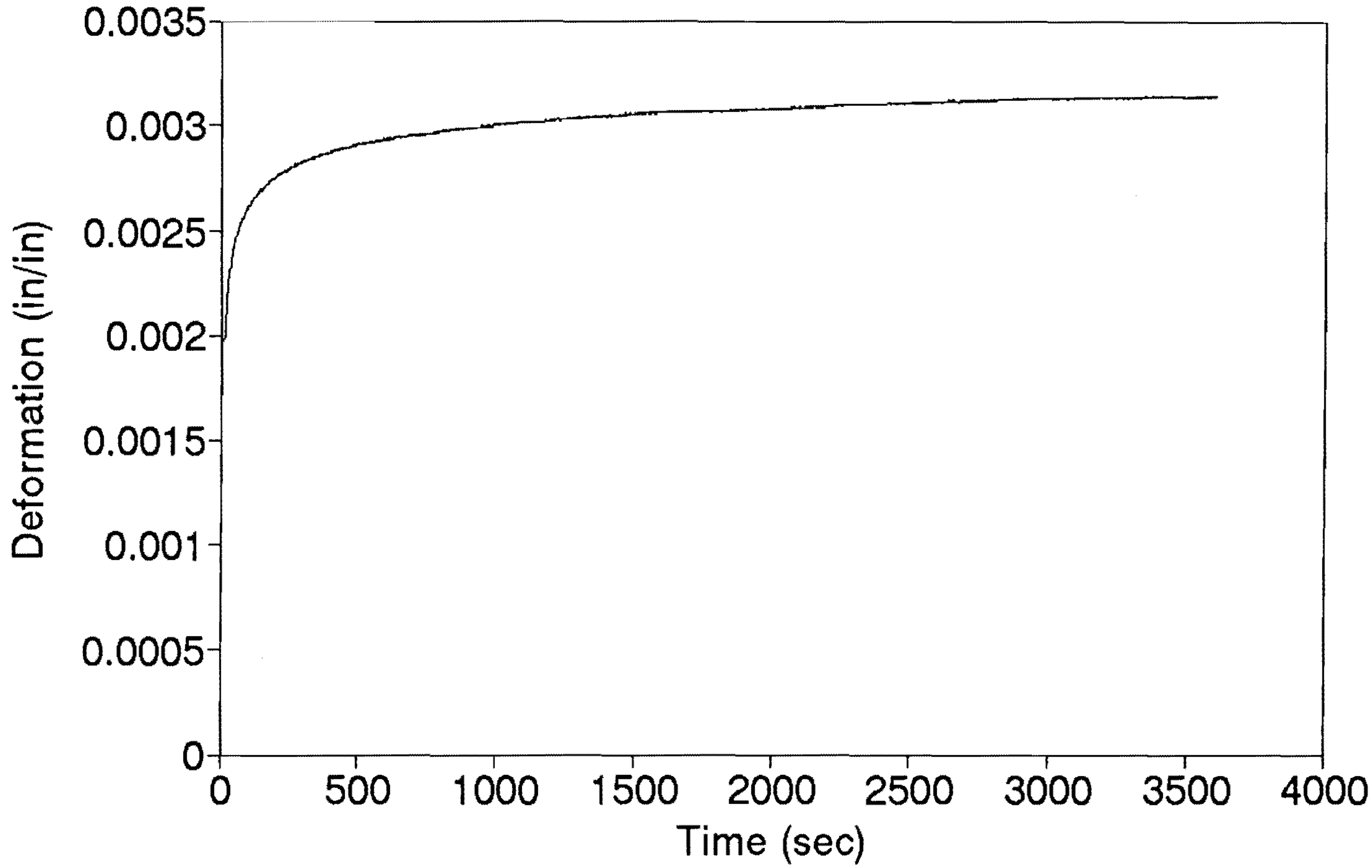


Figure 4.5. Static Creep Deformation Versus Time of Loading for DGF Mix (Dense-Graded Mix with Fine, Dry CRM).

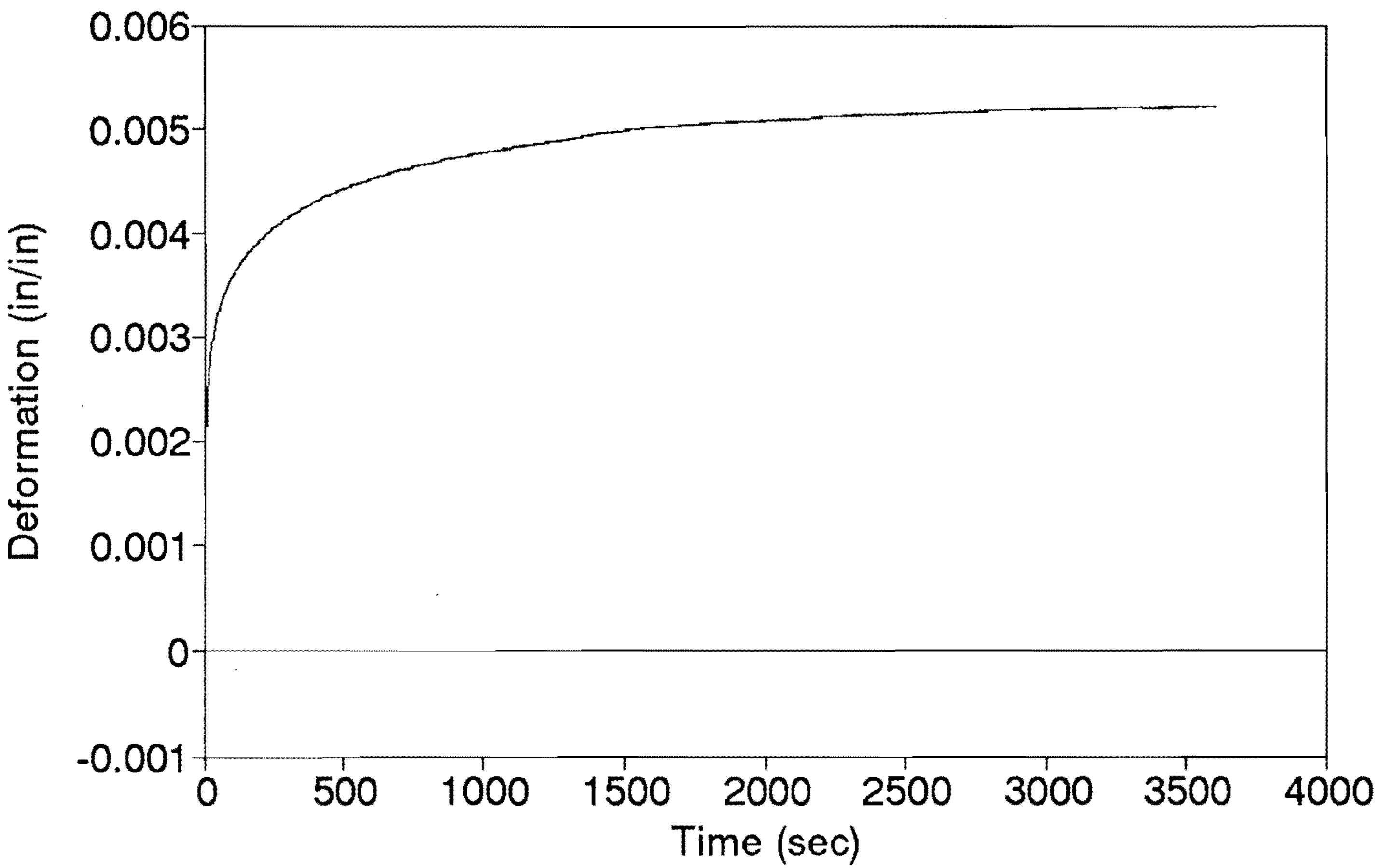


Figure 4.6. Static Creep Deformation Versus Time of Loading for DGC Mix (Dense-Graded Mix with Coarse, Dry CRM).

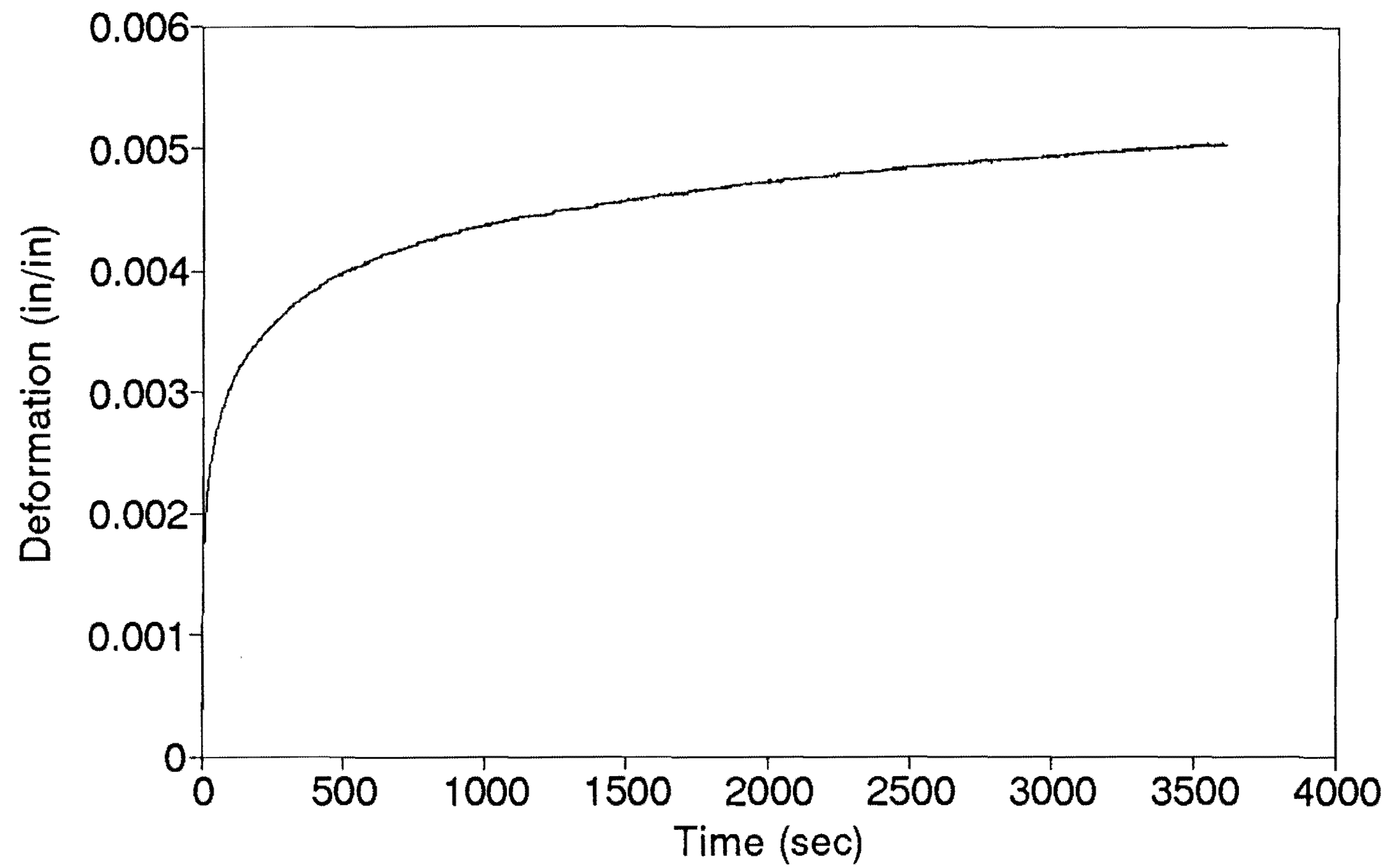


Figure 4.7. Static Creep Deformation Versus Time of Loading for 10%FW Mix (*10* percent *fine* CRM via *wet* process).

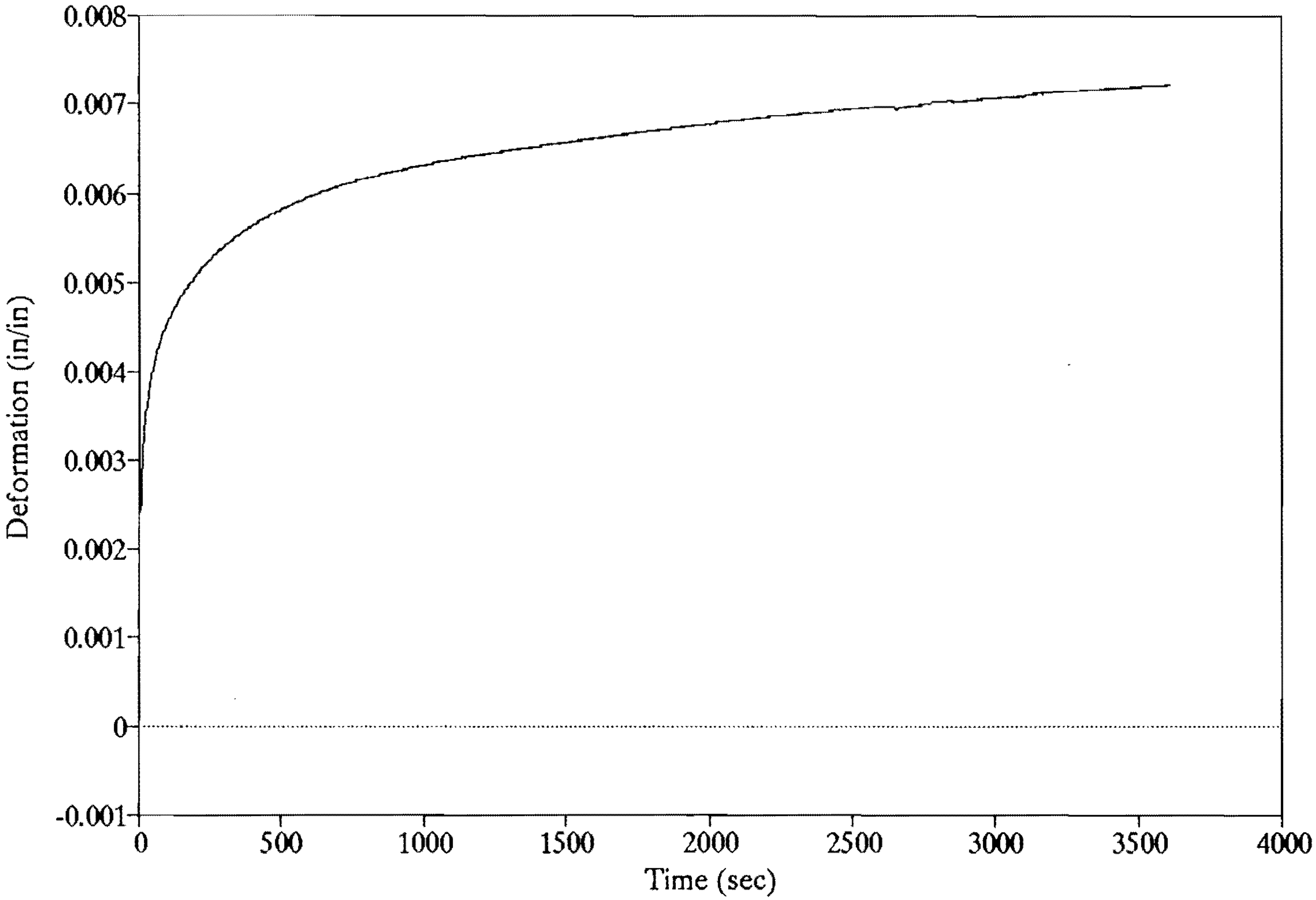


Figure 4.8. Static Creep Deformation Versus Time of Loading for 10%CW Mix (*10* percent *coarse* CRM via *wet* process).

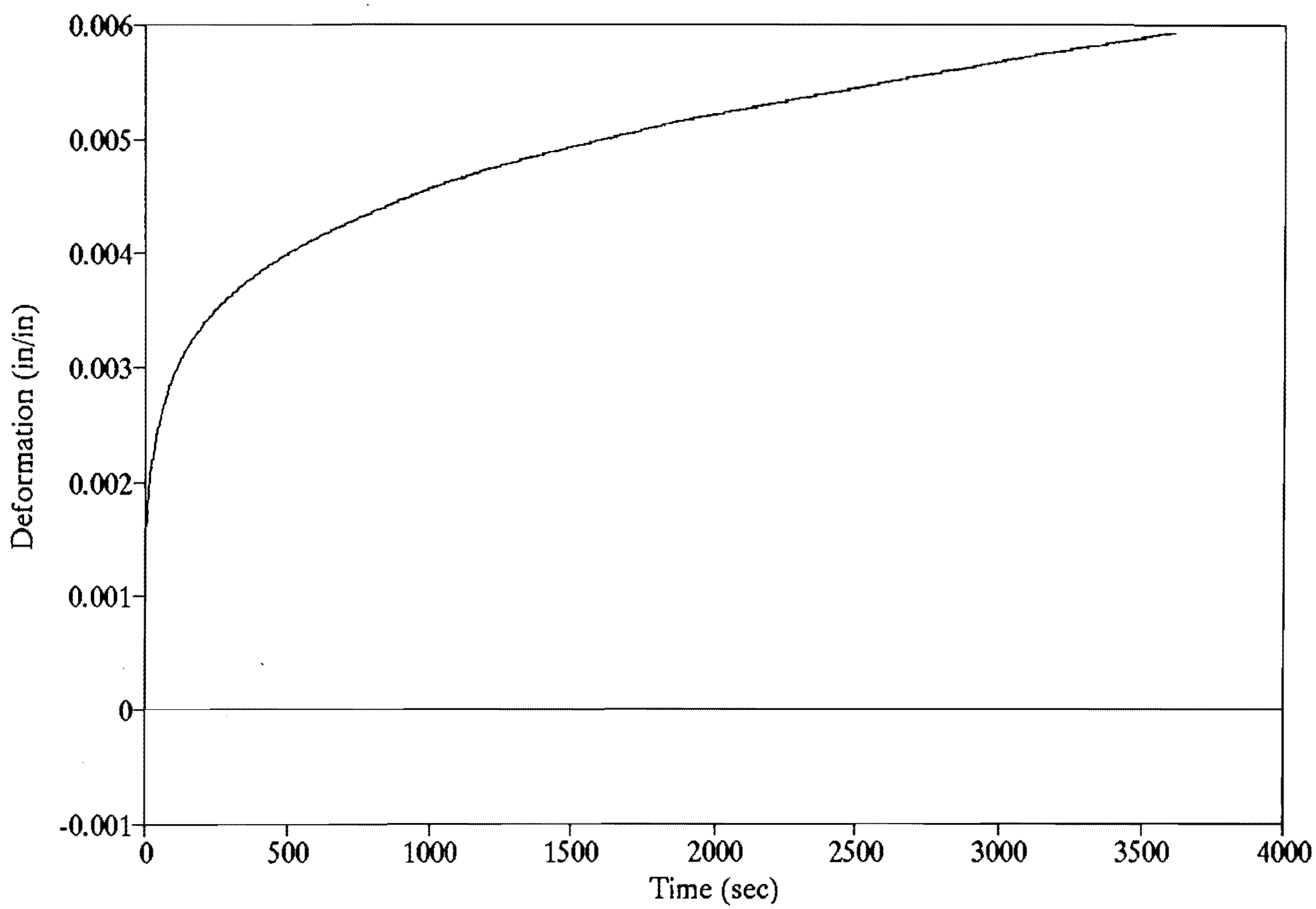


Figure 4.9. Static Creep Deformation Versus Time of Loading for 18%FW Mix (*18* percent *fine* CRM via *wet* process).

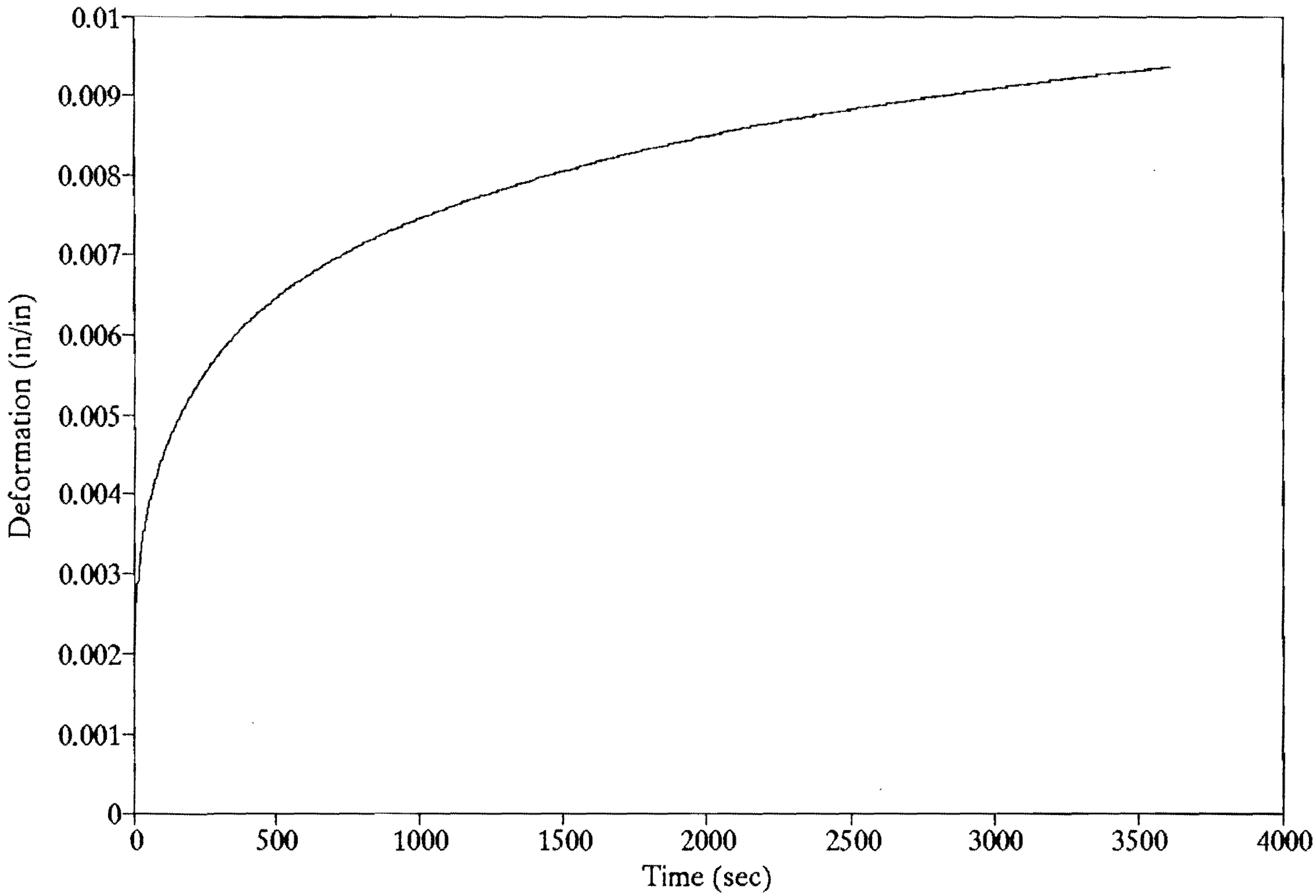


Figure 4.10. Static Creep Deformation Versus Time of Loading for 18%CW Mix (*18* percent *coarse* CRM via *wet* process).

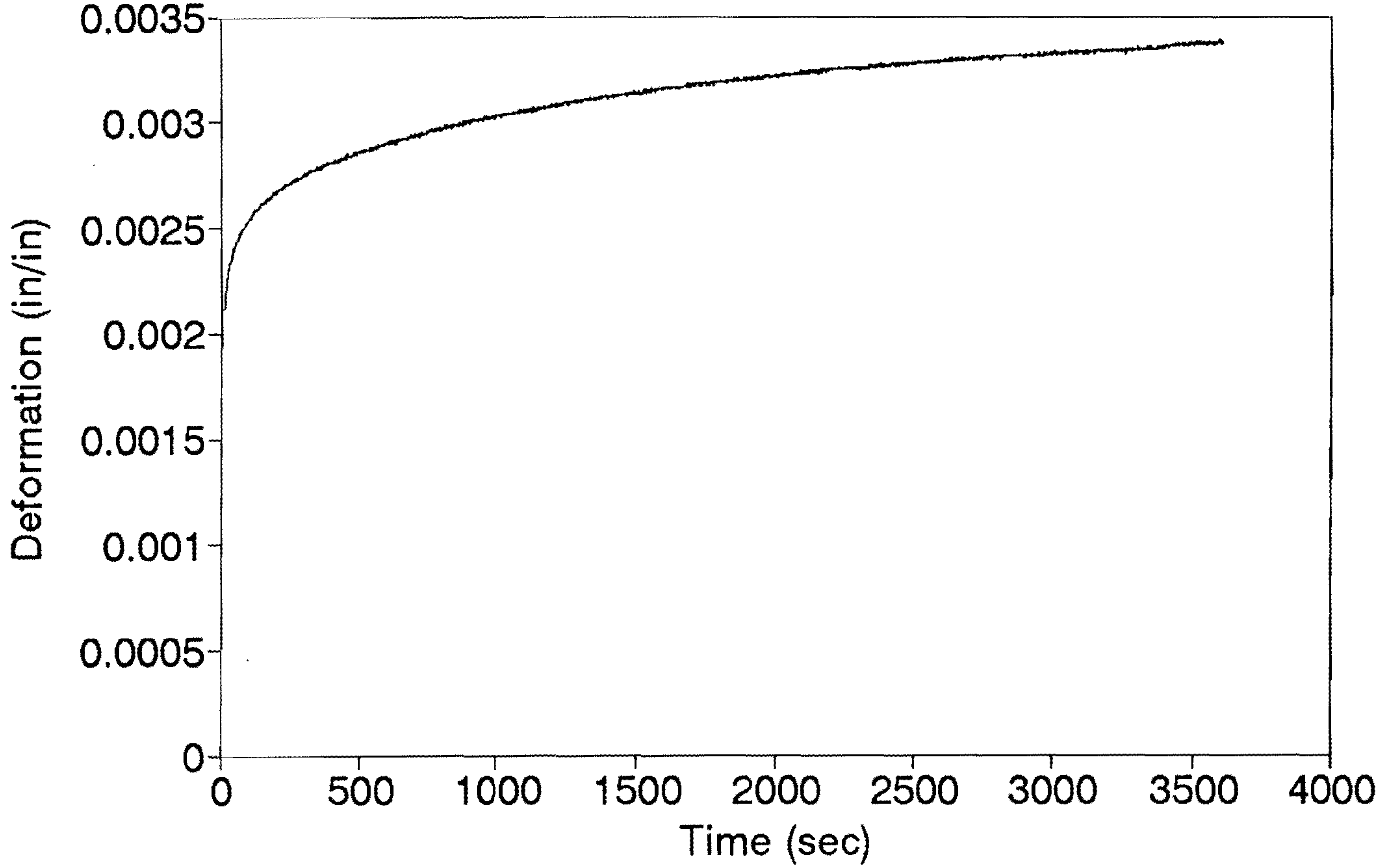


Figure 4.11. Static Creep Deformation Versus Time of Loading for 18%FD Mix (*18* percent *fine* CRM via *dry* process).

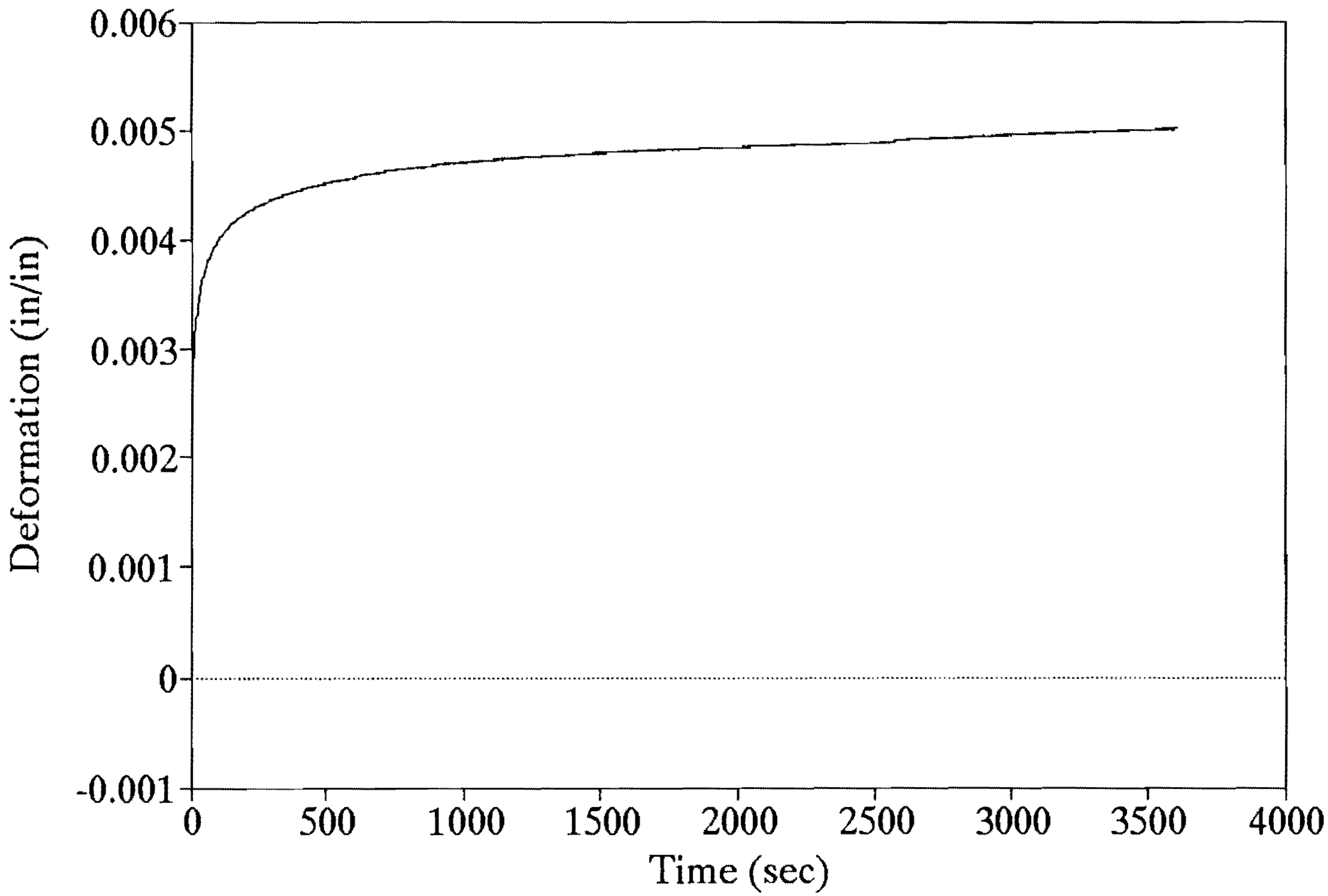


Figure 4.12. Static Creep Deformation Versus Time of Loading for 18%CD Mix (*18* percent *coarse* CRM via *dry* process).

A log-log slope of the creep versus time of loading curve (as shown in Table 4.1) of less than 0.25 is indicative of a mixture which will not become unstable (reach tertiary creep) within the testing period of 3,600 seconds (Little and Youssef 1992). All of the mixtures shown here have a slope less than 0.25. When observing the curves in Figures 4.4 through 4.12 and the slope data in Table 4.1, it does appear that some mixtures have significantly higher slopes than others. However, each of these slopes represents an average of three tests and a statistical analysis performed on these data revealed that none of these mixtures are significantly different from each other in terms of slope of the creep curve in the steady-state region.

The resilience offered by the aggregate and crumb rubber matrix should be considered if practicable in order to evaluate the permanent deformation potential of the mixture. Probably the most direct and simplest way to account for the effects of the aggregate/crumb rubber matrix on the resilience or "recoverability" of the mixture is by performing a recovery test immediately following the creep test. This allows one to judge the effects of the resilience or "recoverability" of the aggregate/crumb rubber matrix on the performance of the entire mixture.

After the 1-hour loading period in the creep test, there is a 1-hour recovery period. The percent strain recovered at the end of the 1-hour recovery is shown in Table 4.1. It appears, from this data, that the 2 dense-graded CRM mixtures (DGF and DGC) which have a similar gradation as the control have a much better recovery than the control. The dense-graded/fine CRM mixture had a higher creep stiffness at 3,600 seconds than the dense-graded/coarse CRM mixture. However, the dense-graded/coarse CRM mixture had a better recovery. Of the CMHB rubber mixtures, all but three (10%FW, 10%CW, and 18% CW) had a better recovery than the control mixture.

Based on the total strain at the end of the test, the mixtures were grouped into three categories in order to rank the mixtures from best to worst. The following criteria were arbitrarily selected for categorizing the mixtures using the total strain at the end of the static creep test:

Category 1	Total strain < 0.005 in/in,
Category 2	Total strain between 0.005 and 0.0075 in/in, and
Category 3	Total strain between 0.0075 and 0.0010 in/in.

Based on these criteria the laboratory mixtures can be categorized as follows (Category 1 mixtures being the best in terms of total strain at one hour):

Category 1: 18%FD (18% Fine CRM - Dry Method),
DGF (Dense-Graded with Fine CRM),
Control (Type D - no rubber);

Category 2: 18%CD (18% Coarse CRM - Dry Method),
10%FW (10% Fine CRM - Wet Method),
DGC (Dense-Graded with Coarse CRM),
18%FW (18% Fine CRM - Wet Method); and

Category 3: 10%CW (10% Coarse CRM - Wet Method),
18%CW (18% Coarse CRM - Wet Method).

It should again be emphasized that these criteria were arbitrarily selected in order to rank the mixtures for comparison with each other. The criteria is not related to field performance. In fact, based on AAMAS criteria in Figure 4.1, all of the mixtures above except 2 (10%CW and 18%CW) have a modulus at 1 hour loading considered to provide for low rutting potential. The 10%CW and 18%CW mixtures would be considered to have marginal mixture characteristics.

Repeated Load Uniaxial Creep Testing

The repeated load uniaxial creep test was performed to more closely simulate wheel loading than the static creep test. The laboratory simulation of wheel loading is generally considered the most reliable as it is believed to more closely simulate the stress conditions that occur in the pavement. The obvious difficulty with this test is that the equipment is expensive and intricate, and testing time is far too long for routine mixture design and/or analysis.

The primary difference between a repeated load creep test and a static test is the plastic deformation that occurs between loading applications. Bolk (1981) explains that the difference between static and repeated load testing can be much better understood by considering static load tests versus repeated load tests on aggregate systems without binder. Creep tests on these systems reveal that deformation is virtually independent of loading times. However, deformation is highly dependent on number of cycles. This difference is due to the plastic deformation that occurs at the particle-to-particle dry contacts. This plastic deformation or relative movement among particles is most effectively produced under dynamic loading conditions as the dynamic effect of each repetition produces some level of relative movement.

Generally, the static uniaxial creep test is sufficient to prioritize different mixtures in terms of relative resistance to permanent deformation. 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). The uniaxial creep test is highly dependent on the cohesion of the binder and the mastic portion of the mixture. In the case of a true triaxial test, mineral interlock plays an important role in the deformation resistance. Therefore, the deformation behavior in the pavement or in a realistic triaxial test is much more dependent on mineral interlock than in the uniaxial creep test. The only way to improve the creep test to better account for mineral interlock is through applying confinement. 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 triaxial test or Hveem stability test to evaluate the mineral aggregate internal friction.

The uniaxial repeated load permanent deformation test still suffers from the inability of the test 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 due to the ability to evaluate the effect of repeated loading on plastic deformation among aggregate particles. Thus, one of the most complete laboratory evaluations of permanent deformation would be one which incorporates confinement and cyclic loading.

All of the creep tests performed thusfar in this study (both repeated and static load creep), were performed without confining pressure. The CMHB rubber mixtures analyzed in this study are very similar in gradation to a stone mastic-type mixture. When performing creep tests on these mixtures without confining pressure, these type of mixtures may lack the lateral support that is present during the field. It is believed that because of the aggregate interlock that exists in a dense-graded mixture, unconfined uniaxial creep properties may be better for dense-graded mixtures than for stone mastic-type mixtures. However, field performance may be better for a stone mastic-type mixture. This important factor must be kept in mind when reviewing unconfined uniaxial creep properties for both dense and CMHB mixtures. It is appropriate to compare or rank CMHB mixtures against each other but it may not be appropriate to compare CMHB mixtures to dense-graded mixtures.

The repeated load uniaxial creep curves for all the mixtures are presented in Figures 4.13 through 4.21 along with the corresponding static load creep curves. Tabulated data is shown below in Table 4.2. The data for the repeated load creep test are plotted to 3600 seconds for comparison with the static creep data; however, in the repeated load test, the samples were loaded to 10,000 cycles.

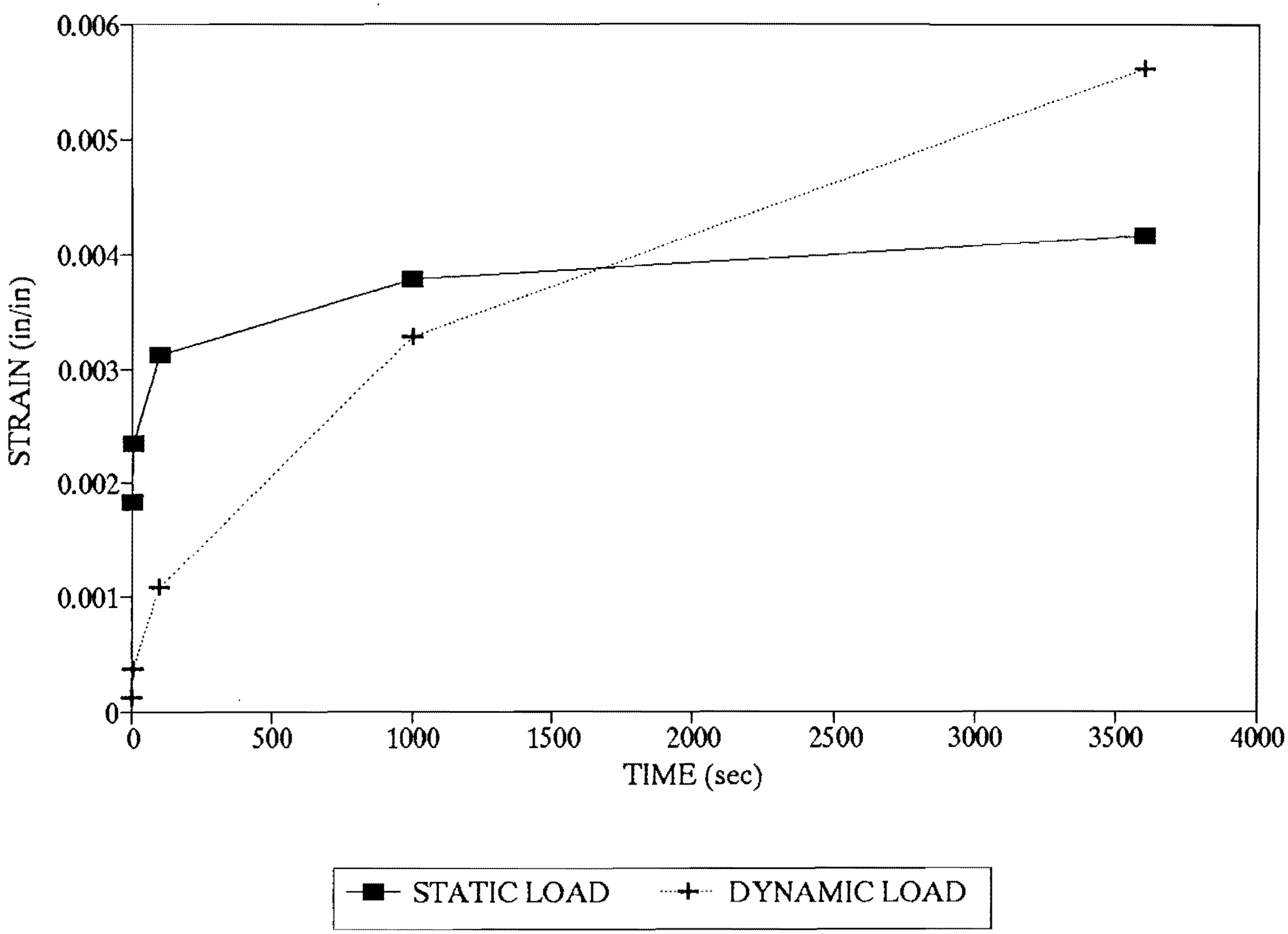


Figure 4.13. Repeated Load and Static Creep Strain Versus Time of Loading for Type D, Control Mix (no rubber).

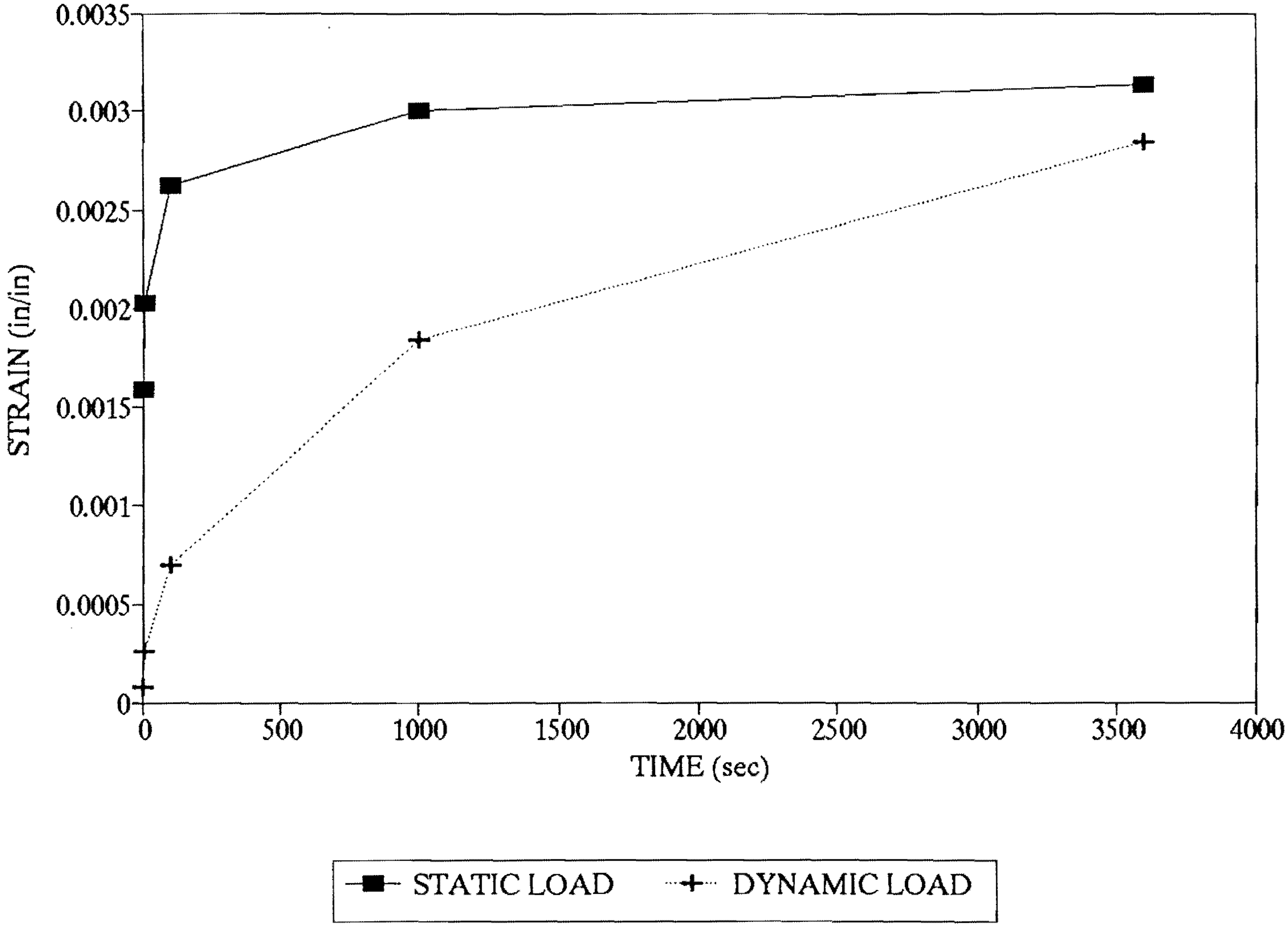


Figure 4.14. Repeated Load and Static Creep Strain Versus Time of Loading for DGF Mix (Dense-Graded Mix with Fine, Dry CRM).

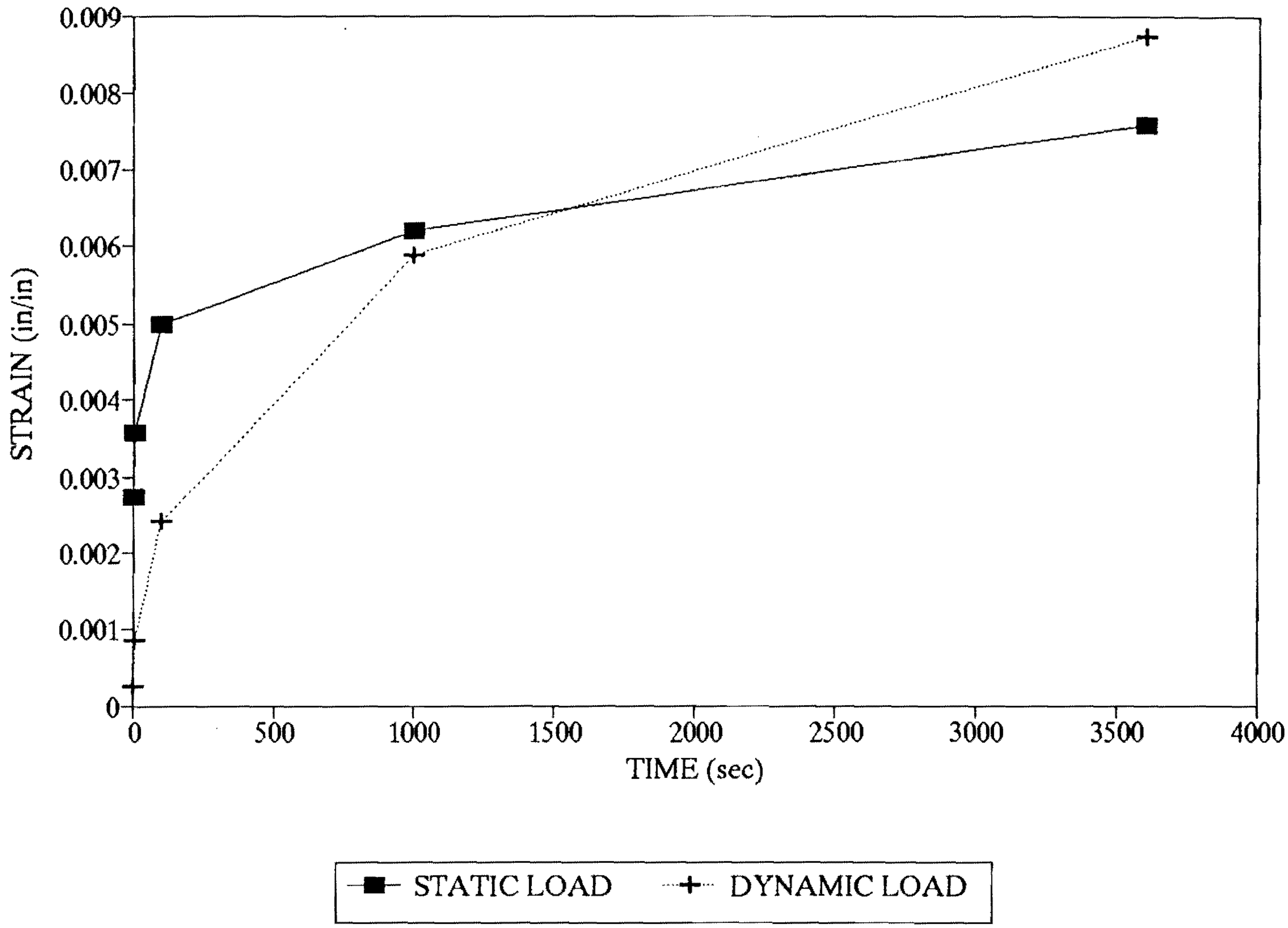


Figure 4.15. Repeated Load and Static Creep Strain Versus Time of Loading for DGC Mix (Dense-Graded Mix with Coarse, Dry CRM).

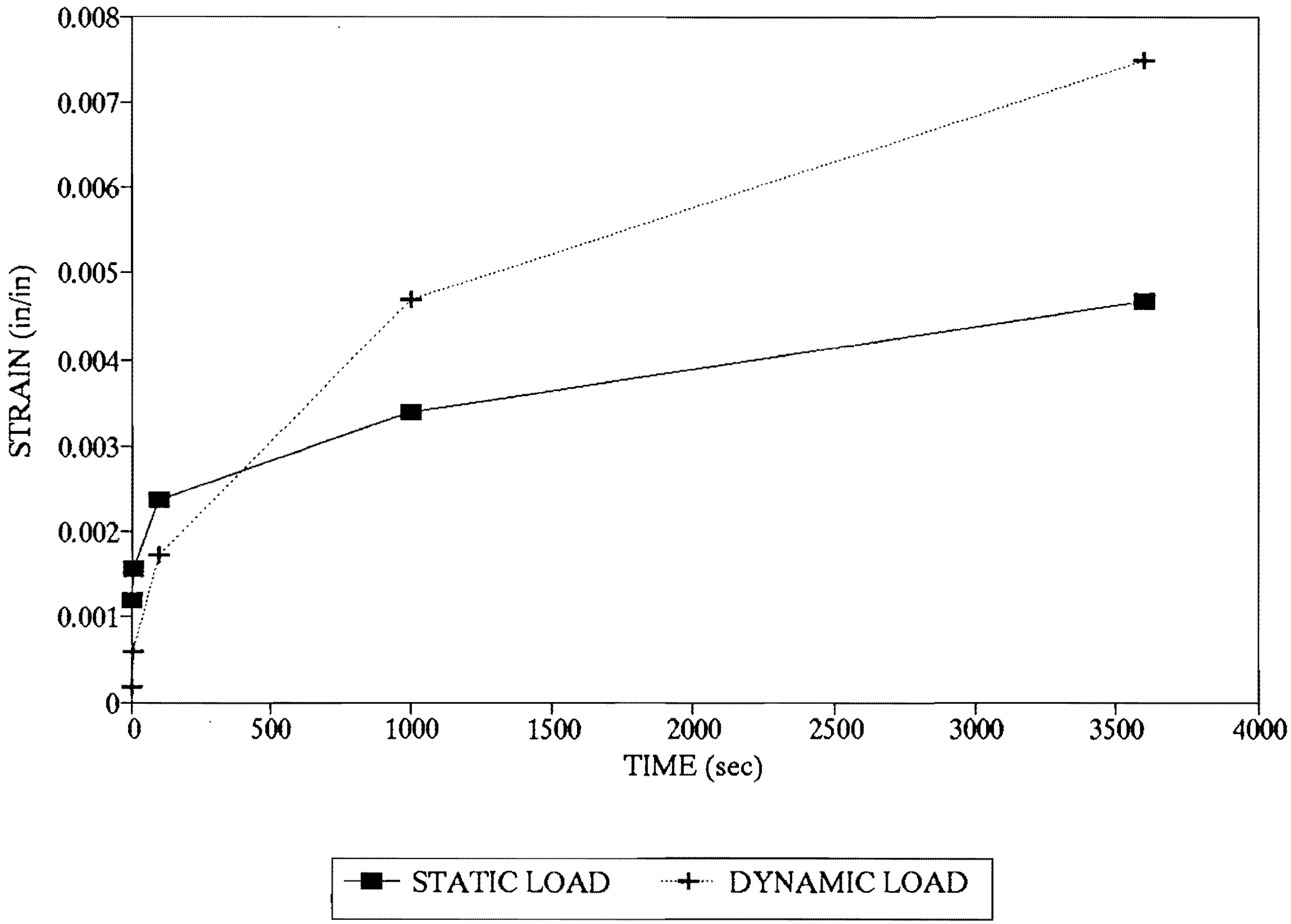


Figure 4.16. Repeated Load and Static Creep Strain Versus Time of Loading for 10%FW Mix (*10* percent *fine* CRM via *wet* process).

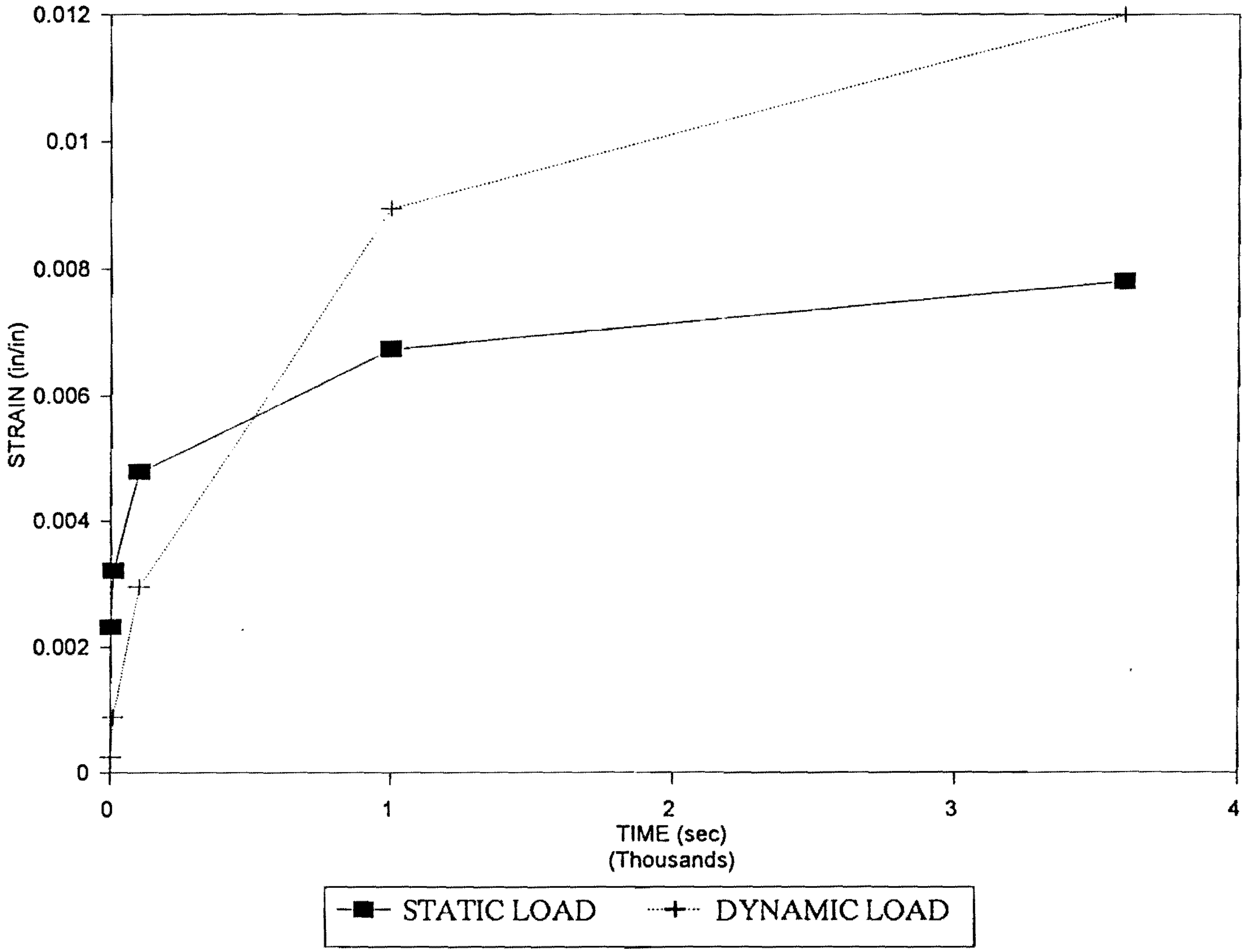


Figure 4.17. Repeated Load and Static Creep Strain Versus Time of Loading for 10%CW Mix (*10* percent *coarse* CRM via *wet* process).

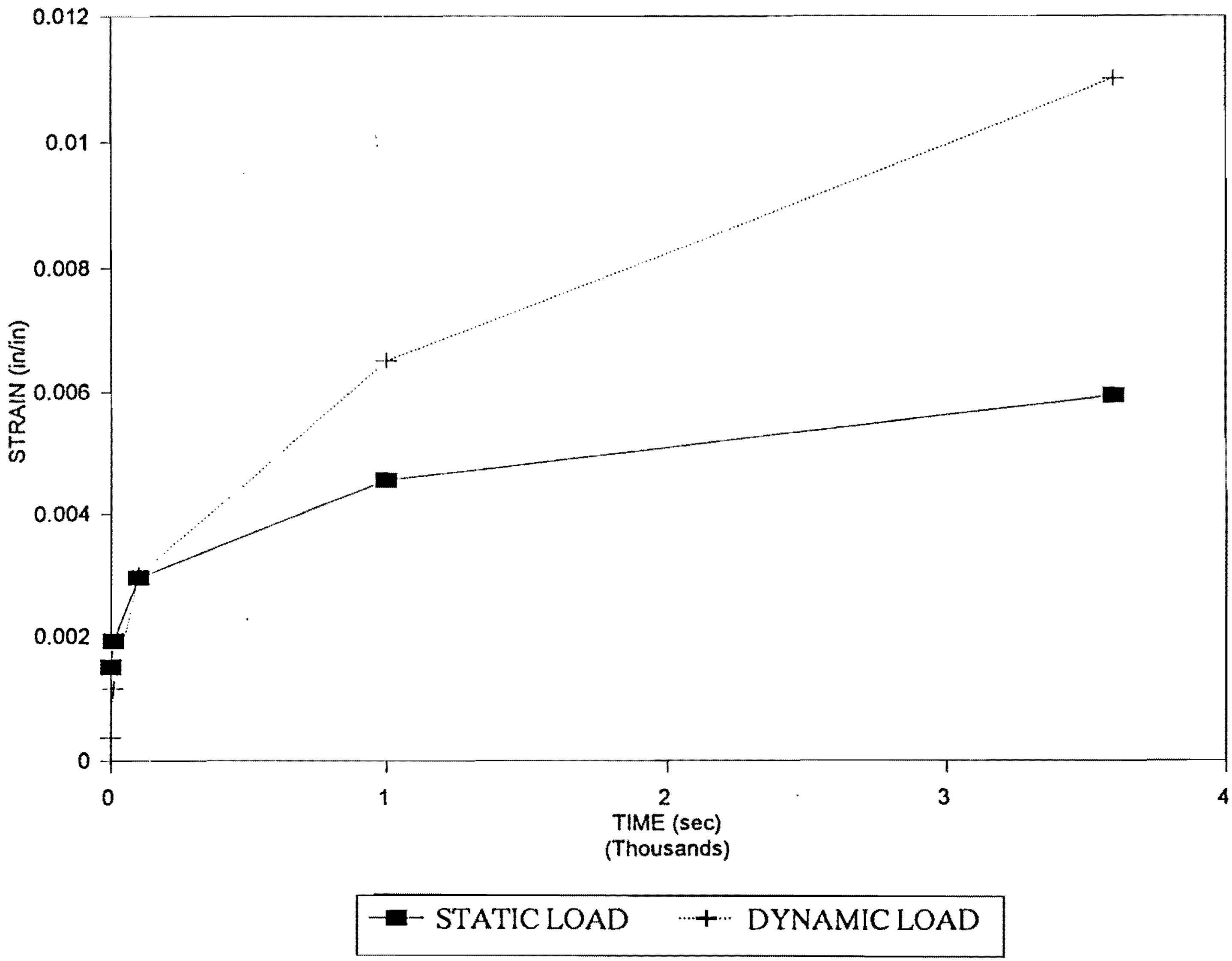


Figure 4.18. Repeated Load and Static Creep Strain Versus Time of Loading for 18%FW Mix (*18* percent *fine* CRM via *wet* process).

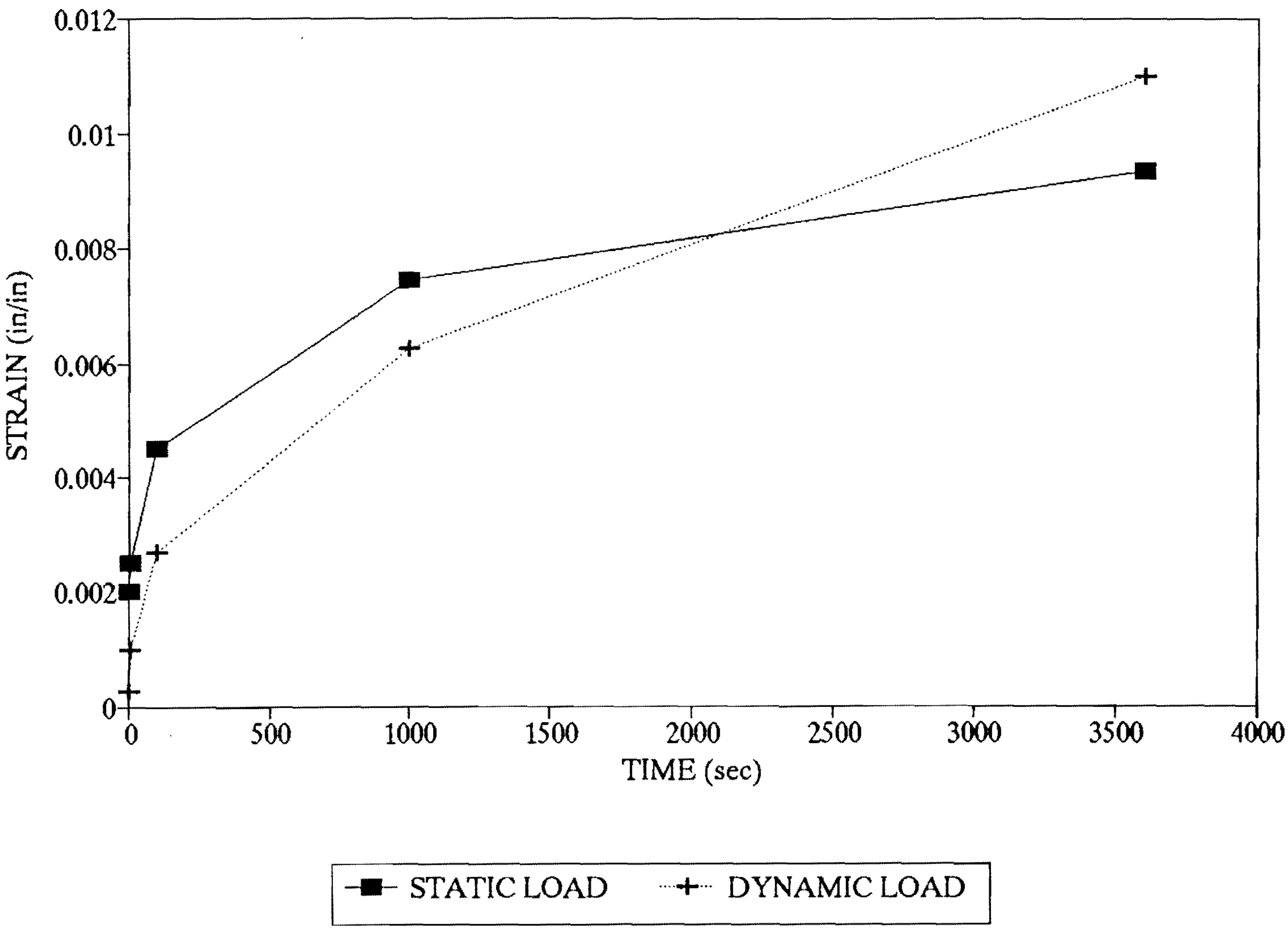


Figure 4.19. Repeated Load and Static Creep Strain Versus Time of Loading for 18%CW Mix (*18* percent *coarse* CRM via *wet* process).

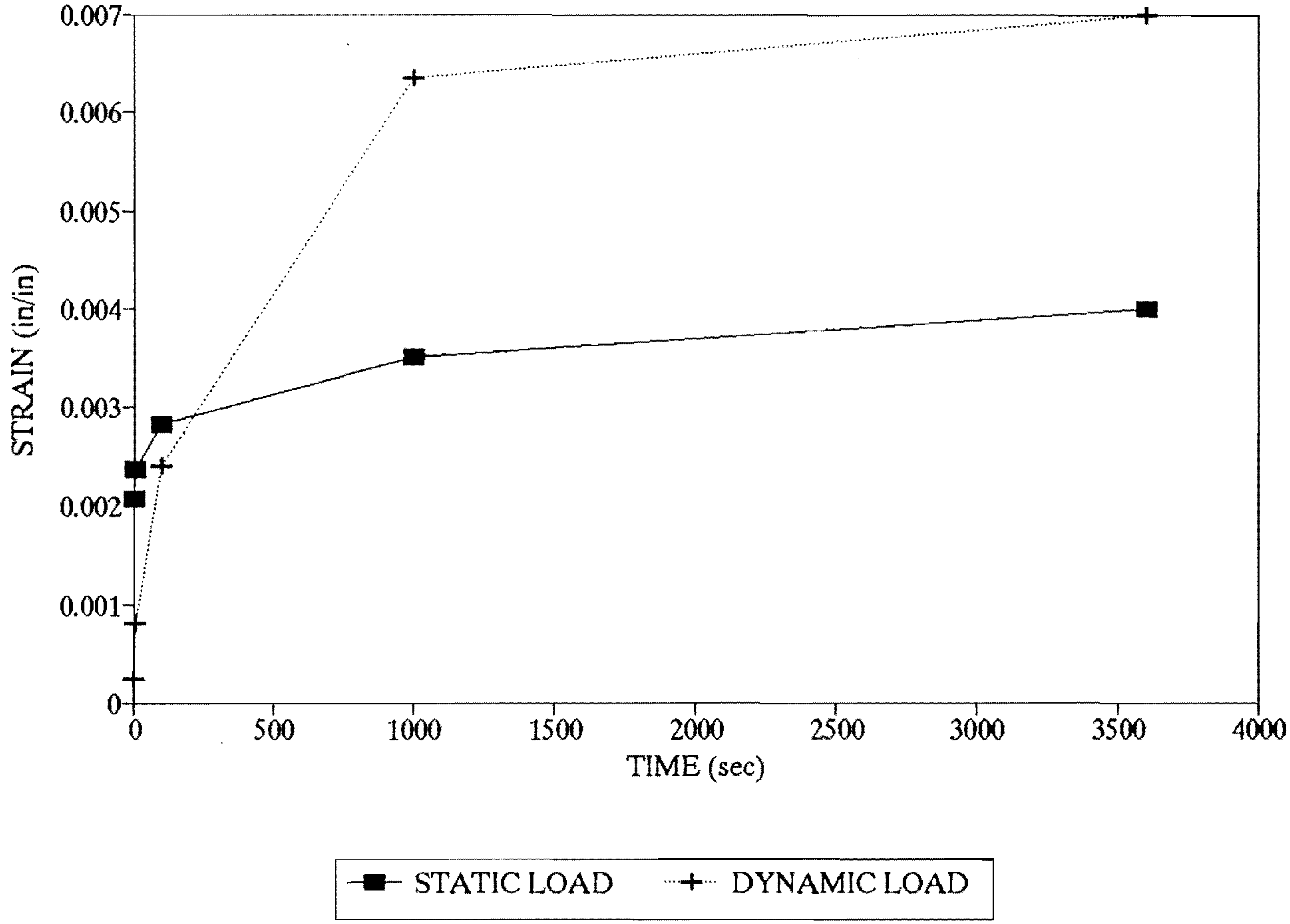


Figure 4.20. Repeated Load Static Creep Strain Versus Time of Loading for 18%FD Mix (*18* percent *fine* CRM via *dry* process).

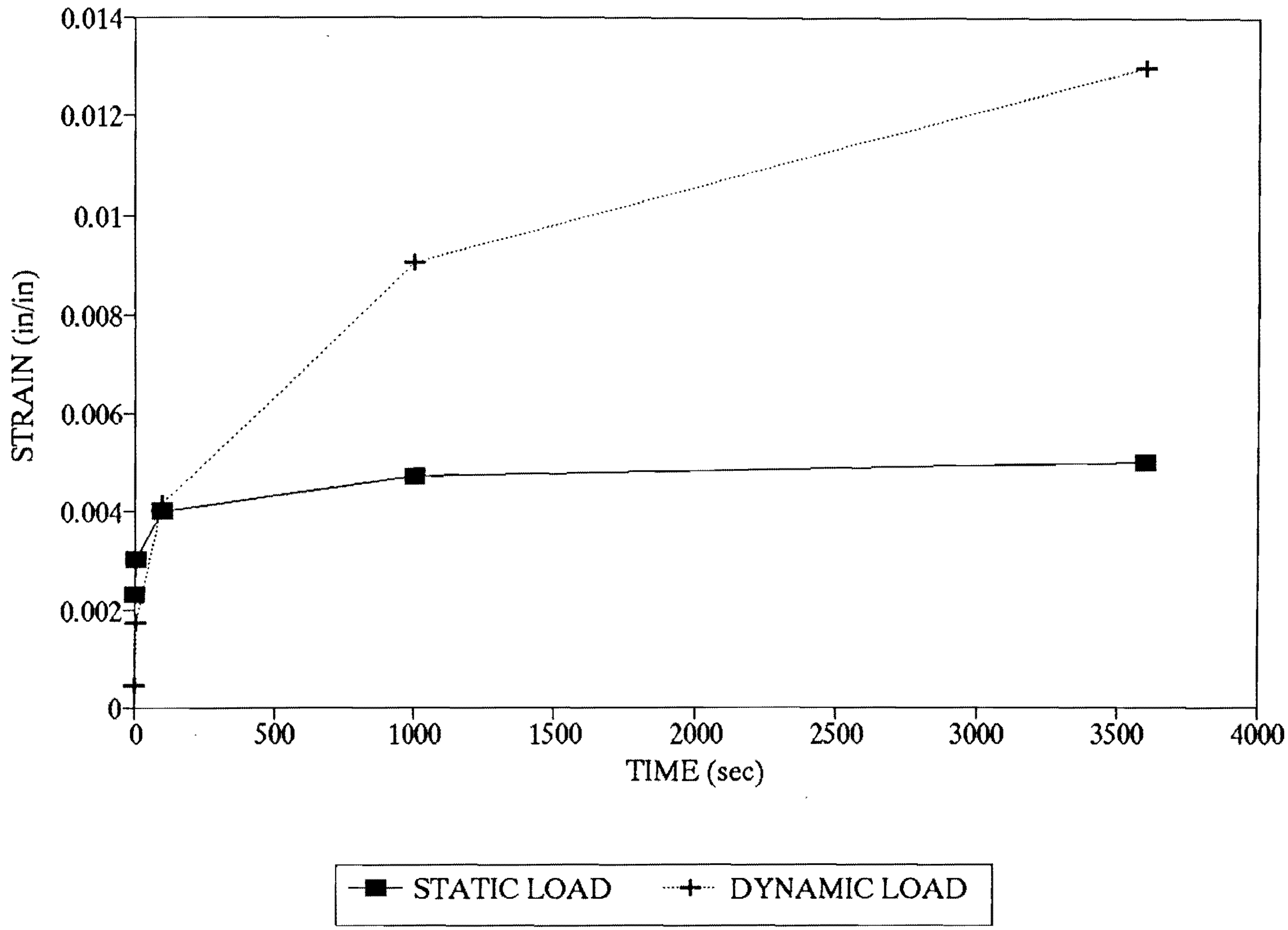


Figure 4.21. Repeated Load and Static Creep Strain Versus Time of Loading for 18%CD Mix (*18* percent *coarse* CRM via *dry* process).

Table 4.2. **Uniaxial Repeated Load Creep Data (σ_1 = 60 psi [414 kPa]) for Control and Crumb-Rubber Mixtures.**

Mixture Type	Log-Log Slope of Steady-State Creep Curve	Strain at End of 10,000 Cycles in/in	Strain Recovery, percent
Control	0.423	0.007079	1.4
Dense-Graded with Fine CRM (DGF)	0.360	0.004149	3.0
Dense-Graded with Coarse CRM (DGC)	0.313	0.011609	3.0
10% Fine CRM - Wet Method (10%FW)	0.463	0.016657	0.3
10% Coarse CRM - Wet Method (10%CW)	0.428	0.018085	0.7
18% Fine CRM - Wet Method (18%FW)	0.324	0.014358	1.8
18% Coarse CRM - Wet Method (18%CW)	0.356	0.015528	2.8
18% Fine CRM - Dry Method (18%FD)	0.365	0.013312	2.3
18% Coarse CRM - Dry Method (18%CD)	0.328	0.018772	4.9

As shown in Figures 4.13 through 4.21, the dynamic loading causes the samples to deform at a much higher rate (as evidenced by the greater slope of the secondary portion of the curve) than in the static creep test. The slopes for these mixtures are also shown in Table 4.2. As in the static creep test, none of these mixtures are significantly different (statistically) from each other in terms of slope in the steady state portion. The percent strain recovery for all the mixtures (Table 4.2) is very low which is probably to be expected after 10,000 load cycles.

As with the static creep data, the mixtures were categorized and ranked according to the total strain at the end of 10,000 load cycles. Based on the total strain at the end of the test, the mixtures were grouped into three categories. The following criteria were arbitrarily selected for categorizing the mixtures using the total strain at the end of the static creep test:

Category 1	Total strain < 0.010 in/in,
Category 2	Total strain between 0.010 and 0.015 in/in, and
Category 3	Total strain between 0.015 and 0.020 in/in.

Based on these criteria the laboratory mixtures can be categorized as follows in Table 4.3 (Category 1 mixtures being the best in terms of total strain) and compared with the rankings of the mixtures based on static creep tests.

Table 4.3. Ranking of Laboratory Mixtures (from Best to Worst) Based on Total Strain at the End of the Test Period for Both Static and Repeated Load Creep Tests.

Ranking Category	Based on Static Creep Test	Based on Repeated Load Creep Test
Category 1	DGF Control 18%FD	DGF Control
Category 2	DGC 18%FW 18%CD 10%FW	18%FD DGC 18%FW
Category 3	10%CW 18%CW	18%CD 10%FW 10%CW 18%CW

Because the repeated load creep test is more rigorous than the static creep test, some of the mixture rankings changed slightly. Both the DGF and Control mixtures remained in Category 1 after repeated load creep testing; however, the 18%FD dropped to Category 2. The DGC and 18%FW remained in Category 2, while the 18%CD and 10%FW dropped to Category 3. Mixtures designated as 10%CW, and 18%CW remained in Category 3.

TxDOT Static Creep Test

The TxDOT Static Creep Test is performed on 4-inch (10.2 cm) diameter by 2-inch (5.1 cm) high samples at a stress level of 10 psi (70 kPa). The specimen is loaded for one hour with a 10 minute recovery period. This test was performed on all mixture specimens along with Hveem stability; however, analysis and verification of this data is incomplete at the time of this report.

4.2 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 development of fatigue cracks is related to the tensile strain at the bottom of the asphaltic concrete layer. Figure 4.22 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 4.22 is derived based on the generalized fatigue relationship:

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

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

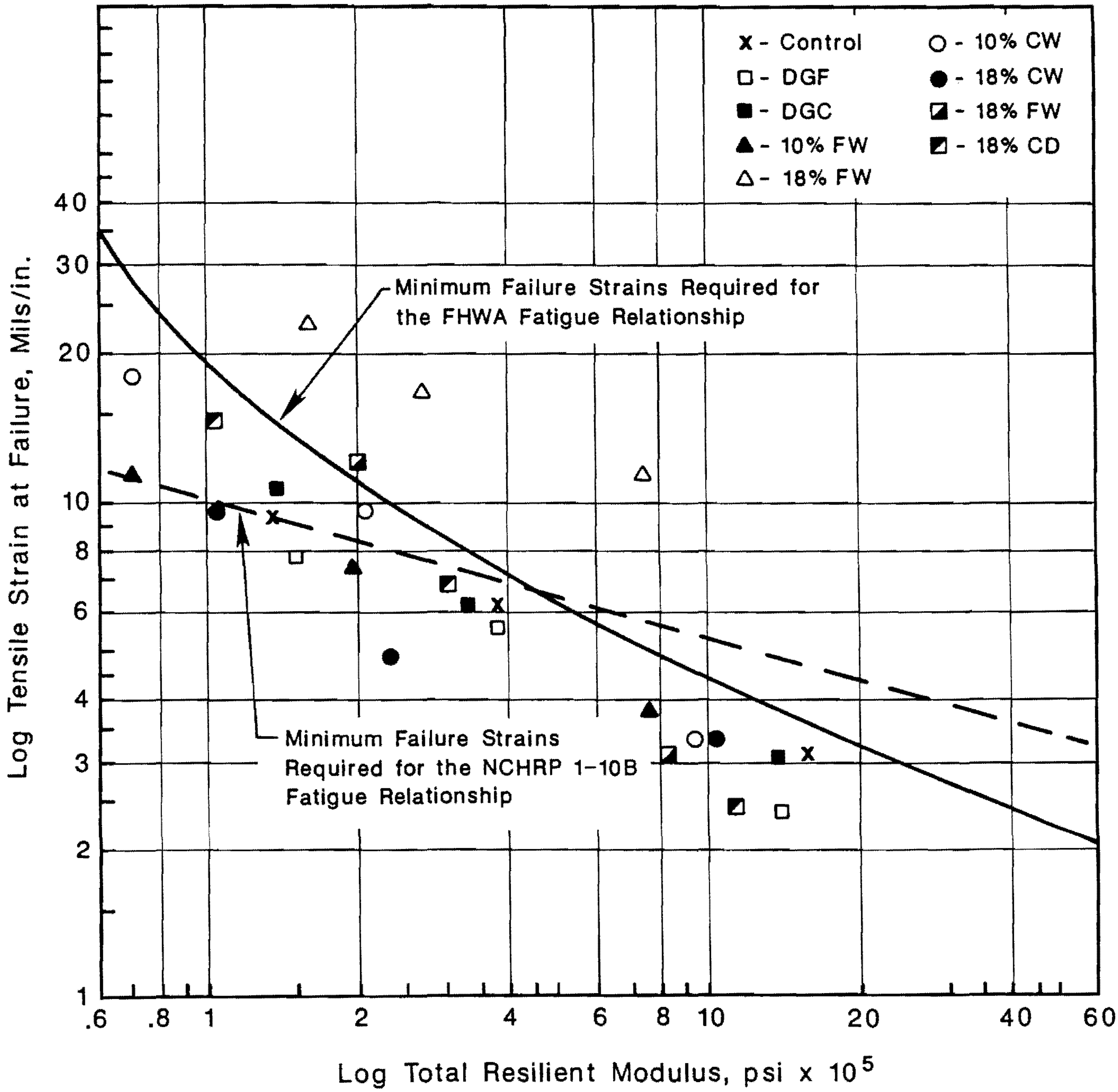


Figure 4.22. Control and CRM Mixtures - AAMAS Chart for Indirect Tensile Strains Versus Resilient Modulus.

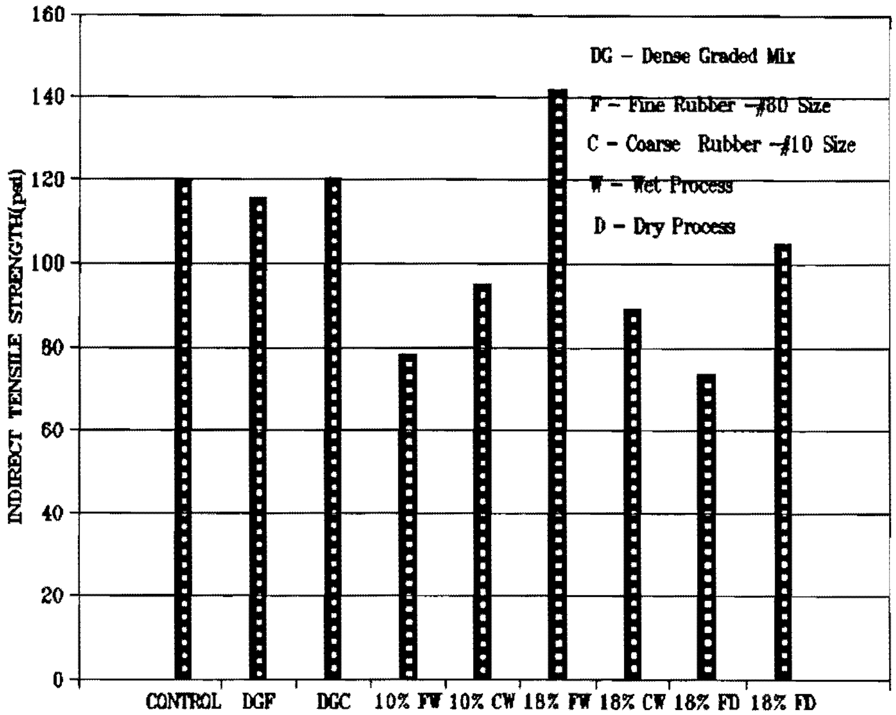
constants.

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, Inc. 1975). Figure 4.22 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 (Von Quintus et al. 1991). From Figure 4.22, 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% fine CRM by the wet method has a significantly better fatigue resistance than the others.

Indirect Tensile Tests

Indirect tensile strength tests were performed at 41°F (5°C), 77°F (25°C) and 104°F (40°C). Figure 4.23 shows the indirect tensile strengths and failure strains at 41°F (5°C). The tensile strength of the control mixture which is a dense graded mix is 120 psi (827 kPa). The addition of dry fine and coarse rubber to a dense graded mixture (DGF and DGC) did not cause a decrease in the tensile strength. Tensile strength and failure strain data at 41°F (5°C) for all three dense graded mixtures (Control, DGF,



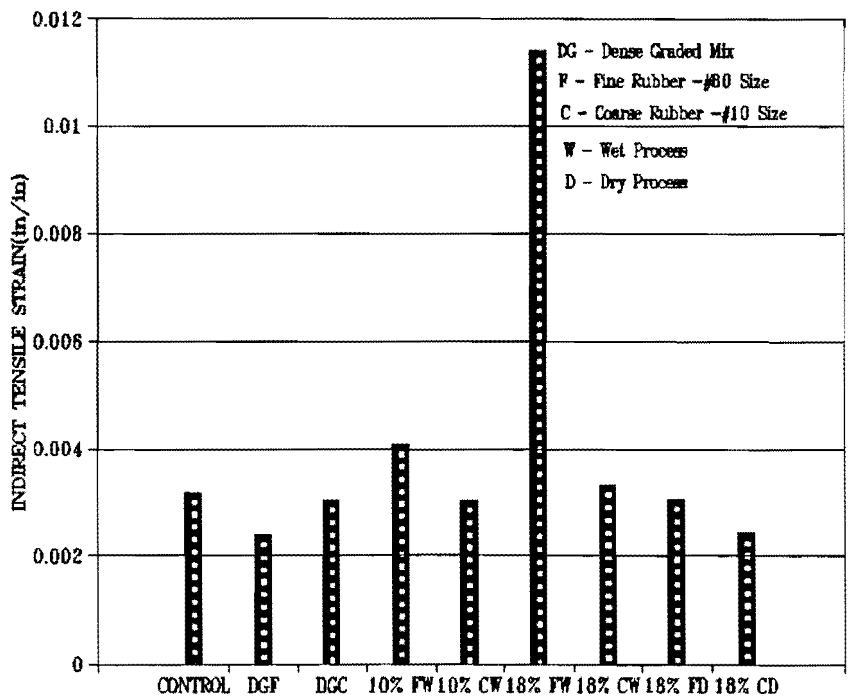


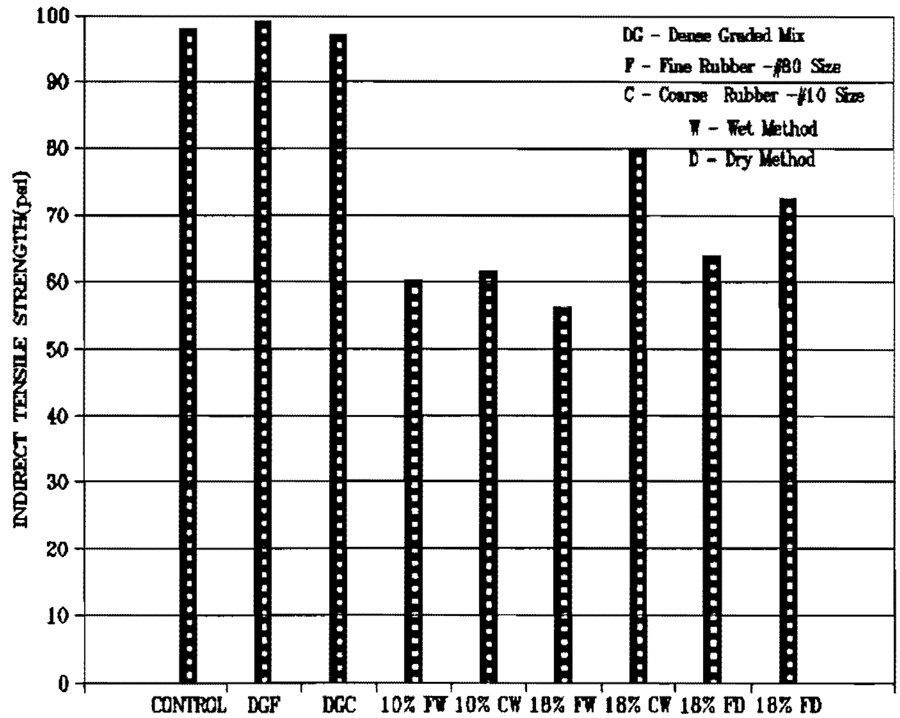
Figure 4.23. Indirect Tensile Strength and Strain at Failure at 41°F (5°C) for Control and CRM Mixtures.

and DGC) were about the same. The remaining crumb rubber mixtures (which were CMHB) exhibited a decrease in tensile strength ranging from about 15 to 35% with the exception of the 18%FW (18% fine rubber - wet method). This mixture displayed a significant increase in tensile strength over the control mixture. Tensile strain at failure was also much higher for this mixture than for any of the others.

While the tensile strengths shown in Figure 4.23 for 41°F (5°C) decreased for most of the CMHB rubber mixtures as compared with the control, there was no decrease in the tensile strain at failure for these mixtures. In fact, the tensile strain at failure for the CMHB rubber mixtures was as good or better than the control in most cases.

Tensile strengths and failure strains at 77°F (25°C) are shown in Figure 4.24. A similar trend in the data is observed here as for 41°F (5°C). Tensile strength and failure strain data at 77°F (25°C) for all three dense graded mixtures (Control, DGF, and DGC) were about the same. The remaining crumb rubber mixtures exhibited a decrease in tensile strength ranging from 15 to 35%. Higher failure strains were observed for mixtures 10%FW and 10%CW, and dramatically higher failure strains were observed for the 18%FW mixture.

Tensile strengths and failure strains at 104°F (40°C) are shown in Figure 4.25. As one would expect, tensile strength of bituminous mixtures greatly decreases at 104°F (40°C). Tensile strength of the three dense graded mixtures (Control, DGF, and DGC) ranged from 30 to 36 psi. The two CMHB mixtures made with 10 percent CRM (10%FW and 10%CW) had the lowest tensile strengths at about 17 psi. The four remaining CMHB crumb rubber mixtures all contained 18% CRM (18%FW, 18%CW, 18%FD, and 18%FD) with tensile strengths ranging from 23 to 36 psi. Failure strains for the CMHB crumb rubber mixtures were generally higher than the control and dense-graded crumb rubber mixtures.



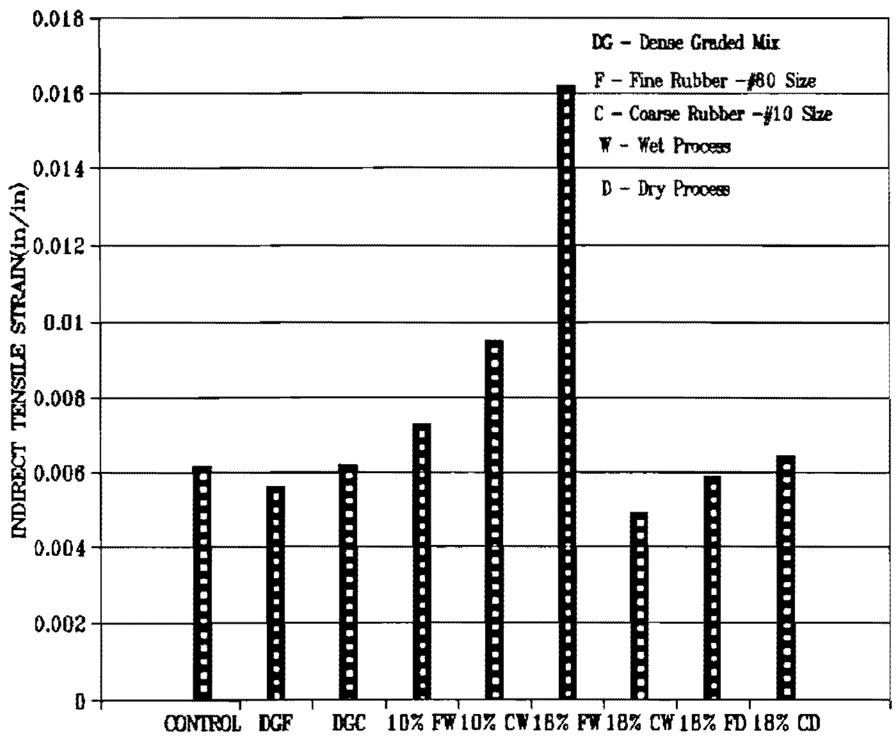
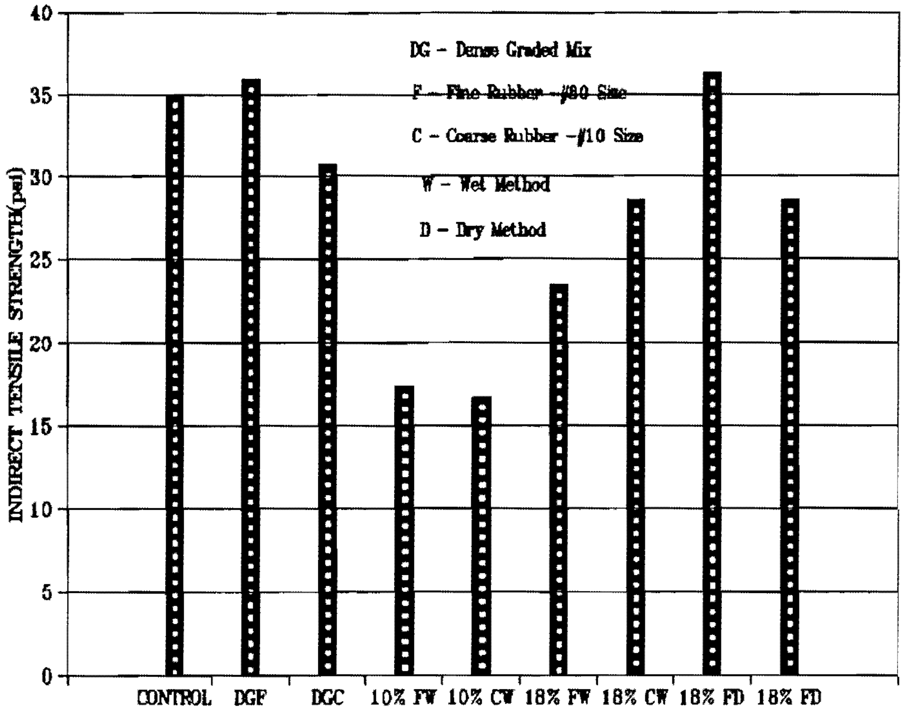


Figure 4.24. Indirect Tensile Strength and Strain at Failure at 77°F (25°F) for Control and CRM Mixtures.



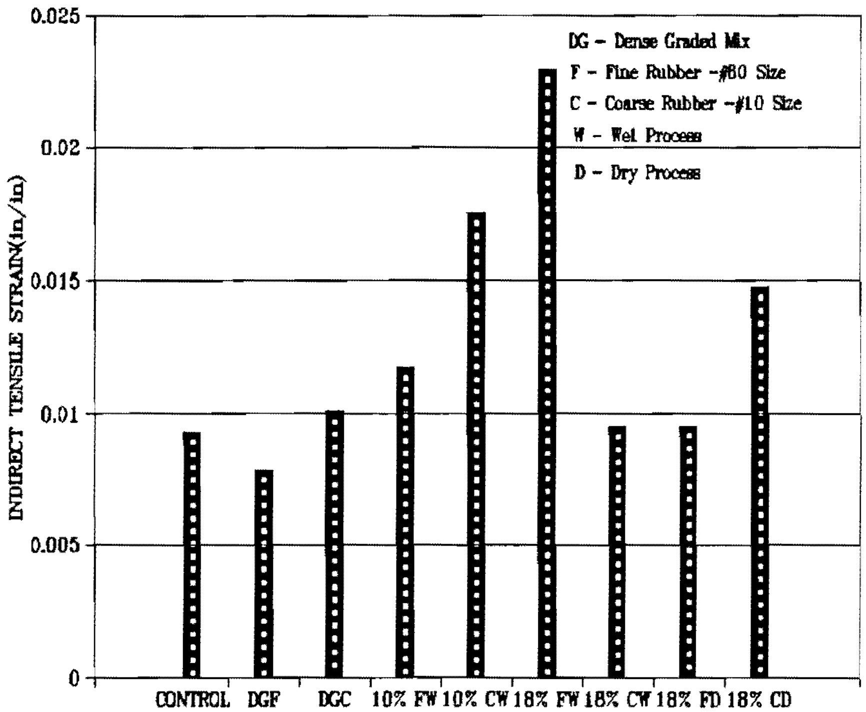


Figure 4.25. Indirect Tensile Strength and Strain at Failure at 104°F (40°C) for Control and CRM Mixtures.

In summary of the indirect tensile test data, the following observations can be made:

- The 2 dense-graded crumb rubber mixtures, 1 containing coarse CRM and one containing fine CRM (DGF and DGC), had the same tensile strength as the dense-graded control mixture at all 3 temperatures tested.
- There was no difference between the dense-graded mixture produced with coarse rubber and the 1 produced with fine rubber in terms of both tensile strength and failure strain (at all three temperatures tested).
- There was quite a bit of variation among the CMHB crumb rubber mixtures; however, in general, the CMHB crumb rubber mixtures had lower tensile strengths than all three dense-graded mixtures tested (15 to 35% lower). Discretion should be used when making comparisons between the CMHB crumb rubber mixtures and the control mixture which is dense graded. While lower tensile strength may be attributed to the crumb rubber type and process used to incorporate the rubber, other factors (in particular, aggregate gradation) can affect the tensile strength.
- Failure strains were somewhat higher for the CMHB crumb rubber mixtures than for the control and dense-graded crumb rubber mixtures.
- One mixture seemed to "stand-out" from all others in terms of having significantly higher failure strains at all three test temperatures: the CMHB mixture containing 18% fine rubber via the wet process (18%FW).

Resilient Modulus Testing

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 4.26 and are tabulated in the appendices. The addition of CRM in the dense-graded mixtures caused a decrease in the resilient modulus at 41° (5°C) and 77°F (25°C) over that of the control. All of the CMHB crumb rubber mixtures had lower stiffnesses than the dense-graded mixtures at all three test temperatures. It appears that CRM may have some propensity for decreasing the mixture's temperature susceptibility. In particular, the mixture made with 18 percent fine CRM by the wet process (18%FW). This mixture had the lowest

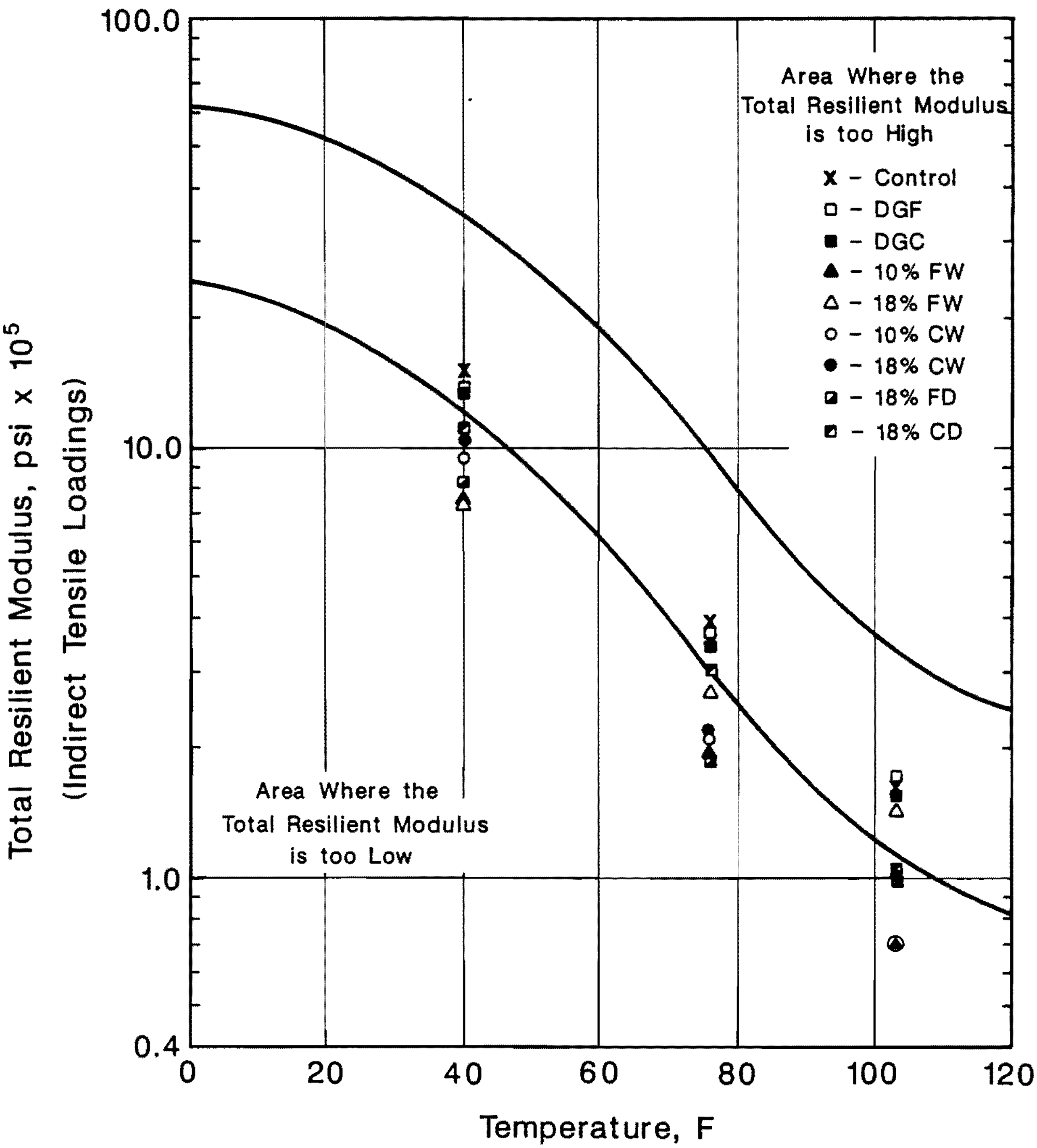


Figure 4.26. Control and CRM Mixtures - AAMAS Chart for Resilient Modulus Versus Temperature.

stiffness at 41°F (5°C) and yet a relatively high stiffness at 104°F (40°C). This trend was not observed, however, for the two CMHB mixtures made with 10 percent CRM (10%CW and 10%FW) which exhibited the lowest stiffnesses of all the mixtures at 104°F (40°C).

Figure 4.26 is the AAMAS chart for plotting the test results of total resilient modulus (unconditioned) versus temperature as compared to the range of values that are appropriate for higher volume roadways. In general, the CMHB crumb rubber mixtures have resilient modulus values that are considered to be too low based on this particular criteria. However, this criteria was developed for dense-graded asphaltic concrete mixtures and may not be applicable to CMHB mixtures.

4.3 Moisture Damage

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 and resilient moduli ratios, TSR and MRR) of AAMAS is simply used as a means of accepting or rejecting a mixture. Both of these values should exceed a value of 0.80 for a dense-graded mixture. Tensile strength ratio (TSR) is shown in Figure 4.27. TSR is the tensile strength after moisture conditioning divided by the tensile strength of unconditioned specimens tested at 77°F (25°C). All of the mixtures exceeded the minimum requirement of 0.80 for tensile strength ratio.

Resilient modulus ratio (MMR) is calculated as the modulus value after moisture conditioning divided by the modulus of unconditioned specimens. This test is also performed at 77°F (25°C). All of the mixtures, except two, exhibited excellent resilient modulus ratios (see Figure 4.28). The dense-graded mixture produced with fine CRM (DGF) and the CMHB mixture produced with 18% coarse, dry CRM had resilient modulus ratios below the minimum recommended value of 0.80. These mixtures have one thing in common in that they both contain rubber which was added to the mixture as a dry process. However, the other mixtures which employed a dry

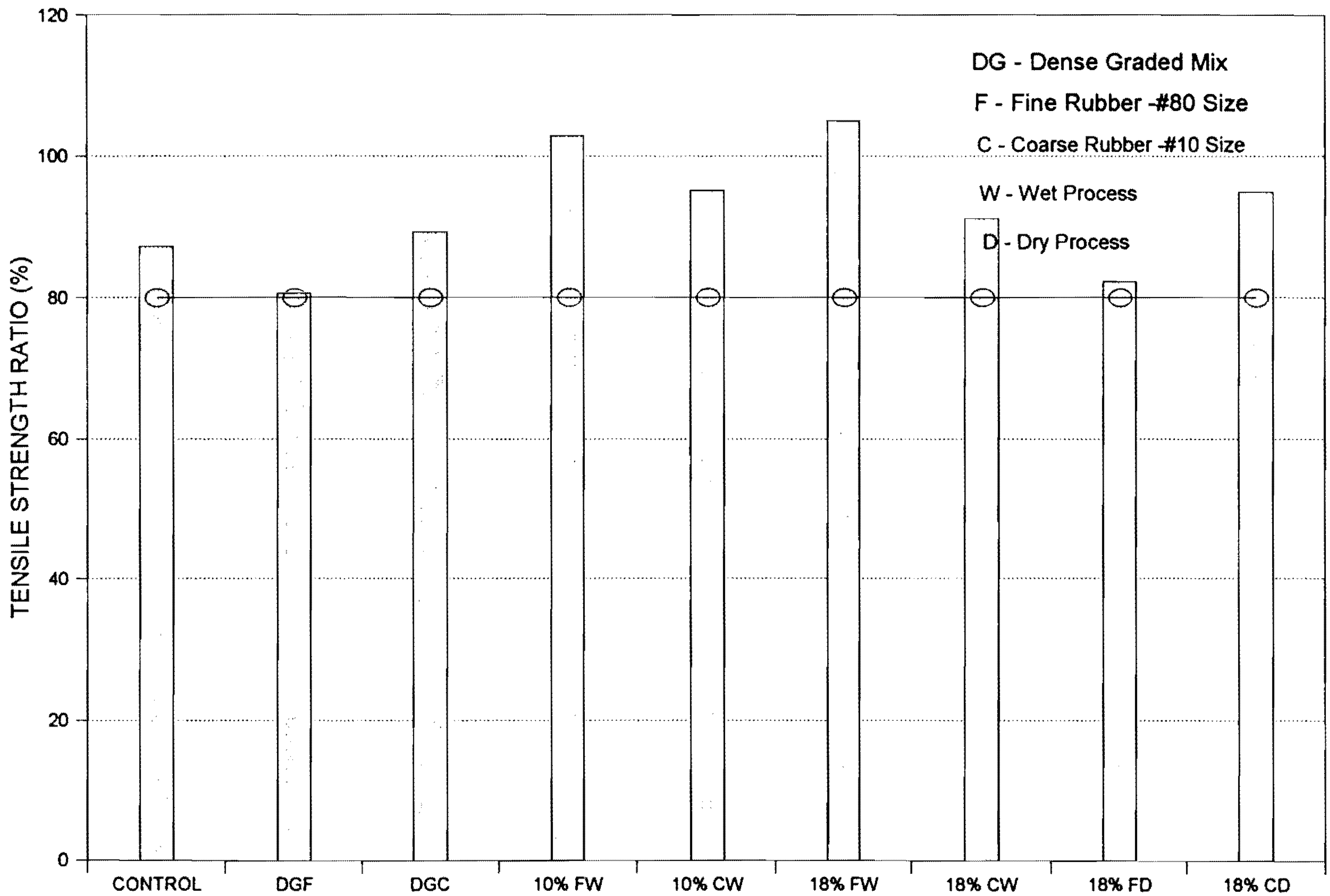


Figure 4.27. Tensile Strength Ratio for Control and CRM Mixtures at 77°F (25°C).

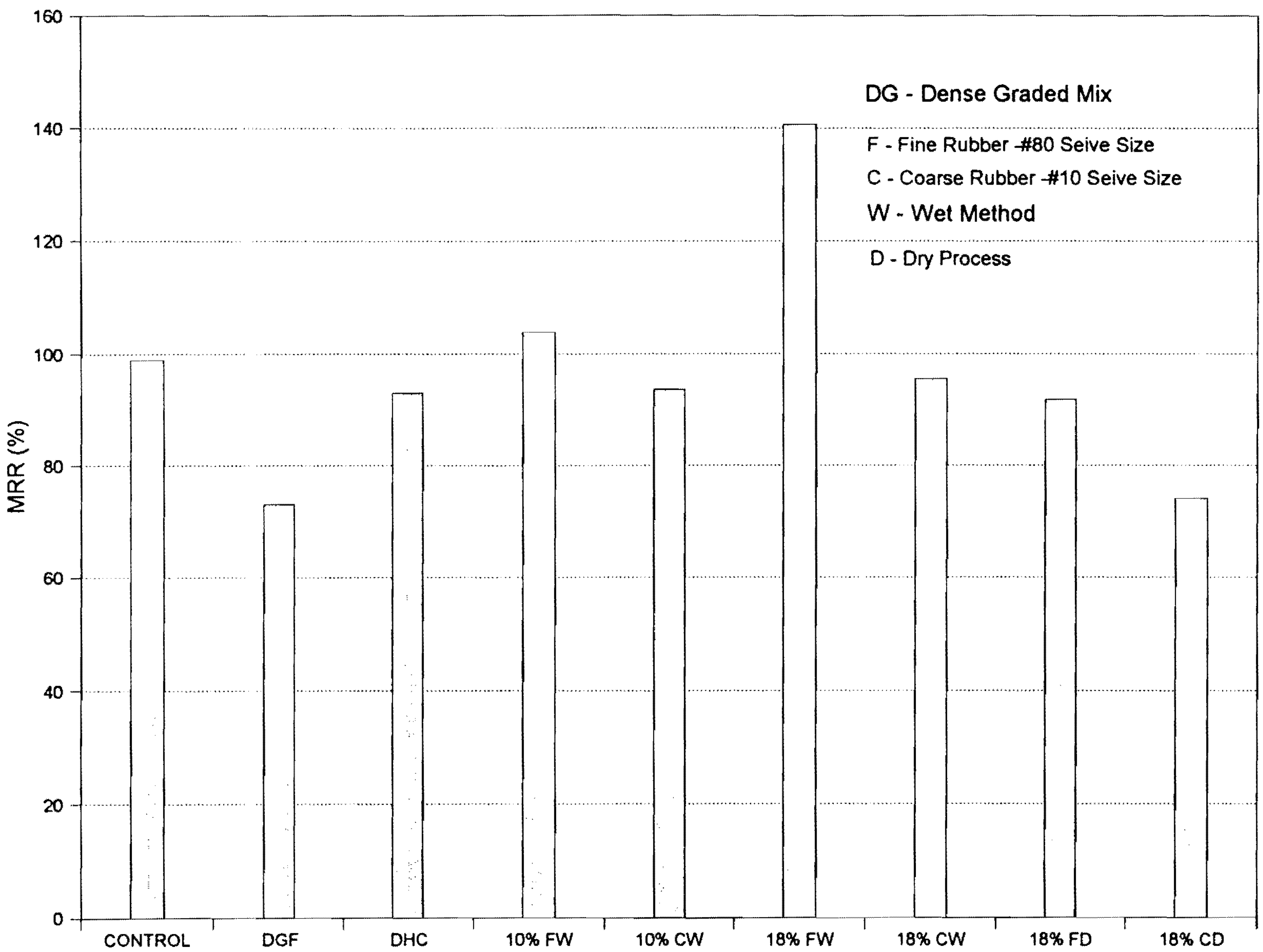


Figure 4.28. Resilient Modulus Ratio for Control and CRM Mixtures at 77°F (25°C).

process (18%FD and DGC) had very good resilient modulus ratios. The mixture produced with 18 percent fine CRM via the wet process (18%FW) had resilient modulus values significantly larger after moisture conditioning. This may be attributed to this particular mixture having a high degree of saturation (70 percent) and the development of pore water pressures during the testing procedure.

4.4 Thermal Cracking

Thermal cracking is considered a nontraffic-associated fracture distress that is common, but not confined, to the northern United States (Von Quintus et al 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, and failure strains. These mixture properties are measured on aged/hardened specimens (environmental aging simulation). The thermal cracking analysis of the laboratory mixtures is not complete at the time of this report but some of the material properties resulting from the laboratory tests are discussed below.

One method of evaluating the potential of a mixture to develop thermal cracking is to calculate the tensile stress induced in the pavement at a specific temperature, $\sigma(T_i)$, caused by a drop in temperature, ΔT. Asphalt concrete has a thermal coefficient of contraction, α_A, of between about 1.0×10^{-5} and 1.8×10^{-5} in/in/°F. This relationship is expressed as:

$$\sigma(T_i) = \alpha_A \, (\Delta T_i) \, \Delta E_{ct}.$$

In this relationship, the creep modulus used in low temperature cracking evaluations is estimated by regression relationships from a creep modulus determined at 3,600 seconds.

The strength of the mixture at low temperatures can be approximated by IDT strength measurements. The AAMAS procedure recommends loading at a rate of 0.05 to 0.065 in/min (0.13 to 0.17 cm/min) when performing tests at low temperatures (41°F (5°C) and below).

Assuming that the thermal coefficient of asphalt concrete and crumb-rubber mixtures are approximately the same, the only material property affecting the thermal stress induced within the pavement is the tensile creep stiffness, E_{ct}.

Table 4.4. summarizes the creep stiffness at 3,600 seconds and the indirect tensile strength data at 41°F (5°C). To provide resistance to thermal cracking, a mixture should have a low tensile creep modulus, but a relatively high tensile strength. The creep modulus of the control mix is significantly higher than all of the crumb rubber modified mixtures (2 to 5 times higher).

The indirect tensile strengths at 41°F (5°C) and at a loading rate of 0.05 in/min (0.13 cm/min) for some of the crumb rubber mixtures were about the same as (in one case higher than) the control mixture. Thus, some of the crumb rubber mixtures (18%FW, DGC, and DGF) appear to offer greater resistance to thermal cracking than the control mixture.

Table 4.4. Tensile Creep Modulus (3,600 sec) and Indirect Tensile Strength at 41°F (5°C) for Control and Crumb Rubber Modified Mixtures.

Mixture Type	Indirect Tensile Creep Modulus, psi (kg/cm^2)	Indirect Tensile Strength, psi (kg/cm^2)
Control	126,910 (8,923)	163.5 (11.5)
Dense-Graded with Fine CRM (DGF)	62,260 (4,378)	143.5 (10.1)
Dense-Graded with Coarse CRM (DGC)	66,482 (4,674)	148.1 (10.4)
10% Fine CRM - Wet Method (10%FW)	20,484 (1,440)	116.3 (8.2)
10% Coarse CRM - Wet Method (10%CW)	22,555 (1,586)	109.0 (7.7)
18% Fine CRM - Wet Method (18%FW)	59,615 (4,195)	190.2 (13.4)
18% Coarse CRM - Wet Method (18%CW)	41,180 (2,895)	93.1 (6.6)
18% Fine CRM - Dry Method (18%FD)	22,306 (1,568)	83.7 (5.9)
18% Coarse CRM - Dry Method (18%CD)	55,136 (3,877)	110.3 (7.8)

4.5 Disintegration

The following is a discussion as described by Von Quintus et al. (1991) concerning disintegration in asphalt mixtures and how AAMAS evaluates this potential distress. "Disintegration is primarily related to environmental and material factors, but the severity of the distress is dependent on the magnitude and number of wheel load applications. Raveling and skid resistance are the two disintegration distresses considered in AAMAS. Increasing the asphalt content in the mix will increase the film thickness and decrease asphalt aging, reducing the severity of raveling. Conversely, this increase in asphalt content will also reduce air voids, which can increase the possibility of flushing (or bleeding) and reduce skid resistance. Thus, both upper and lower bounds on asphalt content exist and must be considered in mixture design to reduce disintegration distresses."

"Raveling is directly related to the adhesion between the asphalt and aggregate. The factors that have an effect on the adhesion property include a combination of asphalt consistency and film thickness, aggregate cleanliness, shape and texture, air void content of the mix, and absorption. Reduced skid resistance in the form of flushing is also related to a combination of these same factors (asphalt consistency and amount, air voids, and aggregate shape and texture)." (Von Quintus et al. 1991)

"Tensile strain at failure is a measure of the bond or adhesion between the aggregate and asphalt. Obviously the greater the bond, the less probability for raveling. A low tensile strength ratio is a measure of moisture damage or loss of bond between the asphalt and aggregate caused by water. Thus, if a surface mixture is susceptible to moisture damage, it is similarly susceptible to raveling. Reducing the air voids will generally reduce moisture damage and asphalt aging. Conversely, for asphaltic concrete mixes to be resistant to reduced skid resistance and flushing at the surface, the mix must contain adequate air voids after traffic densification." (Von Quintus et al. 1991)

"The following summarizes the criteria that can be used as guidelines to evaluate the acceptability of surface mixtures as related to disintegration: (1) air voids at refusal > 3%; (2) indirect tensile strength ratio, TSR > 0.80; (3) bonding loss < 50; and (4) tensile strain at failure > 10 mils/in at 77°F (25°C) and greater than 2.0 mils/in at 41°F (5°C) after accelerated aging.

$$\text{Bonding Loss} = [1 - \epsilon_{ht}/\epsilon_{ho}] \times 100$$

where ϵ_{ht} is the indirect tensile strain at failure measured on specimens that have been temperature conditioned (accelerated aging); and ϵ_{ho} is the indirect tensile strain at failure measured on unconditioned specimens." (Von Quintus et al. 1991)

"Retained bond is simply a value that represents the decrease in tensile strain at failure as a result of age/hardening and/or moisture damage." (Von Quintus et al. 1991) Table 4.5 summarizes some of the mixture properties which are used to evaluate the disintegration potential of the mixtures.

Table 4.5. Material Properties Used to Evaluate Disintegration Potential of Control and Crumb Rubber Modified Mixtures.

Mixture Type	TSR	Failure Strain 77°F, mils/in	Failure Strain 41°F, mils/in	Bond Loss
Control	87.3	9.64*	1.05*	67.1*
Dense-Graded with Fine CRM (DGF)	80.6	8.45*	1.28*	46.4
Dense-Graded with Coarse CRM (DGC)	89.2	10.14	1.25*	58.7*
10% Fine CRM - Wet Method (10%FW)	102.9	17.40	1.91*	53.2*
10% Coarse CRM - Wet Method (10%CW)	95.1	12.17	1.07*	68.0*
18% Fine CRM - Wet Method (18%FW)	105.0	20.95	4.24	62.8*
18% Coarse CRM - Wet Method (18%CW)	91.2	7.85*	1.35*	59.3*
18% Fine CRM - Dry Method (18%FD)	81.2	8.05*	1.80*	41.2
18% Coarse CRM - Dry Method (18%CD)	95.0	9.54*	1.14*	53.1*

* Fails AAMAS criteria.

All of the mixtures have a high tensile strength ratio indicating these mixtures would not be susceptible to moisture damage. After accelerated aging, the failure strains at 77°F (25°C) are acceptable for four mixtures: DGC, 10%FW, 10%CW, 18%FW. All of the mixtures except one (18%FW) fail the tensile strain criteria at 41°F (5°C). The bonding loss represents the loss in tensile strain as a result of accelerated aging. All of the mixtures failed the bonding loss criteria except two: DGF and 18%FD. These two mixtures both contain fine rubber via the dry process.

4.6 Summary of Mixture Evaluation

Nine mixtures were evaluated in the laboratory. Three of these mixtures were designed according to the standard TxDOT (C-14 Bulletin) design procedure. A control (Type D) mixture was designed according to this method and two mixtures incorporating crumb rubber (added dry as part of the aggregate): dense-graded with fine CRM (DGF) and dense graded with coarse CRM (DGC). These two mixtures contain 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% by weight of the aggregate. This would be equivalent to about 10% rubber by weight of the asphalt.

Six of the nine mixtures evaluated in the laboratory were designed according to TxDOT's recently developed mixture design procedure for asphalt-rubber mixtures. These mixtures are CMHB and similar in gradation to a stone mastic-type mixture. These six different mixtures include both wet and dry processes for adding the crumb rubber to the mix. Rubber concentration was varied and for comparison purposes all rubber concentrations are expressed as a percent by weight of the asphalt whether a dry or wet process was used. Another variable included here is the size of the rubber: fine (-80 mesh) or coarse (-10 mesh). These six mixtures are designated as follows:

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

Rutting Potential

Two test procedures were used to evaluate the rutting potential of the laboratory mixtures: static and repeated load creep tests. Data from the static load creep test were compared to AAMAS criteria which was developed to predict rutting potential. Based on AAMAS criteria all of the mixtures except 2 are considered to have low rutting potential (modulus at one hour). The mixtures designated as 10%CW and 18%CW are judged to have a moderate rutting potential. These 2 mixtures are the only 2 containing coarse CRM via the wet process. When asphalt cement and CRM are blended together at elevated temperatures for 1 hour or more (wet process), the rubber particles tend to swell. Therefore, the coarse CRM rubber particles which were incorporated through the wet process may be significantly larger than the coarse rubber particles which were added via the dry process. It is likely that a different aggregate gradation for these two mixtures (such as a larger top-size aggregate) might produce acceptable, rut-resistant mixtures.

The repeated load creep test was performed to more closely simulate wheel loading than the static creep test. Plastic deformation or relative movement among aggregate particles is most effectively produced under dynamic loading conditions as the dynamic effect of each repetition produces some level of relative movement. Based on the total strain at the end of 10,000 cycles the mixtures were categorized as follows (Category 1 mixtures being the best):

Category 1: DGF,
Control;

Category 2: 18%FD,
DGC,
18%FW; and

Category 3: 18%CD,
10%FW,
10%CW, and
18%CW.

All of the creep tests performed thus far in this study (both repeated and static load creep), were performed without confining pressure. The CMHB rubber mixtures analyzed in this study are very similar in gradation to a stone mastic-type mixture. When performing creep tests on these mixtures without confining pressure, these type of mixtures may lack the lateral support that is present during the field. It is believed that because of the aggregate interlock that exists in a dense-graded mixture, unconfined uniaxial creep properties may be better for dense-graded mixtures than for stone mastic-type mixtures. However, field performance may be better for a stone mastic-type mixture. This important factor must be kept in mind when reviewing unconfined uniaxial creep properties for both dense and CMHB mixtures. It is appropriate to compare or rank CMHB mixtures against each other but it may not be appropriate to compare CMHB mixtures to dense-graded mixtures.

Fatigue Cracking Potential

Using the AAMAS criteria for fatigue cracking (which was developed for dense-graded mixtures), most of the crumb rubber modified mixtures (and the control mix) 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 designated as 18%FW had a significantly better fatigue resistance than the other mixtures. This mixture was also better than the AAMAS standard mixture.

Based on indirect tensile test data, one mixture seemed to "stand-out" from all others in terms of having significantly higher failure strains at all three test temperatures: again, 18%FW.

Resilient modulus testing revealed that the addition of CRM in the dense-graded mixtures caused a decrease in the resilient modulus. All of the CMHB crumb rubber mixtures had lower stiffnesses than the dense-graded mixtures at all three temperatures. It appears that CRM may have some propensity for decreasing the mixture's temperature susceptibility. The 18%FW mixture had a markedly improved temperature susceptibility.

Thermal Cracking

Tensile creep and tensile strength properties were used to evaluate thermal cracking potential. To provide resistance to thermal cracking, a mixture should have a low tensile creep modulus, but a relatively high tensile strength. The creep modulus of the control mix is significantly higher than all of the crumb rubber modified mixtures (2 to 5 times higher). The indirect tensile strengths at 41°F (5°C) and at a loading rate of 0.05 in/min (0.13 cm/min) for some of the crumb rubber mixtures were about the same as (in one case higher than) the control mixture. Thus, some of the crumb rubber mixtures (18%FW, DGC, and DGF) appear to offer greater resistance to thermal cracking than the control mixture. A more thorough analysis of thermal cracking potential was not complete at the time of this report.

Moisture Damage

Moisture damage was evaluated using tensile strength ratio (TSR) and resilient modulus ratio (MMR). AAMAS criteria requires that these ratios should exceed a value of 0.80 (for dense-graded mixtures). TSR for all nine mixtures exceeded the minimum requirement of 0.80.

All of the mixtures, except two, exhibited excellent MMRs: DGF and 18%CD. These mixtures have one thing in common in that they both contain rubber which was added to the mixture as a dry process. However, the other mixtures which employed a dry process had very good MMRs.

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Appendix A

Laboratory Data for Control Mix

(Conventional Type D Mix, No Rubber)

MIX DESIGN FOR CONTROL MIX
(STANDARD TEXAS TYPE-D GRADATION)
USING TxDOT STANDARD DESIGN PROCEDURE

Table A1. Standard Texas Type-D Gradation Blended With 10% Field Sand.

Sieve Size	% Passing	% Each	Combination Each %	Cumulative Weight (grams)
Crushed Stone				
1/2"	100.0	0.0	0.0	0.0
3/8"	92.0	8.0	8.0	320.0
#4	60.0	32.0	32.0	1600.0
#10	37.0	23.0	23.0	2520.0
#40	21.0	16.0	16.0	3160.0
#80	9.0	3.5	12.0	3300.0
#200	4.0	2.25	5	3390.0
Passing #200		2.55	4	3492.0
Sand				
#40	100.0	0.0		3492.0
#80	85.0	8.5		3832.0
#200	42.5	2.75		3942.0
Passing #200	14.5	1.45		4000.0

Table A2. **Summary Of Mix Design Data.**

Asphalt Cement Content[1] (%)	Rice Specific Gravity	Bulk Specific Gravity				Air Voids (%)	Hveem Stability			
		Sample #1	Sample #2	Sample #3	Average		Sample #1	Sample #2	Sample #3	Average
4.0	2.432	2.331	2.301	2.303	2.311	5.0	53.8	47.0	50.7	50.7
4.5	2.444	2.337	2.331	2.334	2.334	4.5	56.0	51.5	54.3	53.9
5.0	2.406	2.334	2.332	2.337	2.334	3.0	48.0	50.9	50.8	49.0
5.5	2.387	2.361	2.368	2.359	2.363	1.0	39.1	48.9	54.6	47.2
6.0	2.380	2.369	2.367	2.377	2.371	0.4	36.4	45.9	33.4	40.2
6.5	2.367	2.361	2.367	2.372	2.366	0.0	19.7	22.5	21.8	21.0

[1] - By weight of the Aggregate

Figure A1. Control Mix, Type D.

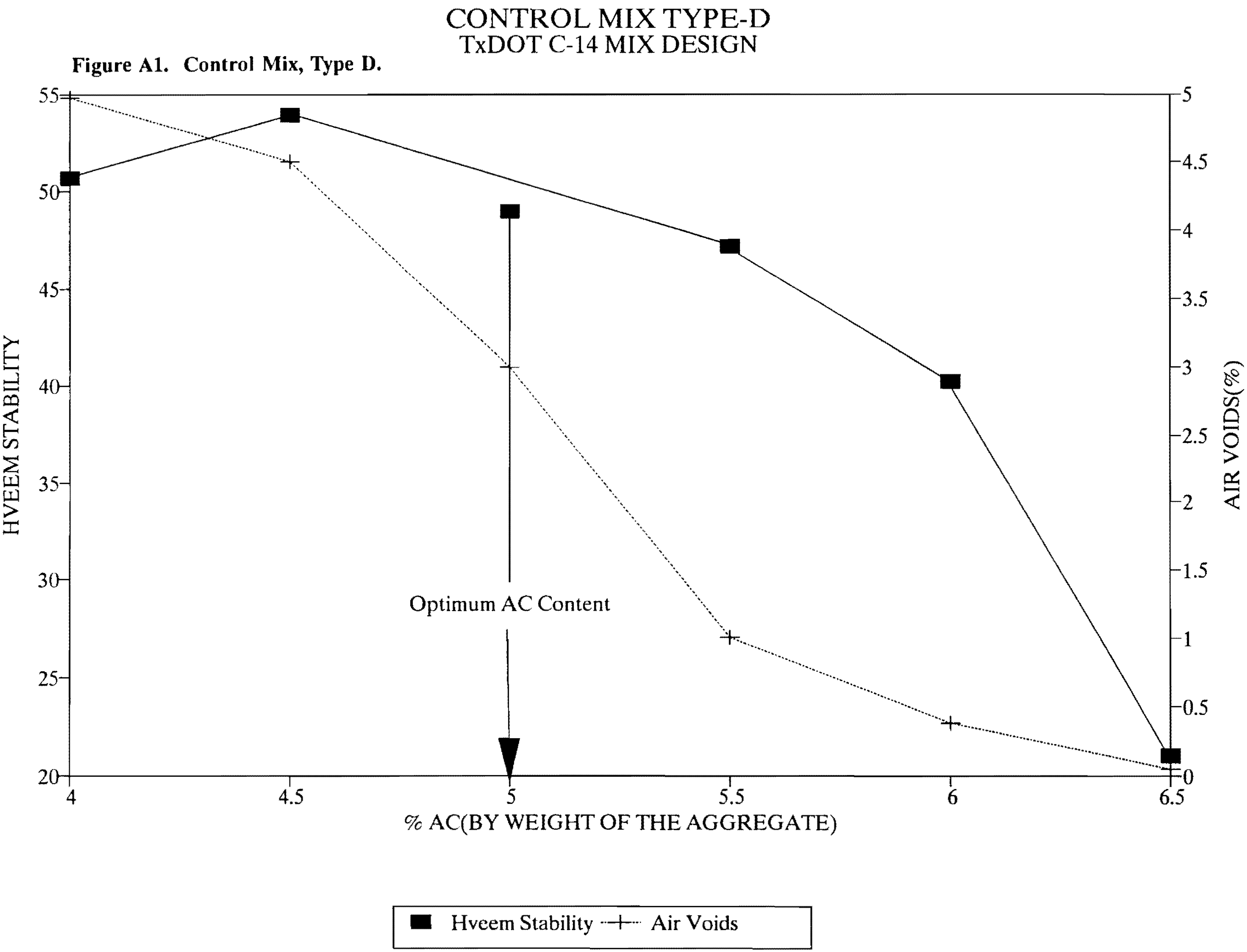


Table A3. Summary Of The Static Creep Test Data For Control Mix.

Sample#	1	2	3	Average
Air Voids %	3.09	3.09	2.9	3.02
AC Content[1] %	5.0	5.0	5.0	5.0
Permanent Strain in/in(cm/cm)	3.17×10^{-4}	3.34×10^{-4}	3.17×10^{-4}	3.2×10^{-4}
Slope in/in sec (cm/cm sec)	4.95×10^{-8}	2.40×10^{-8}	2.60×10^{-8}	3.3×10^{-8}
Creep Stiffness psi (Kg/cm^2)	6986 (491.19)	7285.0 (512.21)	8474.0 (595.81)	7582 (533.1)

[1] - By weight of the aggregate

PERFORMANCE EVALUATION OF THE MIXTURE USING AAMAS

Table A4. AAMAS Test Results For *Unconditioned Specimens @41°F.*

Rice Specific Gravity	2.406			
Sample#	8	11	18	Average
Bulk Specific Gravity	2.273	2.268	2.258	2.266
Air Voids,%	5.53	5.74	6.15	5.80
Total Resilient Modulus[1], psi (Kg/cm^2)	15.21×10^5 (106938)	16.18×10^5 (113783)	16.14×10^5 (113498)	15.84×10^5 (111405)
Indirect Tensile Strength, psi (Kg/cm^2)	121.83 (8.57)	122.72 (8.63)	115.77 (8.14)	120.11 (8.44)
Indirect Tensile Strain @Failure, in/in(cm/cm) (10^{-3})	2.17	4.12	3.29	3.19

[1] - Average of the two Axes

Table A5. AAMAS Test Results For *Unconditioned Specimens @77°F.*

Rice Specific Gravity	2.406			
Sample#	1	2	12	Average
Bulk Specific Gravity	2.263	2.257	2.265	2.262
Air Voids,%	5.94	6.19	5.86	6.00
Total Resilient Modulus[1], psi (Kg/cm^2)	3.76×10^5 (26437)	4.35×10^5 (30596)	3.71×10^5 (26052)	3.94×10^5 (27702)
Indirect Tensile Strength, psi (Kg/cm^2)	98.42 (6.92)	98.28 (6.91)	96.97 (6.82)	97.89 (6.88)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	7.18	5.84	5.49	6.17

[1] - Average of the two Axes

Table A6. AAMAS Test Results For *Unconditioned Specimens @104°F.*

Rice Specific Gravity	2.406			
Sample#	3	4	15	Average
Bulk Specific Gravity	2.275	2.260	2.246	2.260
Air Voids,%	5.44	6.07	6.65	6.05
Total Resilient Modulus[1], psi (Kg/cm^2)	$1.43 x 10^5$ (10057)	$1.43 x 10^5$ (10074)	$1.13 x 10^5$ (7924)	$1.33 x 10^5$ (9351)
Indirect Tensile Strength, psi (Kg/cm^2)	43.44 (3.05)	31.40 (2.21)	30.12 (2.12)	34.99 (2.46)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	8.64	8.10	11.10	9.28

[1] - Average of the two Axes

Table A7. AAMAS Test Results For *Moisture Conditioned* Specimens Tested @77°F.

Rice Specific Gravity	2.406			
Sample#	5	6	10	Average
Bulk Specific Gravity	2.260	2.275	2.250	2.262
Air Voids,%	6.07	5.44	6.48	6.00
Degree Of Saturation, %	58.33	60.07	60.5	59.63
Total Resilient Modulus[1], psi (Kg/cm^2)	$4.07 x 10^5$ (28623)	$3.53 x 10^5$ (24791)	$4.09 x 10^5$ (28739)	$3.90 x 10^5$ (27407)
Indirect Tensile Strength, psi (KG/cm^2)	81.08 (5.70)	90.71 (6.38)	84.54 (5.94)	85.44 (6.01)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	9.31	10.78	8.83	9.64

[1] - Average of the two Axes

Table A8. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-1.**

Rice Specific Gravity	2.406			
Sample#	9	13	16	Average
Bulk Specific Gravity	2.267	2.257	2.249	2.258
Air Voids,%	5.78	6.19	6.53	6.17
Total Resilient Modulus[1], psi (Kg/cm^2)	$23.75x10^5$ (166986)	$23.43x10^5$ (164701)	$24.01x10^5$ (168823)	$23.73x10^5$ (166836)
Recovery Efficiency	0.66	0.60	0.62	0.63
Indirect Tensile Creep Modulus @3600sec, psi(kg/cm^2)	173100 (12171)	75740 (5325)	131880 (9272)	126910 (8923)

[1] - Average of the two Axes

TABLE A9. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-2.**

Rice Specific Gravity	2.406			
Sample#	7	14	17	Average
Bulk Specific Gravity	2.260	2.253	2.260	2.258
Air Voids,%	6.07	6.36	6.07	6.17
Total Resilient Modulus[1], psi (Kg/cm^2)	$22.32x10^5$ (156932)	$23.79x10^5$ (167232)	$24.69x10^5$ (173595)	$23.60x10^5$ (165912)
Indirect Tensile Strength, psi (Kg/cm^2)	158.54 (11.15)	164.67 (11.58)	167.34 (11.77)	163.52 (11.50)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	1.13	0.8	1.22	1.05

[1] - Average of the two Axes

Table A10. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-1.

Rice Specific Gravity	2.406			
Sample#	3	6	5	Average
Bulk Specific Gravity	2.344	2.341	2.311	2.340
Air Voids,%	2.58	2.70	3.12	2.78
Total Uniaxial Resilient Modulus, psi(kg/cm^2)		109400 (7692)	172458 (12126)	140929 (9909)
Slope Of Compressive Creep Test Curve, b	0.13920	0.05592	0.03940	0.07817
Intercept Of Compressive Creep Test Curve, a	0.0011	0.0034	0.0028	0.00243
Total Permanent Deformation @3600 seconds, in/in(cm/cm)	0.00332	0.00388	0.00528	0.00415
Compressive Creep Modulus @3600sec, psi(kg/cm^2)	17874 (1257)	15329 (1078)	11061 (778)	14755 (1037)

Table A11. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-2.

Rice Specific Gravity	2.406			
Sample#	4	7	8	Average
Bulk Specific Gravity	2.342	2.330	2.337	2.340
Air Voids,%	2.66	3.16	2.87	2.91
Unconfined Compressive Strength, psi (Kg/cm^2)	508.6 (35.8)	444.5 (31.3)	472.9 (33.3)	475.3 (33.4)
Compressive Strain @Failure, in/in (cm/cm) (10^{-3})	18.44	20.3	27.5	22.08

[1] - Average of the two Axes

Table A12. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-3.

Rice Specific Gravity	2.406		
Sample#	1	2	Average
Bulk Specific Gravity	2.353	2.335	2.344
Air Voids,%	2.21	2.96	2.58
Dynamic Resilient Modulus @200th cycle, psi(kg/cm^2)	161300 (11341)	173900 (12227)	140929 (9909)
Slope Of Repetitive Creep Test Curve, b	0.31634	0.52966	0.42300
Intercept Of Repetitive Creep Test Curve, a	0.00079	0.00016	0.00048
Total Permanent Deformation @10000 cycles, in/in(cm/cm)	0.007025	0.007132	0.007079

Appendix B

Laboratory Data for DGF Mixture

(Dense-Graded, Type D Mixture with Fine Rubber)

MIX DESIGN FOR OPTIMUM FINE CRM (-#80 SIEVE SIZE) IN CONTROL MIX (STANDARD TEXAS TYPE-D GRADATION) USING STANDARD TxDOT DESIGN PROCEDURE

STEP 1: The objective in this section is to determine the optimum rubber content with minimum changes in aggregate gradation according to TxDOT's standard design procedure.

Table B1. Standard Texas Type-D Gradation Blended With 10% Field Sand and *0.2%*[1] *Fine (-80) CRM.*

Sieve Size	% Passing	% Each	Combination Each %	Cumulative Weight (grams)
Crushed Stone				
1/2"	100.0	0.0	0.0	0.0
3/8"	92.0	8.0	8.0	320.0
#4	60.0	32.0	32.0	1600.0
#10	37.0	23.0	23.0	2520.0
#40	21.0	16.0	16.0	3160.0
#80	9.0	3.5	12.0	3300.0
#200	4.0	2.25	5	3390.0
Passing #200		2.55	4	3492.0
Sand				
#40	100.0	0.0		3492.0
#80	85.0	8.5		3832.0
#200	42.5	2.75		3921.0[2]
Passing #200	14.5	1.45		3979.0

[1] - By weight of the aggregate.

[2] - Since the maximum nominal size of the rubber is #80 size equivalent volume of the sand is replaced on #200 sieve size.

Table B2. Summary Of Mix Design Data For 0.2% Fine (-80) CRM By Weight of Aggregate.

Asphalt Cement Content[1] (%)	Rice Specific Gravity	Bulk Specific Gravity				Air Voids (%)	Hveem Stability			
		Sample #1	Sample #2	Sample #3	Average		Sample #1	Sample #2	Sample #3	Average
4.0	2.466	2.274	2.277	2.277	2.276	7.7	52.3	48.1	45.9	47.8
5.0	2.401	2.322	2.322	2.317	2.320	3.4	52.3	48.5	51.0	50.6
6.0	2.376	2.342	2.339	2.347	2.342	1.4	42.5	45.6	43.4	43.8
7.0	2.347	2.345	2.351	2.347	2.348	0.0	18.4	21.1	21.2	20.2

[1] - By weight of the Aggregate

Figure B1. Type D Mix Design with 0.2% Fine CRM.

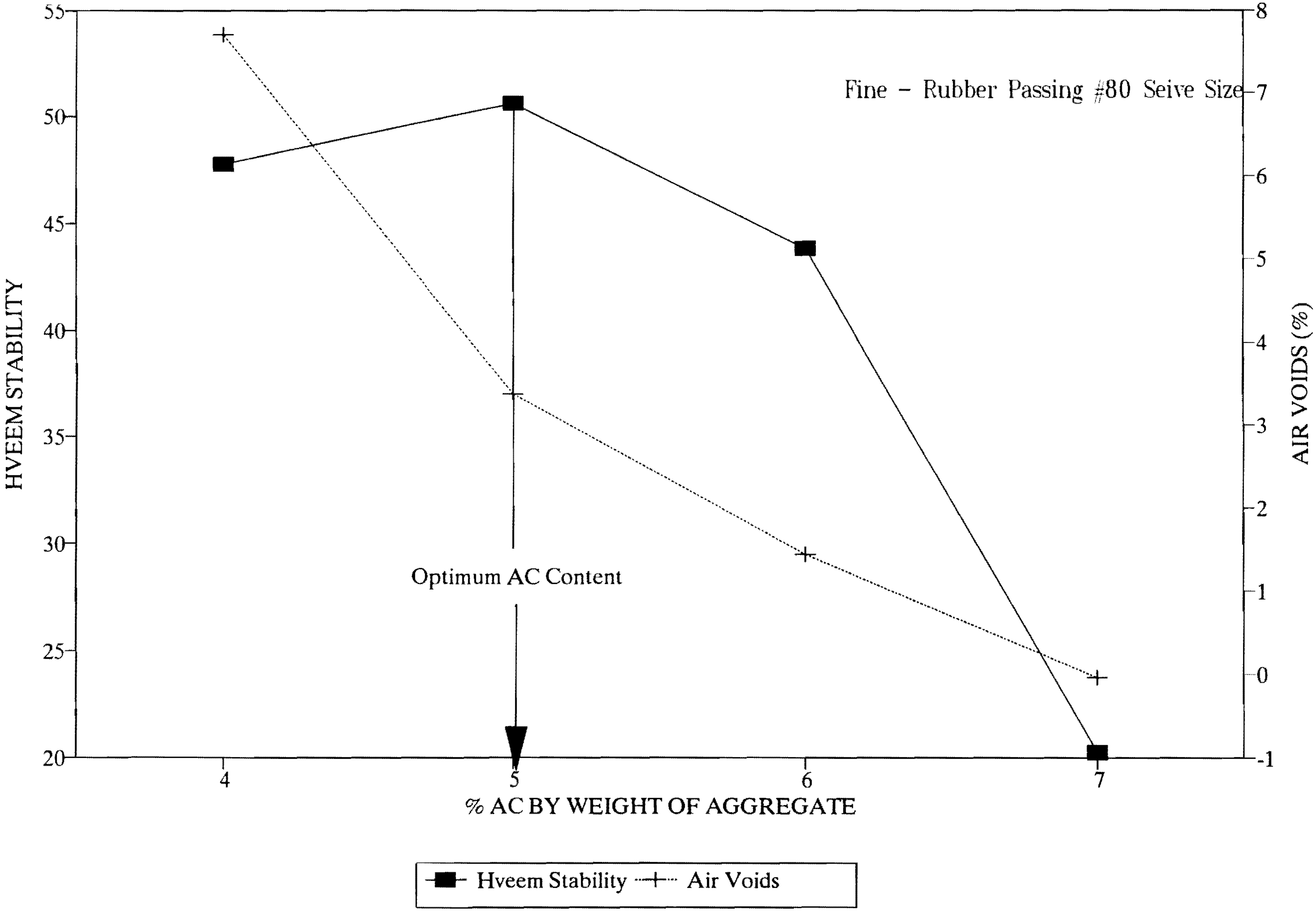


Table B3. Standard Texas Type-D Gradation Blended With 10% Field Sand and *0.8%*[1] *Fine (-80) CRM.*

Sieve Size / Crushed Stone	% Passing	% Each	Combination Each %	Cumulative Weight (grams)
1/2"	100.0	0.0	0.0	0.0
3/8"	92.0	8.0	8.0	320.0
#4	60.0	32.0	32.0	1600.0
#10	37.0	23.0	23.0	2520.0
#40	21.0	16.0	16.0	3160.0
#80	9.0	3.5	12.0	3300.0
#200	4.0	2.25	5	3390.0
Passing #200		2.55	4	3492.0
Sand				
#40	100.0	0.0		3492.0
#80	85.0	8.5		3832.0
#200	42.5	2.75		3858.0[2]
Passing #200	14.5	1.45		3916.0

1 - By weight of the aggregate.
2 - Since the maximum nominal size of the rubber is #80 size equivalent volume of the sand is replaced on #200 sieve size.

Table B4. Summary Of Mix Design Data For *0.8% Fine (-80) CRM* By Weight of Aggregate.

Asphalt Cement Content[1] (%)	Rice Specific Gravity	Bulk Specific Gravity				Air Voids (%)	Hveem Stability			
		Sample #1	Sample #2	Sample #3	Average		Sample #1	Sample #2	Sample #3	Average
4.5	2.434	2.254	2.266	2.258	2.259	7.2	36.8	41.2	39.6	39.2
5.0	2.402	2.298	2.303	2.301	2.301	4.2	37.5	35.4	34.3	35.7
5.5	2.383	2.323	2.312	2.312	2.316	2.8	32.2	28.2	35.4	31.9
6.0	2.359	2.309	2.318	2.318	2.315	1.9	25.0	29.7	31.0	28.6

[1] - By weight of the Aggregate

Figure B2. Type D Mix Design with 0.8% Fine CRM.

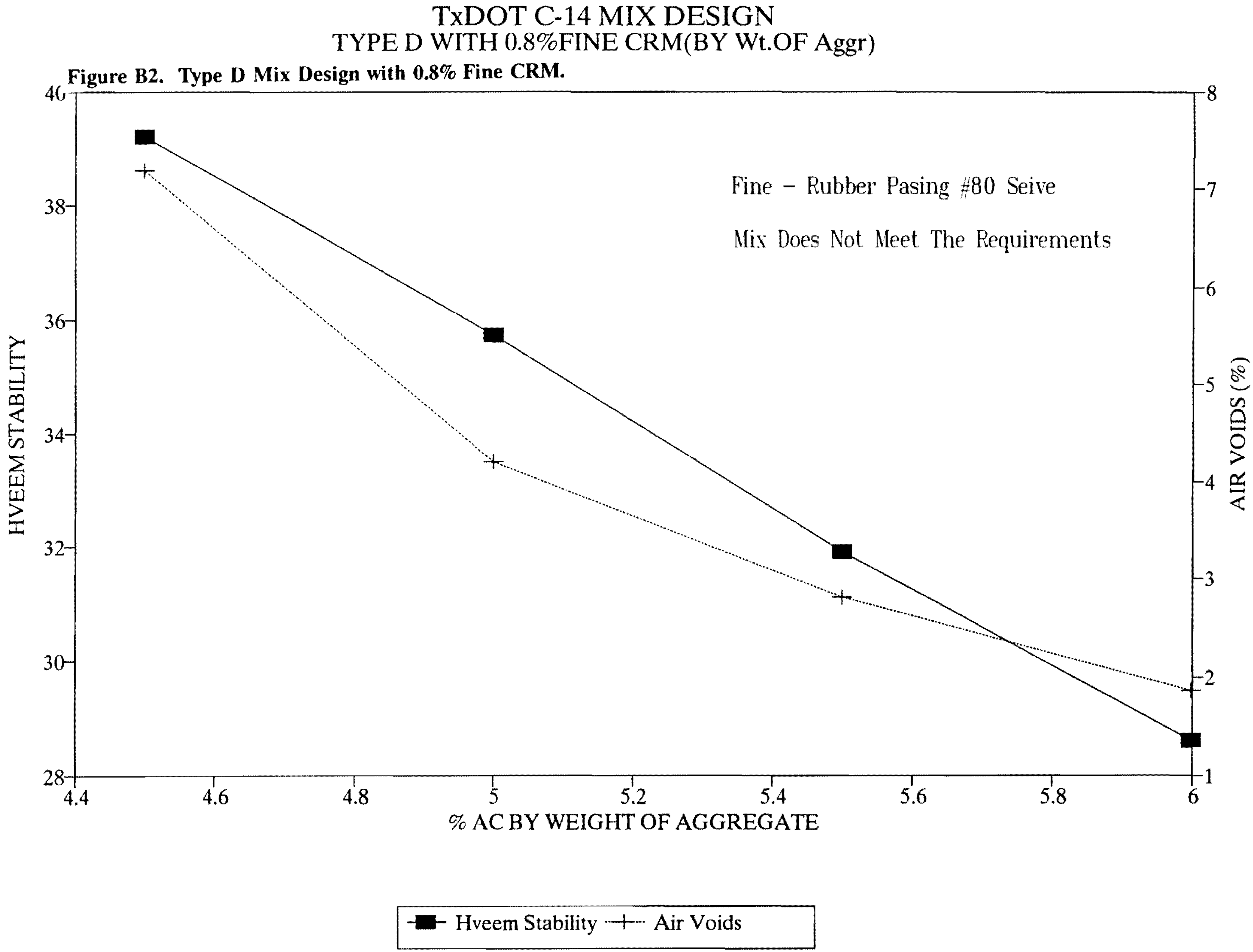


Table B5. **Standard Texas Type-D Gradation Blended With 10% Field Sand and *0.5%*[1] *Fine (-80) CRM.***

Sieve Size / Crushed Stone	% Passing	% Each	Combination Each %	Cumulative Weight (grams)
1/2"	100.0	0.0	0.0	0.0
3/8"	92.0	8.0	8.0	320.0
#4	60.0	32.0	32.0	1600.0
#10	37.0	23.0	23.0	2520.0
#40	21.0	16.0	16.0	3160.0
#80	9.0	3.5	12.0	3300.0
#200	4.0	2.25	5	3390.0
Passing #200		2.55	4	3492.0
Sand				
#40	100.0	0.0		3492.0
#80	85.0	8.5		3832.0
#200	42.5	2.75		3889.5[2]
Passing #200	14.5	1.45		3947.5

[1] - By weight of the aggregate.
[2] - Since the maximum nominal size of the rubber is #80 size equivalent volume of the sand is replaced on #200 sieve size.

Table B6. Summary Of Mix Design Data For *0.5% Fine (-80) CRM* By Weight of Aggregate.

Asphalt Cement Content[1] (%)	Rice Specific Gravity	Bulk Specific Gravity				Air Voids (%)	Hveem Stability			
		Sample #1	Sample #2	Sample #3	Average		Sample #1	Sample #2	Sample #3	Average
4.0	2.392	2.272	2.283	2.290	2.282	4.6	40.3	39.7	39.5	39.8
5.0	2.385	2.311	2.307	2.301	2.306	3.3	36.5	38.1	40.4	38.3
5.5	2.372	2.324	2.324	2.319	2.322	2.2	33.0	33.7	33.6	33.4
6.0	2.360	2.333	2.331	2.337	2.334	1.1	32.1	33.4	31.6	32.4

[1] - By weight of the Aggregate

TxDOT C-14 MIX DESIGN
TYPE D WITH 0.5%FINE CRM(BY Wt.OF Aggr)

Figure B3. Type D Mix Design with 0.5% Fine CRM.

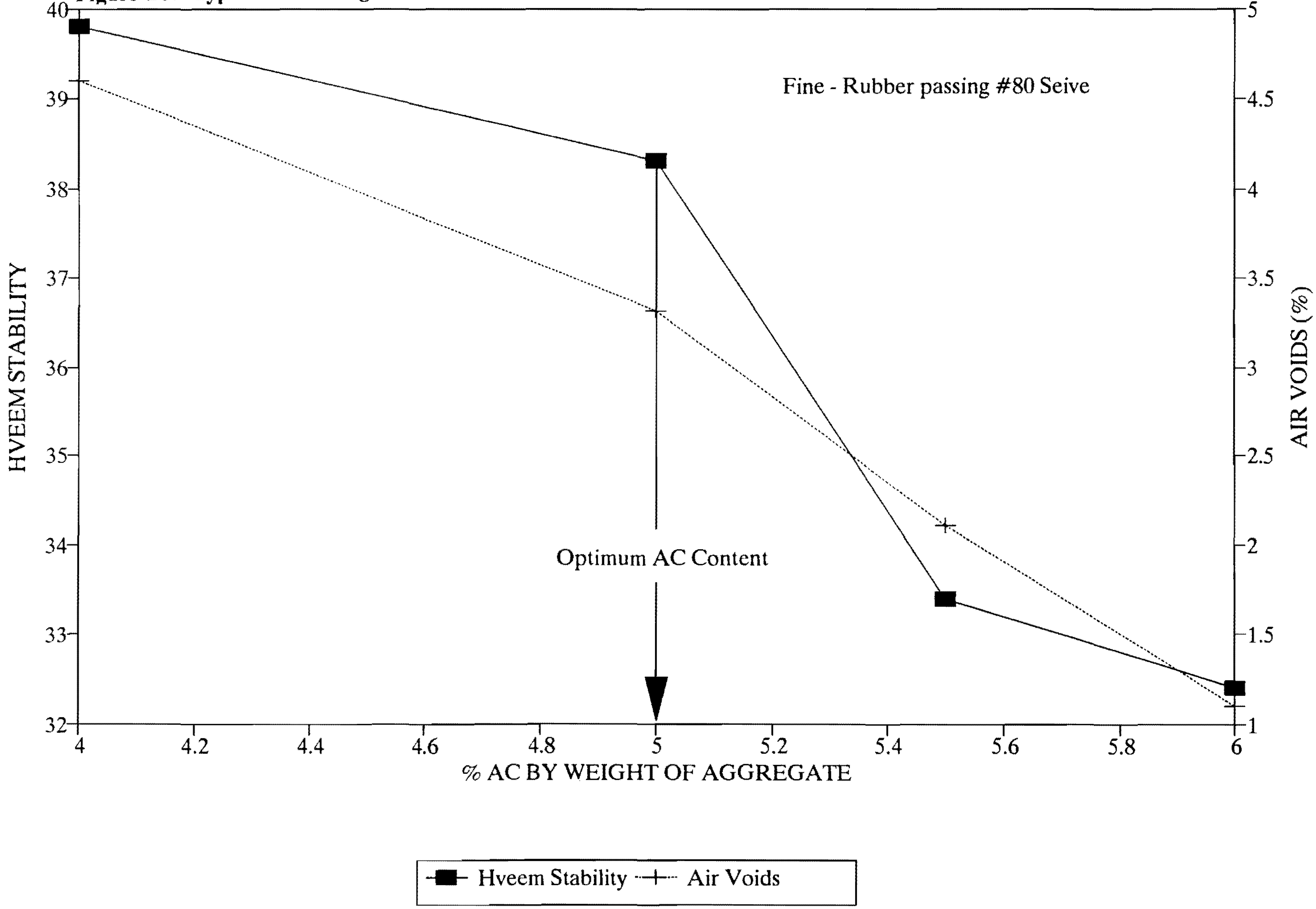


Table B7. Standard Texas Type-D Gradation Blended With 10% Field Sand and *0.7%*[1] *Fine (-80) CRM.*

Sieve Size / Crushed Stone	% Passing	% Each	Combination Each %	Cumulative Weight (grams)
1/2"	100.0	0.0	0.0	0.0
3/8"	92.0	8.0	8.0	320.0
#4	60.0	32.0	32.0	1600.0
#10	37.0	23.0	23.0	2520.0
#40	21.0	16.0	16.0	3160.0
#80	9.0	3.5	12.0	3300.0
#200	4.0	2.25	5	3390.0
Passing #200		2.55	4	3492.0
Sand				
#40	100.0	0.0		3492.0
#80	85.0	8.5		3832.0
#200	42.5	2.75		3868.5[2]
Passing #200	14.5	1.45		3926.5

1 - By weight of the aggregate.
2 - Since the maximum nominal size of the rubber is #80 size equivalent volume of the sand is replaced on #200 sieve size.

Table B8: Summary Of Mix Design Data For *0.7% Fine (-80) CRM* By Weight of Aggregate.

Asphalt Content	5.25% (By weight of the aggregate)			
Rice Sp.Gravity	2.385			
Sample #	1	2	3	Average
Bulk Sp.Gravity	2.304	2.313	2.320	2.312
Air Voids,%	3.4	3.0	2.7	3.0
Hveem Stability	33.5	32.2	32.2	32.6

Table B9. Summary Of The Static Creep Test Data For Control Mix With Optimum Fine (-80) CRM Content (0.5% By Weight Of The Aggregate).

Sample#	1	2	3	Average
Air Voids %	3.1	3.3	3.5	3.3
AC Content[1] %	5.0	5.0	5.0	5.0
Permanent Strain in/in(cm/cm)	9.82×10^{-4}	10.2×10^{-4}	7.12×10^{-4}	9.05×10^{-4}
Slope in/in sec (cm/cm sec)	7.9×10^{-8}	6.7×10^{-8}	5.8×10^{-8}	6.8×10^{-8}
Creep Stiffness psi (Kg/cm^2)	4210 (296)	4091 (288)	4660 (328)	4320 (304)

[1] - By weight of the aggregate

PERFORMANCE EVALUATION OF THE MIXTURE USING AAMAS

Table B10. AAMAS Test Results For *Unconditioned Specimens @41°F.*

Rice Specific Gravity	2.385			
Sample#	12	14	18	Average
Bulk Specific Gravity	2.238	2.256	2.259	2.251
Air Voids,%	6.17	5.42	5.30	5.63
Total Resilient Modulus[1], psi (Kg/cm^2)	14.12×10^5 (99267)	14.57×10^5 (102437)	16.10×10^5 (113177)	14.93×10^5 (104960)
Indirect Tensile Strength, psi (Kg/cm^2)	106.80 (7.51)	117.57 (8.27)	122.91 (8.64)	115.76 (8.14)
Indirect Tensile Strain @Failure, in/in(cm/cm) (10^{-3})	2.13	2.67	2.38	2.39

[1] - Average of the two Axes

Table B11. AAMAS Test Results For *Unconditioned Specimens @77°F.*

Rice Specific Gravity	2.385			
Sample#	9	11	15	Average
Bulk Specific Gravity	2.253	2.248	2.257	2.253
Air Voids,%	5.52	5.73	5.38	5.54
Total Resilient Modulus[1], psi (Kg/cm^2)	3.50×10^5 (24609)	3.86×10^5 (27174)	3.86×10^5 (27112)	3.74×10^5 (26298)
Indirect Tensile Strength, psi (Kg/cm^2)	93.79 (6.59)	100.48 (7.08)	103.30 (7.27)	99.19 (6.97)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	5.62	5.63	5.63	5.63

[1] - Average of the two Axes

Table B12. AAMAS Test Results For *Unconditioned Specimens @104°F.*

Rice Specific Gravity	2.385			
Sample#	1	3	6	Average
Bulk Specific Gravity	2.261	2.241	2.236	2.246
Air Voids,%	5.20	6.02	6.24	5.82
Total Resilient Modulus[1], psi (Kg/cm^2)	$1.48x10^5$ (10373)	$1.62x10^5$ (11395)	$1.41x10^5$ (9913)	$1.50x10^5$ (10560)
Indirect Tensile Strength, psi (Kg/cm^2)	38.13 (2.74)	32.75 (2.31)	36.93 (2.61)	35.93 (2.53)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	7.45	8.54	7.53	7.84

[1] - Average of the two Axes

Table B13. AAMAS Test Results For *Moisture Conditioned* Specimens Tested @77°F.

Rice Specific Gravity	2.385			
Sample#	5	7	17	Average
Bulk Specific Gravity	2.245	2.265	2.233	2.248
Air Voids,%	5.88	5.01	6.39	5.76
Degree Of Saturation, %	48.27	47.64	47.12	47.68
Total Resilient Modulus[1], psi (Kg/cm^2)	$2.98x10^5$ (20953)	$2.85x10^5$ (20059)	$2.34x10^5$ (16480)	$2.73x10^5$ (19164)
Indirect Tensile Strength, psi (KG/cm^2)	77.04 (5.41)	82.29 (5.79)	69.15 (4.86)	76.16 (5.35)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	7.84	9.64	7.88	8.45

[1] - Average of the two Axes

Table B14. AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-1.

Rice Specific Gravity	2.385			
Sample#	8	10	13	Average
Bulk Specific Gravity	2.262	2.247	2.251	2.258
Air Voids,%	5.78	6.19	6.53	6.17
Total Resilient Modulus[1], psi (Kg/cm^2)	$20.06x10^5$ (141032)	$18.58x10^5$ (130610)	$18.47x10^5$ (129840)	$19.03x10^5$ (133827)
Recovery Efficiency	0.82	0.53	0.60	0.65
Indirect Tensile Creep Modulus @3600sec, psi(kg/cm^2)	86271 (6066)	51718 (3636)	48792 (3431)	62260 (4378)

[1] - Average of the two Axes

Table B15. AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-2.

Rice Specific Gravity	2.385			
Sample#	2	4	16	Average
Bulk Specific Gravity	2.259	2.243	2.238	2.247
Air Voids,%	5.2	5.96	6.15	5.77
Total Resilient Modulus[1], psi (Kg/cm^2)	$21.68x10^5$ (152452)	$19.95x10^5$ (140301)	$19.37x10^5$ (136181)	$20.34x10^5$ (142978)
Indirect Tensile Strength, psi (Kg/cm^2)	142.51 (10.02)	141.78 (9.97)	146.52 (10.3)	143.47 (10.09)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	1.14	1.28	1.41	1.28

[1] - Average of the two Axes

Table B16. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-1.

Rice Specific Gravity	2.385			
Sample#	3	4	8	Average
Bulk Specific Gravity	2.33	2.34	2.34	2.336
Air Voids,%	2.32	1.90	1.89	2.05
Total Resilient Modulus, psi(kg/cm^2)	145345 (10219)	165152 (11612)	185531 (13045)	165343 (11625)
Slope Of Compressive Creep Test Curve, b	0.04268	0.00042	0.04741	0.03017
Intercept Of Compressive Creep Test Curve, a	0.00297	0.00213	0.00209	0.00240
Total Compressive Strain @3600sec, in/in(cm/cm)	0.00419	0.00215	0.00308	0.00315
Compressive Creep Modulus @3600sec, psi(kg/cm^2)	14274 (1004)	27759 (1952)	19348 (1360)	20460 (1439)

Table B17. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-2.

Rice Specific Gravity	2.385		
Sample#	2	7	Average
Bulk Specific Gravity	2.338	2.335	2.337
Air Voids,%	1.97	2.1	2.03
Unconfined Compressive Strength, psi (Kg/cm^2)	418.5 (29.4)	377.3 (26.5)	397.9 (28.0)
Compressive Strain @Failure, in/in (cm/cm) (10^{-3})	14.4	26.1	20.25

Table B18. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-3.

Rice Specific Gravity	2.385		
Sample#	5	6	Average
Bulk Specific Gravity	2.315	2.323	2.319
Air Voids,%	2.94	2.61	2.77
Dynamic Resilient Modulus @200thcycle, psi(kg/cm^2)	261800 (18407)	249300 (17528)	255550 (17968)
Slope Of Repetitive Creep Test Curve, b	0.29566	0.42544	0.36055
Intercept Of Repetitive Creep Test Curve, a	0.00607	0.00143	0.00375
Total Permanent Deformation @10000 cycles, in/in(cm/cm)	0.00452	0.00377	0.00415

Appendix C

Laboratory Data for DGC Mixture

(Type D, Dense Grade Mixture with Coarse Rubber)

MIX DESIGN FOR OPTIMUM COARSE RUBBER (-#10 SIEVE SIZE) CONTENT (STANDARD TEXAS TYPE-D GRADATION) USING TxDOT STANDARD DESIGN PROCEDURE

STEP 1: The objective in this section is to determine the optimum rubber content with minimum changes in aggregate gradation according to the standard design procedure.

Table C1. Standard Texas Type-D Gradation Blended With 10% Field Sand and *0.2%*[1] *Coarse (-10) CRM.*

Sieve Size	% Passing	% Each	Combination Each %	Cumulative Weight (grams)
Crushed Stone				
1/2"	100.0	0.0	0.0	0.0
3/8"	92.0	8.0	8.0	320.0
#4	60.0	32.0	32.0	1600.0
#10	37.0	23.0	23.0	2520.0
#40	21.0	16.0	16.0	3160.0
#80	9.0	3.5	12.0	3300.0
#200	4.0	2.25	5	3390.0
Passing #200		2.55	4	3492.0
Sand				
#40	100.0	0.0		3492.0
#80	85.0	8.5		3811.0[2]
#200	42.5	2.75		3921.0
Passing #200	14.5	1.45		3979.0

1 - By weight of the aggregate.
2 - Since the maximum nominal size of the rubber is #10 size equivalent volume of the sand is replaced on #80 sieve size.

Table C2. Summary Of Mix Design Data For 0.2% Coarse (-10) CRM By Weight of Aggregate.

Asphalt Cement Content[1] (%)	Rice Specific Gravity	Bulk Specific Gravity				Air Voids (%)	Hveem Stability			
		Sample #1	Sample #2	Sample #3	Average		Sample #1	Sample #2	Sample #3	Average
4.5	2.478	2.299	2.300	2.301	2.300	7.2	47.6	46.3	48.5	47.5
4.75	2.454	2.280	2.296	2.310	2.295	6.5	46.6	46.9	47.4	47.0
5.0	2.414	2.330	2.346	2.338	2.338	3.1	48.4	45.3	46.1	46.6
5.5	2.389	2.350	2.359	2.34	2.350	1.6	40.5	43.5	43.1	42.4
6.0	2.384	2.355	2.353	2.358	2.355	1.2	34.5	30.2	34.7	34.5

[1] - By weight of the Aggregate

TxDOT C-14 MIX DESIGN
TYPE-D WITH 0.2%COARSE CRM(BY Wt.Aggr)

Figure C1. Type D Mix Design with 0.2% Coarse CRM.

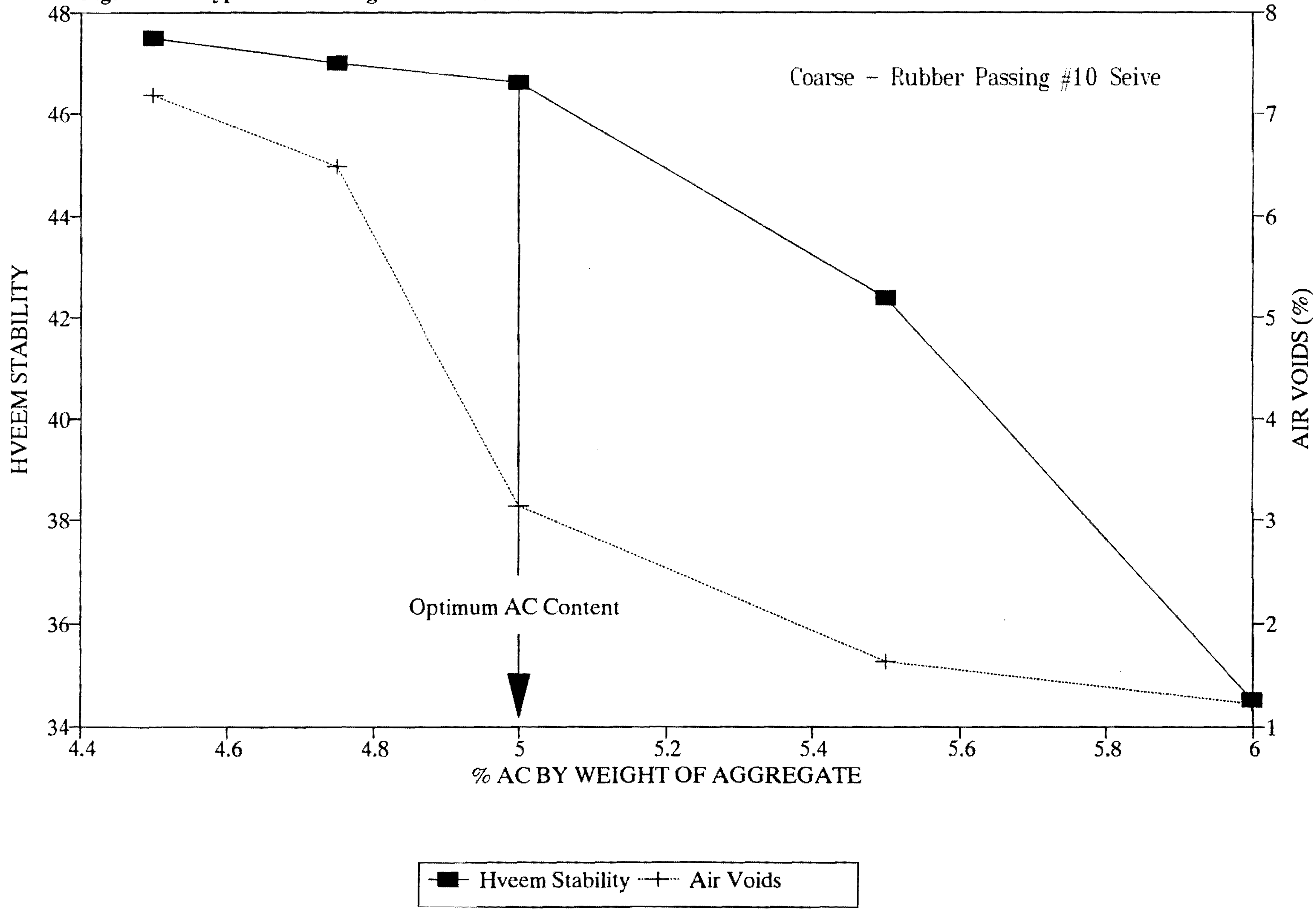


Table C3. Standard Texas Type-D Gradation Blended With 10% Field Sand and *0.8%*[1] *Coarse (-10) CRM.*

Sieve Size / Crushed Stone	% Passing	% Each	Combination Each %	Cumulative Weight (grams)
1/2"	100.0	0.0	0.0	0.0
3/8"	92.0	8.0	8.0	320.0
#4	60.0	32.0	32.0	1600.0
#10	37.0	23.0	23.0	2520.0
#40	21.0	16.0	16.0	3160.0
#80	9.0	3.5	12.0	3300.0
#200	4.0	2.25	5	3390.0
Passing #200		2.55	4	3492.0
Sand				
#40	100.0	0.0		3492.0
#80	85.0	8.5		3748.0[2]
#200	42.5	2.75		3858.0
Passing #200	14.5	1.45		3916.0

1 - By weight of the aggregate.
2 - Since the maximum nominal size of the rubber is #10 size equivalent volume of the sand is replaced on #80 sieve size.

Table C4. Summary Of Mix Design Data For *0.8% Coarse (-10) CRM* By Weight of Aggregate.

Asphalt Cement Content[1] (%)	Rice Specific Gravity	Bulk Specific Gravity				Air Voids (%)	Hveem Stability			
		Sample #1	Sample #2	Sample #3	Average		Sample #1	Sample #2	Sample #3	Average
4.5	2.415	2.208	2.214	2.227	2.216	8.2	33.5	34.3	34.9	34.2
5.0	2.396	2.249	2.239	2.251	2.246	6.3	32.2	27.7	35.1	31.7
5.5	2.372	2.265	2.248	2.270	2.261	4.7	31.6	28.1	31.5	30.4
6.0	2.354	2.299	2.273	2.301	2.291	2.7	25.1	23.5	25.5	24.7

[1] - By weight of the Aggregate

Figure C2. Type D Mix Design with 0.8% Coarse CRM.

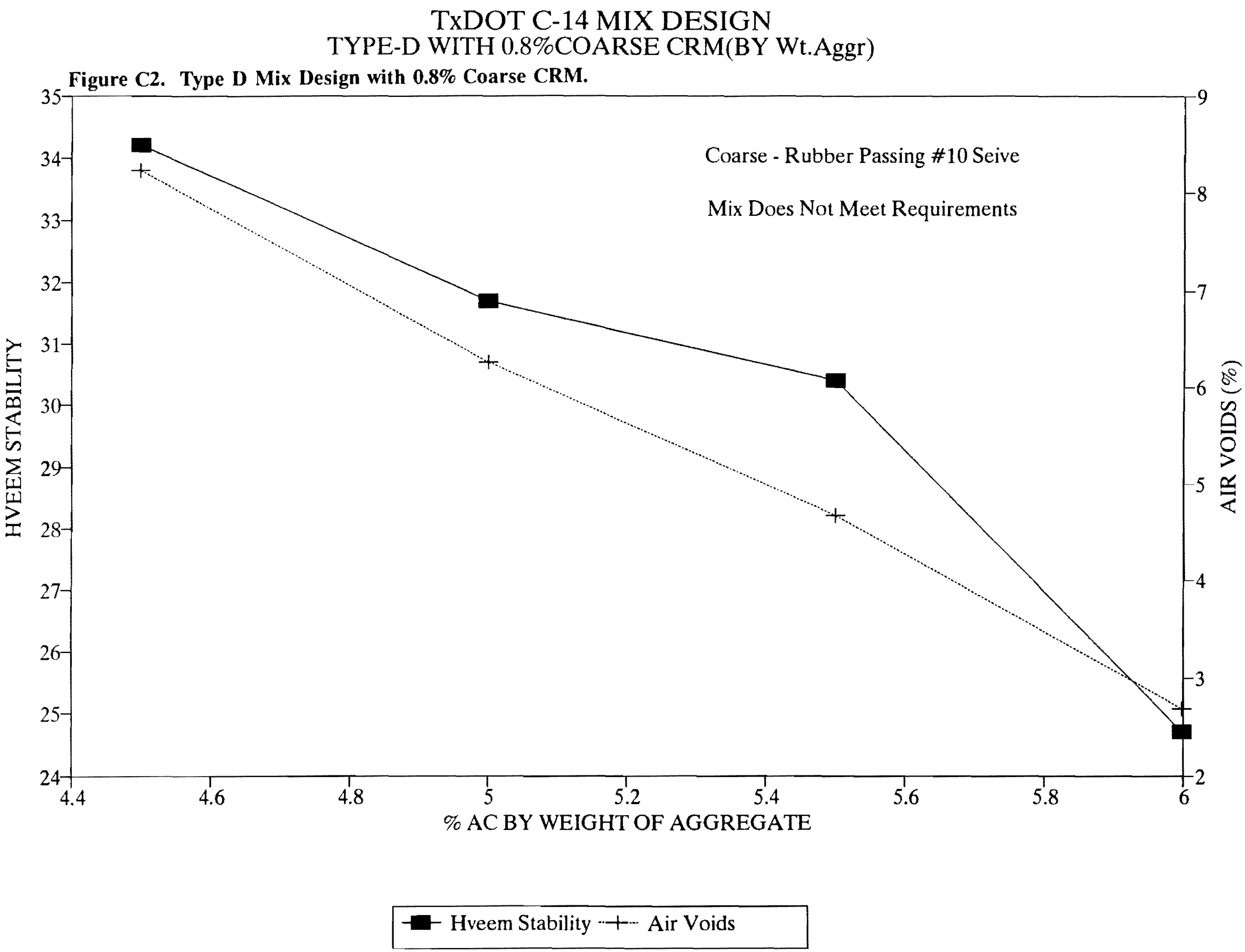


Table C5. Standard Texas Type-D Gradation Blended With 10% Field Sand and *0.5%*[1] *Coarse (-10) CRM.*

Sieve Size	% Passing	% Each	Combination Each %	Cumulative Weight (grams)
Crushed Stone				
1/2"	100.0	0.0	0.0	0.0
3/8"	92.0	8.0	8.0	320.0
#4	60.0	32.0	32.0	1600.0
#10	37.0	23.0	23.0	2520.0
#40	21.0	16.0	16.0	3160.0
#80	9.0	3.5	12.0	3300.0
#200	4.0	2.25	5	3390.0
Passing #200		2.55	4	3492.0
Sand				
#40	100.0	0.0		3492.0
#80	85.0	8.5		3779.5[2]
#200	42.5	2.75		3889.5
Passing #200	14.5	1.45		3947.5

[1] - By weight of the aggregate.
[2] - Since the maximum nominal size of the rubber is #80 size equivalent volume of the sand is replaced on #200 sieve size.

Table C6. Summary Of The Mix Design Data For *0.5% Coarse (-10) CRM* By Weight of Aggregate.

Asphalt Cement Content[1] (%)	Rice Specific Gravity	Bulk Specific Gravity				Air Voids (%)	Hveem Stability			
		Sample #1	Sample #2	Sample #3	Average		Sample #1	Sample #2	Sample #3	Average
4.0	2.411	2.277	2.283	2.278	2.279	5.5	39.8	37.5	36.2	37.8
5.0	2.401	2.281	2.278	2.287	2.282	4.9	34.8	42.2	54.3	43.8
5.75	2.372	2.304	2.310	2.281	2.298	3.1	46.8	48.5	51.4	48.9
6.0	2.362	2.303	2.309	2.307	2.306	2.4	41.5	38.3	42.1	40.6

[1] - By weight of the Aggregate

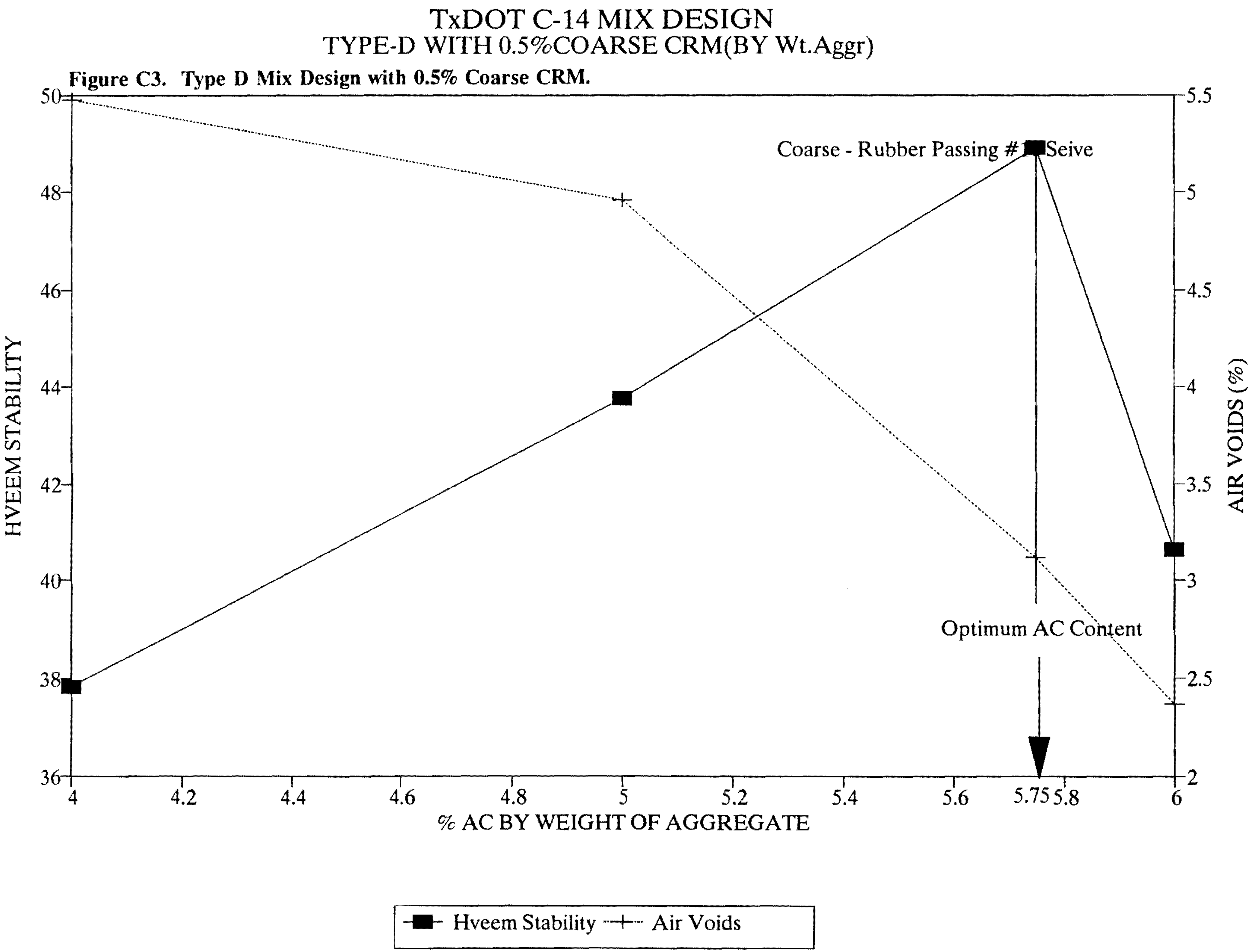


Figure C3. Type D Mix Design with 0.5% Coarse CRM.

Table C7. **Standard Texas Type-D Gradation Blended With 10% Field Sand and *0.6%*[1] *Coarse (-10) CRM*.**

Sieve Size / Crushed Stone	% Passing	% Each	Combination Each %	Cumulative Weight (grams)
1/2"	100.0	0.0	0.0	0.0
3/8"	92.0	8.0	8.0	320.0
#4	60.0	32.0	32.0	1600.0
#10	37.0	23.0	23.0	2520.0
#40	21.0	16.0	16.0	3160.0
#80	9.0	3.5	12.0	3300.0
#200	4.0	2.25	5	3390.0
Passing #200		2.55	4	3492.0
Sand				
#40	100.0	0.0		3492.0
#80	85.0	8.5		3769.0[2]
#200	42.5	2.75		3879.0
Passing #200	14.5	1.45		3937.0

1 - By weight of the aggregate.
2 - Since the maximum nominal size of the rubber is #10 size equivalent volume of the sand is replaced on #80 sieve size.

Table C8: **Summary Of The Mix Design Data For *0.6% Coarse (-10) CRM* By Weight of Aggregate.**

Asphalt Content	5.75% (By weight of the aggregate)			
Rice Sp.Gravity	2.382			
Sample #	1	2	3	Average
Bulk Sp.Gravity	2.287	2.267	2.292	2.282
Air Voids,%	3.9	4.8	3.8	4.2
Hveem Stability	27.2	22.8	27.2	25.7

Table C9. Standard Texas Type-D Gradation Blended With 10% Field Sand and *0.7%*[1] *Coarse (-10) CRM.*

Sieve Size	% Passing	% Each	Combination Each %	Cumulative Weight (grams)
Crushed Stone				
1/2"	100.0	0.0	0.0	0.0
3/8"	92.0	8.0	8.0	320.0
#4	60.0	32.0	32.0	1600.0
#10	37.0	23.0	23.0	2520.0
#40	21.0	16.0	16.0	3160.0
#80	9.0	3.5	12.0	3300.0
#200	4.0	2.25	5	3390.0
Passing #200		2.55	4	3492.0
Sand				
#40	100.0	0.0		3492.0
#80	85.0	8.5		3758.5[2]
#200	42.5	2.75		3868.5
Passing #200	14.5	1.45		3926.5

1 - By weight of the aggregate.
2 - Since the maximum nominal size of the rubber is #10 size equivalent volume of the sand is replaced on #80 sieve size.

Table C10: Summary Of Mix Design Data For *0.7% Coarse (-10) CRM* By Weight of Aggregate.

Asphalt Content	5.75% (By weight of the aggregate)			
Rice Sp.Gravity	2.358			
Sample #	1	2	3	Average
Bulk Sp.Gravity	2.249	2.271	2.251	2.257
Air Voids,%	4.6	3.7	4.5	4.3
Hveem Stability	23.7	27.7	27.6	26.3

Table C11. Summary Of The Static Creep Test Data For Control Mix With Optimum Rouse Content(0.5% By Weight Of The Aggregate).

Sample#	1	2	3	Average
Air Voids %	3.1	3.3	3.5	3.3
AC Content[1] %	5.75	5.75	5.75	5.75
Permanent Strain in/in(cm/cm)	15.2×10^{-4}	4.5×10^{-4}	6.8×10^{-4}	8.8×10^{-4}
Slope in/in sec (cm/cm sec)	13.0×10^{-8}	6.1×10^{-8}	8.8×10^{-8}	9.3×10^{-8}
Creep Stiffness psi (Kg/cm^2)	3076 (216.3)	3660 (257.3)	2373 (166.8)	3036 (213.5)

[1] - By weight of the aggregate

PERFORMANCE EVALUATION OF THE MIXTURE USING AAMAS

Table C12. AAMAS Test Results For *Unconditioned Specimens @41°F.*

Rice Specific Gravity	2.372			
Sample#	1	4	9	Average
Bulk Specific Gravity	2.238	2.253	2.240	2.244
Air Voids,%	5.64	5.02	5.54	5.39
Total Resilient Modulus[1], psi (Kg/cm^2)	$14.44x10^5$ (101493)	$13.40x10^5$ (94209)	$14.68x10^5$ (103230)	$14.17x10^5$ (99644)
Indirect Tensile Strength, psi (Kg/cm^2)	102.75 (7.22)	129.97 (9.14)	127.81 (8.99)	120.18 (8.45)
Indirect Tensile Strain @Failure, in/in(cm/cm) (10^{-3})	3.83	2.77	2.49	3.03

[1] - Average of the two Axes

Table C13. AAMAS Test Results For *Unconditioned Specimens @77°F.*

Rice Specific Gravity	2.372			
Sample#	5	13	15	Average
Bulk Specific Gravity	2.251	2.241	2.228	2.240
Air Voids,%	5.12	5.51	6.10	5.58
Total Resilient Modulus[1], psi (Kg/cm^2)	$3.13x10^5$ (21977)	$3.21x10^5$ (22591)	$3.49x10^5$ (24526)	$3.28x10^5$ (23031)
Indirect Tensile Strength, psi (Kg/cm^2)	99.26 (6.99)	97.94 (6.90)	93.57 (6.58)	96.92 (6.81)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	6.13	6.43	6.04	6.20

[1] - Average of the two Axes

Table C14. AAMAS Test Results For *Unconditioned Specimens @104°F.*

Rice Specific Gravity	2.372			
Sample#	7	17	18	Average
Bulk Specific Gravity	2.236	2.240	2.246	2.240
Air Voids,%	5.72	5.55	5.32	5.58
Total Resilient Modulus[1], psi (Kg/cm^2)	$1.20x10^5$ (8429)	$1.37x10^5$ (9622)	$1.55x10^5$ (10871)	$1.37x10^5$ (9641)
Indirect Tensile Strength, psi (Kg/cm^2)	27.92 (1.97)	31.63 (2.23)	32.54 (2.29)	30.70 (2.16)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	10.38	8.71	11.16	10.08

[1] - Average of the two Axes

Table C15. AAMAS Test Results For *Moisture Conditioned* Specimens Tested @77°F.

Rice Specific Gravity	2.372			
Sample#	3	11	12	Average
Bulk Specific Gravity	2.229	2.242	2.251	2.241
Air Voids,%	6.02	5.46	5.10	5.52
Degree Of Saturation, %	46.62	45.83	45.32	45.92
Total Resilient Modulus[1], psi (Kg/cm^2)	$3.01x10^5$ (21165)	$2.80x10^5$ (19721)	$3.34x10^5$ (23495)	$3.05x10^5$ (21460)
Indirect Tensile Strength, psi (KG/cm^2)	83.46 (5.89)	83.10 (5.84)	93.02 (6.55)	86.53 (6.08)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	10.85	9.67	9.90	10.14

[1] - Average of the two Axes

Table C16. AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-1.

Rice Specific Gravity	2.372			
Sample#	6	10	14	Average
Bulk Specific Gravity	2.246	2.242	2.237	2.242
Air Voids,%	5.32	5.48	5.68	5.49
Total Resilient Modulus[1], psi (Kg/cm^2)	$16.15x10^5$ (113548)	$18.67x10^5$ (131241)	$16.89x10^5$ (118775)	$17.24x10^5$ (121188)
Recovery Efficiency	0.75	0.53	0.60	0.63
Indirect Tensile Creep Modulus @3600sec, psi(kg/cm^2)	74331 (5226)	66770 (4695)	58354 (4102)	66482 (4674)

[1] - Average of the two Axes

Table C17. AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-2.

Rice Specific Gravity	2.372			
Sample#	2	8	16	Average
Bulk Specific Gravity	2.247	2.246	2.232	2.242
Air Voids,%	5.26	5.29	5.88	5.48
Total Resilient Modulus[1], psi (Kg/cm^2)	$19.77x10^5$ (138988)	$20.91x10^5$ (147014)	$22.24x10^5$ (156370)	$20.97x10^5$ (147457)
Indirect Tensile Strength, psi (Kg/cm^2)	148.23 (10.42)	151.03 (10.62)	144.91 (10.19)	148.06 (10.41)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	1.36	1.21	1.19	1.25

[1] - Average of the two Axes

Table C18. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-1.

Rice Specific Gravity	2.372			
Sample#	5	8	[1]	Average
Bulk Specific Gravity	2.336	2.346		2.341
Air Voids,%	1.51	1.08		1.30
Total Resilient Modulus, psi(kg/cm^2)	118619 (8340)	140154 (9854)		129387 (9097)
Slope Of Compressive Creep Test Curve, b	0.07649	0.05720		0.06685
Intercept Of Compressive Creep Test Curve, a	0.00248	0.00368		0.00308
Total Permanent Deformation @3600sec, in/in(cm/cm)	0.00463	0.00583		0.00523
Compressive Creep Modulus @3600sec, psi(kg/cm^2)	12789	10200		11495 (808)

[1] - One of the LVDT's was out of range, so the data was discarded.

Table C19. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-2.

Rice Specific Gravity	2.372			
Sample#	2	4	7	Average
Bulk Specific Gravity	2.355	2.33	2.352	2.346
Air Voids,%	0.7	1.8	0.8	1.1
Unconfined Compressive Strength, psi (Kg/cm^2)	249.1 (17.5)	269.7 (18.9)	240.9 (16.9)	253.2 (17.8)
Compressive Strain @Failure, in/in (cm/cm) (10^{-3})	34.1	24.7	45.0	34.6

[1] - Average of the two Axes

Table C20. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-3.

Rice Specific Gravity	2.372		
Sample#	3	6	Average
Bulk Specific Gravity	2.345	2.341	2.343
Air Voids,%	1.12	1.31	1.22
Dynamic Resilient Modulus @200thcycle, psi(kg/cm^2)	124700 (8768)	147200 (10350)	135950 (9559)
Slope Of Repetitive Creep Test Curve, b	0.34470	0.28159	0.31314
Intercept Of Repetitive Creep Test Curve, a	0.00083	0.00209	0.00146
Total Deformation @10000thcycle, in/in(cm/cm)	0.01363	0.00959	0.01161

Appendix D

Laboratory Data for 10%FW Mixture

(10% Fine Rubber, by Weight of Asphalt, via Wet Method)

MIX DESIGN FOR CRUMB RUBBER MODIFIED ASPHALT MIXES USING TxDOT (TEX-232-F) PROCEDURE WET METHOD -#80 SIZE RUBBER @10% BY WEIGHT OF ASPHALT

STEP 1: Trial Gradation Weights For Varying Coarse To Fine Fraction

Batch Weight: 4000g Binder Content: 5%(by weight of aggregate)

Table D1. Gradation for Various Fractions of +#10 Size to -#10 Size.

Sieve Size	60/40*		65/35		70/30	
Crushed Stone	% Each	Mix Wt.	% Each	Mix Wt.	% Each	Mix Wt.
1/2"	0	0	0	0	0	0
3/8"	1.3	53.1	1.4	57.5	1.5	61.9
#4	36.5	1512.5	39.5	1638.6	42.6	1764.6
#10	22.2	2400	24	2600	25.9	2800
#40	19.4	3175.9	16.6	3263.8	13.8	3351.6
#80	2.1	3259.8	1.8	3335.5	1.5	3411.3
#200	2	3338.4	1.7	3402.8	1.4	3467.2
Passing #200	4.7	3524.4	4.7	3588.8	4.7	3653.2
Sand 1/2"	0	3524.4	0	3588.8	0	3653.2
3/8"	0	3524.4	0	3588.8	0	3653.2
#4	0	3524.4	0	3588.8	0	3653.2
#10	0	3524.4	0	3588.8	0	3653.2
#40	0.5	3545.4	0.4	3606.7	0.4	3668.1
#80	7.1	3828.5	6.1	3848.9	5	3869.4
#200	3.5	3970	3	3970	2.5	3970
Passing #200	0.8	4000	0.8	4000	0.8	4000
Total	100	4000	100	4000	100	4000

* - Coarse to fine fraction (60% material by weight retained on #10 Sieve)

(Table D1 continued......)

Table D1. Continued.

Sieve Size	75/25*		80/20		85/15	
Crushed Stone	% Each	Mix Wt.	% Each	Mix Wt.	% Each	Mix Wt.
1/2"	0	0	0	0	0	0
3/8"	1.7	66.3	1.8	70.8	1.9	75.2
#4	45.6	1890.7	48.6	2016.7	51.7	2142.8
#10	27.7	3000	29.6	3200	31.4	3400
#40	11	3439.5	8.2	3527.4	5.4	3615.3
#80	1.2	3487	0.9	3562.8	0.6	3638.5
#200	1.1	3531.6	0.8	3596	0.5	3660.4
Passing #200	4.7	3717.6	4.7	3782	4.7	3846.4
Sand 1/2"	0	3717.6	0	3782	0	3846.4
3/8"	0	3717.6	0	3782	0	3846.4
#4	0	3717.6	0	3782	0	3846.4
#10	0	3717.6	0	3782	0	3846.4
#40	0.3	3729.5	0.2	3790.8	0.1	3852.2
#80	4.0	3889.8	3.0	3910.3	2.0	3930.7
#200	2.0	3970	1.5	3970	1.0	3970
Passing #200	0.8	4000	0.8	4000	0.8	4000
Total	100	4000	100	4000	100	4000

* - Coarse to fine fraction (75% material by weight retained on #10 Sieve)

Table D2. Summary Of Densities Of The Trial Batches Described In Table D1.

Coarse/ Fine (By Weight)	Wt. of +#10 Material (%)	Volume of +#10 Material (%)	Density(%)			
			Sample #1	Sample #2	Sample #3	Average
60/40	60	50.0	96.7	96.0	96.2	96.3
65/35	65	54.3	96.6	97.2	96.9	96.9
70/30	70	58.5	97.2	98.0	97.4	97.5
75/25	75	62.6	95.3	95.8	94.5	95.2
80/20	80	66.7	97.2	97.8	97.8	97.6
85/15	85	70.2	96.4	96.3	95.8	96.2

STEP 2: The objective of this step is achieve a relative density of 97±0.2% by trial and error method. With the values from Table 2 a graph was plotted with volume of the +#10 material on the X-axis and percent density on the Y-axis. From the graph a gradation that gave the maximum density was selected. To that mix 5% of +#10 size material is added by volume and binder content is increased to 8.2% by weight of the aggregate. The following is the summary of the different trials with the varying volume fraction of the +#10 size material.

TABLE D3. Summary Of Trials To Achieve 97±0.2% Density.

Trial #	Volume of +#10 material(%)	Density(%)			
		Sample#1	Sample#2	Sample#3	Average
1	67.6	97.8	96.8	98.4	97.7
2	74.5	95.7	95.6	94.9	95.4
3	74.0	96.0	96.1	96.1	96.0
4	73.0	97.5	97.1	97.2	97.3
5	72.6	97.2	97.4	97.08	97.2

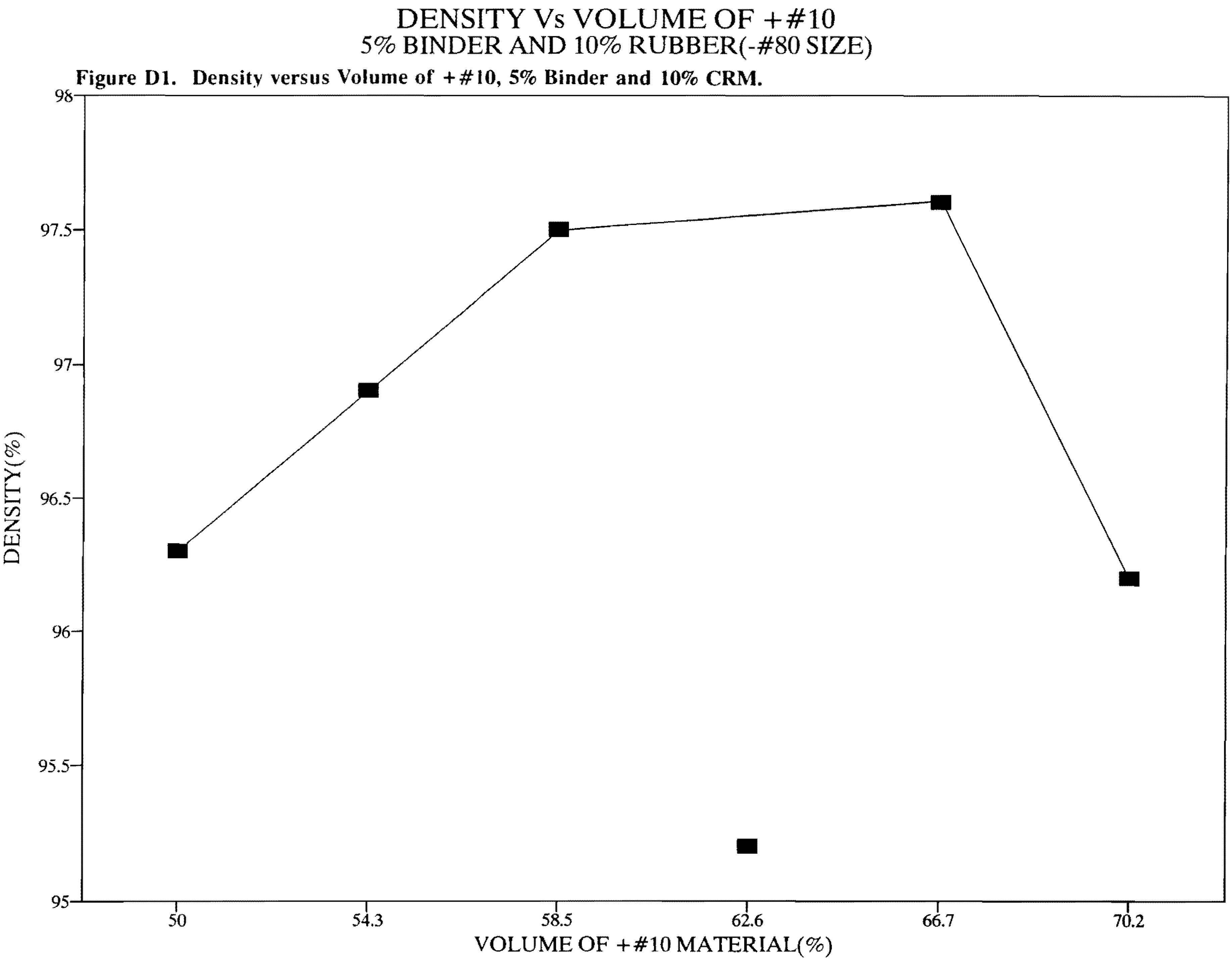


Figure D1. Density versus Volume of +#10, 5% Binder and 10% CRM.

DENSITY Vs VOLUME OF +#10
10% RUBBER(-#80 SIZE)

Figure D2. Density versus Volume of +#10, 10% CRM.

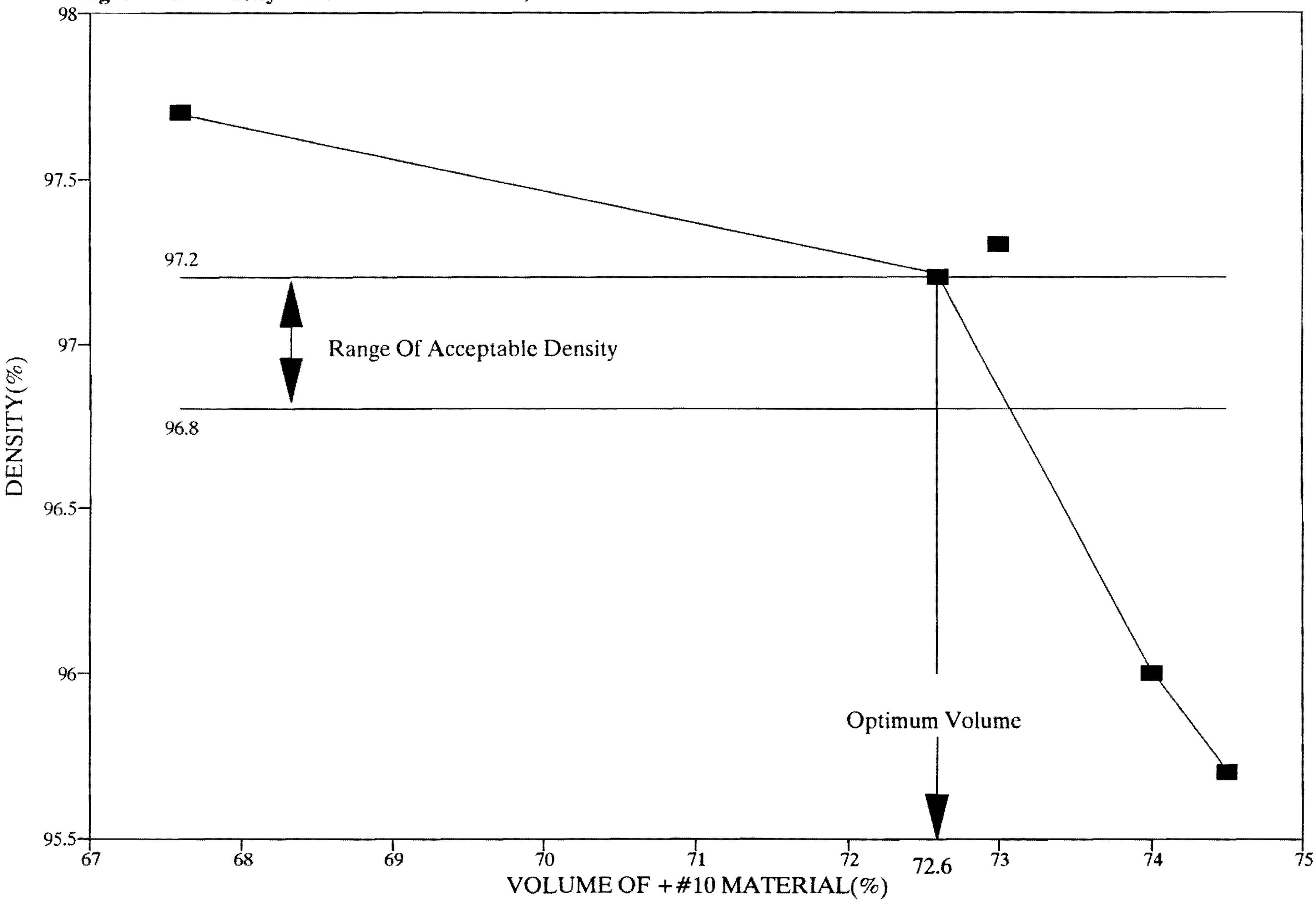


Table D4. Final Gradation For Evaluation Of Mixture Using AAMAS.

Sieve Size		
Crushed Stone	% Each	Mix Weight(Grams)
1/2"	0.0	0.0
3/8"	1.9	80.7
#4	51.3	2301.2
#10	30.8	3633.5
#40	1.9	3715.4
#80	2.2	3809.7
#200	1.4	3868.8
Passing #200	1.8	3947.8
Sand		
1/2"	0.0	3947.8
3/8"	0.0	3947.8
#4	0.0	3947.8
#10	0.0	3947.8
#40	0.0	3947.8
#80	0.8	3984.4
#200	0.2	3993.6
Passing #200	0.1	4000.0

STEP 3: Static creep test was run on the samples with densities 97$\pm$0.2% according to standard test specification TEX-231-F. The results are tabulated as follows.

Table D5. Summary Of The Static Creep Test Data For 10%[1](Passing #80 size Rubber).

Sample#	1	2	3	Average
Air Voids %	2.8	2.6	2.92	2.8
AC Content[2] %	8.2	8.2	8.2	8.2
Permanent Strain in/in(cm/cm)	4.53×10^{-4}	4.06×10^{-4}		4.3×10^{-4}
Slope in/in sec (cm/cm sec)	4.2×10^{-8}	4.5×10^{-8}		4.35×10^{-8}
Creep Stiffness psi (Kg/cm^2)	7423.0 (519.5)	7253.0 (509.95)		7339 (516.0)

[1] - By weight of the asphalt content
[2] - By weight of the aggregate

PERFORMANCE EVALUATION OF THE MIXTURE USING AAMAS

Table D6. AAMAS Test Results For *Unconditioned Specimens @41°F.*

Rice Specific Gravity	2.325			
Sample#	4	5	13	Average
Bulk Specific Gravity	2.171	2.170	2.170	2.170
Air Voids,%	6.62	6.67	6.67	6.65
Total Resilient Modulus[1], psi (Kg/cm^2)	7.40×10^5 (52048)	7.33×10^5 (51557)	8.10×10^5 (56969)	7.61×10^5 (53525)
Indirect Tensile Strength, psi (Kg/cm^2)	81.42 (5.73)	75.52 (5.31)		78.47 (5.52)
Indirect Tensile Strain @Failure, in/in(cm/cm) (10^{-3})	5.03	3.14		4.08

[1] - Average of the two Axes

Table D7. AAMAS Test Results For *Unconditioned Specimens @77°F.*

Rice Specific Gravity	2.325			
Sample#	1	2	12	Average
Bulk Specific Gravity	2.190	1.158	2.159	2.169
Air Voids,%	5.81	7.18	7.14	6.71
Total Resilient Modulus[1], psi (Kg/cm^2)	2.10×10^5 (14780)	1.92×10^5 (13508)	1.89×10^5 (13269)	1.97×10^5 (13852)
Indirect Tensile Strength, psi (Kg/cm^2)	63.44 (4.46)	61.11 (4.30)	56.67 (3.98)	60.41 (4.25)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	5.9	8.16	7.77	7.28

[1] - Average of the two Axes

Table D8. AAMAS Test Results For *Unconditioned Specimens @104°F.*

Rice Specific Gravity	2.325			
Sample#	7	15	16	Average
Bulk Specific Gravity	2.180	2.166	2.164	2.170
Air Voids,%	6.24	6.84	6.92	6.67
Total Resilient Modulus[1], psi (Kg/cm^2)	77540 (5452)	66960 (4708)	61230 (4305)	68577 (4822)
Indirect Tensile Strength, psi (Kg/cm^2)	19.77 (1.39)	16.66 (1.17)	15.72 (1.11)	17.38 (1.22)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	12.77	12.63	9.88	11.76

[1] - Average of the two Axes

Table D9. AAMAS Test Results For *Moisture Conditioned* Specimens Tested @77°F.

Rice Specific Gravity	2.325			
Sample#	11	17	18	Average
Bulk Specific Gravity	2.165	2.165	2.186	2.171
Air Voids,%	6.88	6.88	6.02	6.59
Degree Of Saturation, %	35.7	24.0	38.8	35.85
Total Resilient Modulus[1], psi (Kg/cm^2)	1.95×10^5 (13715)	1.92×10^5 (13508)	2.27×10^5 (15986)	2.05×10^5 (14403)
Indirect Tensile Strength, psi (KG/cm^2)	60.14 (4.23)	65.80 (4.63)	54.12 (3.81)	60.20 (4.22)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	9.90	9.66	32.64	17.40

[1] - Average of the two Axes

Table D10. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-1.**

Rice Specific Gravity	2.325			
Sample#	6	9	10	Average
Bulk Specific Gravity	2.164	2.175	2.174	2.171
Air Voids,%	6.9	6.46	6.5	6.62
Total Resilient Modulus[1], psi (Kg/cm^2)	11.59×10^5 (81454)	9.60×10^5 (67506)	11.86×10^5 (83376)	11.01×10^5 (77445)
Recovery Efficiency	0.47	0.50	0.55	0.51
Indirect Tensile Creep Modulus @3600sec, psi(kg/cm^2)	15753 (1108)	19104 (1343)	26596 (1870)	20484 (1440)

[1] - Average of the two Axes

Table D11. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-2.**

Rice Specific Gravity	2.325			
Sample#	3	8	14	Average
Bulk Specific Gravity	2.167	2.160	2.186	2.171
Air Voids,%	6.80	7.10	5.99	6.62
Total Resilient Modulus[1], psi (Kg/cm^2)	10.44×10^5 (73422)	11.93×10^5 (83877)	12.29×10^5 (86379)	11.55×10^5 (81226)
Indirect Tensile Strength, psi (Kg/cm^2)	112.25 (7.89)	121.16 (8.52)	115.37 (8.11)	116.26 (8.17)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	1.84	1.69	2.20	1.91

[1] - Average of the two Axes

Table D12. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-1.

Rice Specific Gravity			
Sample#	4	5	Average
Bulk Specific Gravity	2.26	2.29	2.28
Air Voids,%	2.8	1.51	2.37
Total Uniaxial Resilient Modulus, psi(kg/cm^2)	115724 (8134)	123488 (8682)	137125 (9641)
Slope Of Compressive Creep Test Curve, b		0.11065	0.11065
Intercept Of Compressive Creep Test Curve, a		0.00204	0.00204
Total Permanent Deformation @3600sec, in/in(cm/cm)		0.00503	0.00503
Compressive Creep Modulus @3600sec, psi(kg/cm^2)	11586[1] (814.6)	11740 (825.4)	11740 (825.4)

[1] - One of the LVDTs was out of range, so we had to disregard this sample

Table D13. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-2.

Rice Specific Gravity	2.325			
Sample#	2	3	6	Average
Bulk Specific Gravity	2.28	2.27	2.27	2.27
Air Voids,%	1.94	2.37	2.37	2.22
Unconfined Compressive Strength, psi (Kg/cm^2)	189.42 (13.32)	176.21 (13.4)	172.13 (12.1)	179.25 (12.6)
Compressive Strain @Failure, in/in (cm/cm) (10^{-3})	44.9	32.4	30.4	35.9

Table D14. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-3.

Rice Specific Gravity	2.325		
Sample#	1	2	Average
Bulk Specific Gravity	2.254	2.259	2.257
Air Voids,%	3.10	2.80	2.95
Dynamic Resilient Modulus @200th cycle, psi(kg/cm^2)	141100 (9921)	144700 (10174)	142900 (10047)
Slope Of Repetitive Creep Test Curve, b	0.51111	0.41650	0.46380
Intercept Of Repetitive Creep Test Curve, a	0.00022	0.00062	0.00042
Total Permanent Deformation @10000cycles, in/in(cm/cm)	0.01523	0.01809	0.01666

Appendix E

Laboratory Data for 18%FW Mixture

(18% Fine Rubber, by Weight of Asphalt, via Wet Method)

MIX DESIGN FOR CRUMB RUBBER MODIFIED ASPHALT MIXES
USING TxDOT (TEX-232-F) PROCEDURE
WET METHOD -#80 SIZE RUBBER @18% BY WEIGHT OF ASPHALT

STEP 1: Trial Gradation Weights For Varying Coarse To Fine Fraction

Batch Weight: 4000g   Binder Content: 5%(by weight of aggregate)

Table E1. Gradation for Various Fractions of +#10 Size to -#10 Size.

Sieve Size	60/40*		65/35		70/30	
Crushed Stone	% Each	Mix Wt.	% Each	Mix Wt.	% Each	Mix Wt.
1/2"	0	0	0	0	0	0
3/8"	1.3	53.1	1.4	57.5	1.5	61.9
#4	36.5	1512.5	39.5	1638.6	42.6	1764.6
#10	22.2	2400	24	2600	25.9	2800
#40	19.4	3175.9	16.6	3263.8	13.8	3351.6
#80	2.1	3259.8	1.8	3335.5	1.5	3411.3
#200	2	3338.4	1.7	3402.8	1.4	3467.2
Passing #200	4.7	3524.4	4.7	3588.8	4.7	3653.2
Sand 1/2"	0	3524.4	0	3588.8	0	3653.2
3/8"	0	3524.4	0	3588.8	0	3653.2
#4	0	3524.4	0	3588.8	0	3653.2
#10	0	3524.4	0	3588.8	0	3653.2
#40	0.5	3545.4	0.4	3606.7	0.4	3668.1
#80	7.1	3828.5	6.1	3848.9	5	3869.4
#200	3.5	3970	3	3970	2.5	3970
Passing #200	0.8	4000	0.8	4000	0.8	4000
Total	100	4000	100	4000	100	4000

* - Coarse to fine fraction (60% material by weight retained on #10 Sieve)

(Table 1 continued......)

Table E1. Continued.

Sieve Size	75/25*		80/20		85/15	
Crushed Stone	% Each	Mix Wt.	% Each	Mix Wt.	% Each	Mix Wt.
1/2"	0	0	0	0	0	0
3/8"	1.7	66.3	1.8	70.8	1.9	75.2
#4	45.6	1890.7	48.6	2016.7	51.7	2142.8
#10	27.7	3000	29.6	3200	31.4	3400
#40	11	3439.5	8.2	3527.4	5.4	3615.3
#80	1.2	3487	0.9	3562.8	0.6	3638.5
#200	1.1	3531.6	0.8	3596	0.5	3660.4
Passing #200	4.7	3717.6	4.7	3782	4.7	3846.4
Sand 1/2"	0	3717.6	0	3782	0	3846.4
3/8"	0	3717.6	0	3782	0	3846.4
#4	0	3717.6	0	3782	0	3846.4
#10	0	3717.6	0	3782	0	3846.4
#40	0.3	3729.5	0.2	3790.8	0.1	3852.2
#80	4.0	3889.8	3.0	3910.3	2.0	3930.7
#200	2.0	3970	1.5	3970	1.0	3970
Passing #200	0.8	4000	0.8	4000	0.8	4000
Total	100	4000	100	4000	100	4000

* - Coarse to fine fraction (75% material by weight retained on #10 Sieve)

Table E2. Summary Of Densities Of The Trial Batches Described In Table E1.

Coarse/ Fine (By Weight)	Wt. of +#10 Material (%)	Volume of +#10 Material (%)	Density(%)			
			Sample #1	Sample #2	Sample #3	Average
60/40	60	48.5	93.2	93.4	93.2	93.3
65/35	65	53.3	95.5	95.4	95.8	95.6
70/30	70	57.5	96.2	95.8	95.7	95.9
75/25	75	61.5	96.3	96.2	95.8	96.1
80/20	80	65.8	97.2	97.1	96.4	96.9
85/15	85	69.5	96.0	96.0	96.2	96.1

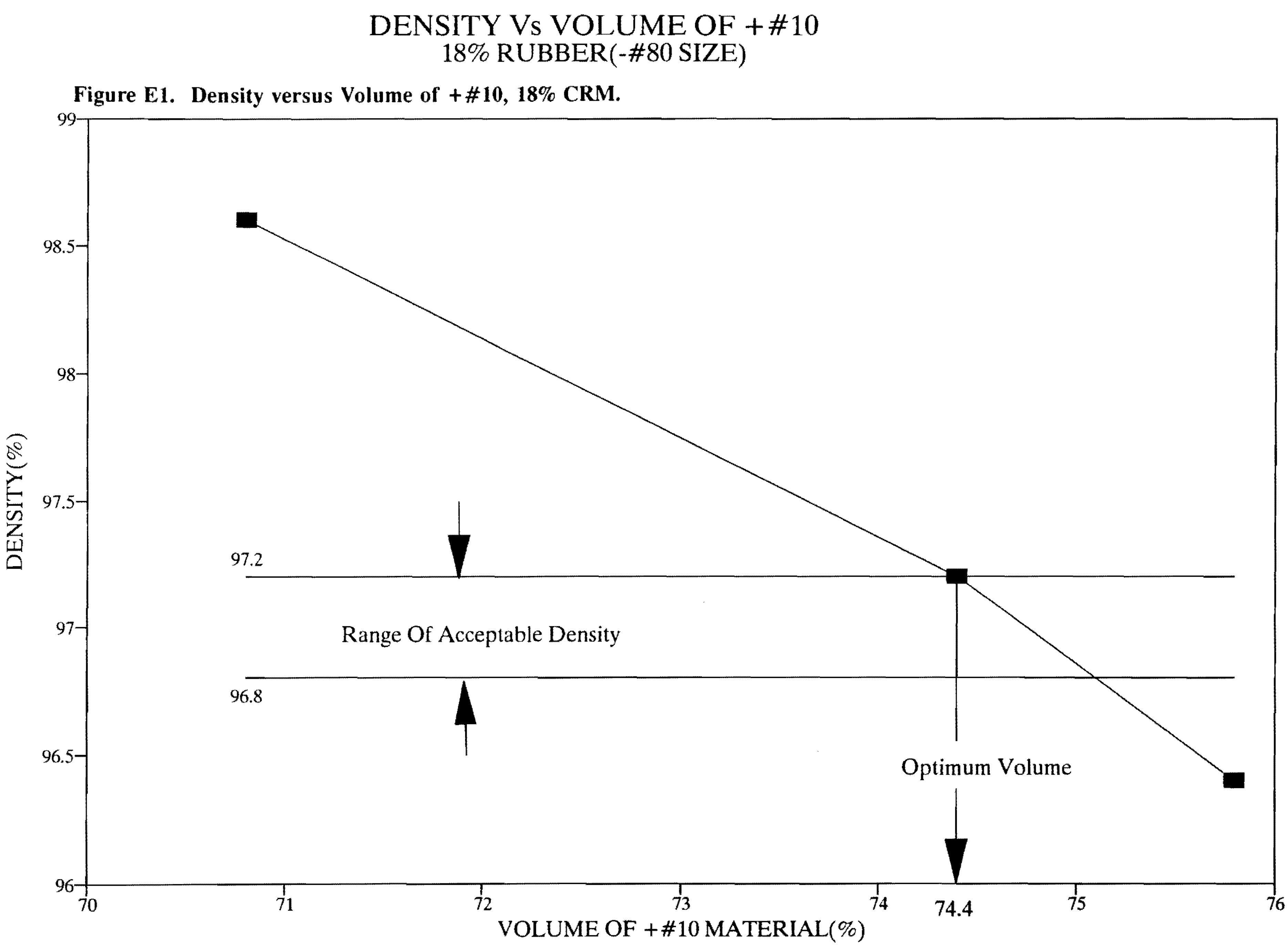
STEP 2: The objective of this step is achieve a relative density of 97±0.2% by trial and error method. With the values from Table 2 a graph was plotted with volume of the +#10 material on the X-axis and percent density on the Y-axis. From the graph a gradation that gave the maximum density was selected. To that mix 5% of +#10 size material is added by volume and binder content is increased to 8.2% by weight of the aggregate. The following is the summary of the different trials with the varying volume fraction of the +#10 size material.

Table E3. Summary Of Trials To Achieve 97±0.2% Density.

Trial #	Volume of +#10 material(%)	Density(%)			
		Sample#1	Sample#2	Sample#3	Average
1	70.8	98.3	98.7	98.6	98.6
2	75.8	96.7	96.6	95.8	96.4
3	74.4	97.2	97.2	97.5	97.2

From the above table the gradation that has 74.4% of the +#10 Sieve size material by volume is selected. These three samples were tested for static creep according to standard test method TEX-231-F. The same gradation is used for evaluating the mixture by using Asphalt-Aggregate Mixture Analysis System (AAMAS). The gradation is tabulated below.

Figure E1. Density versus Volume of +#10, 18% CRM.



DENSITY Vs VOLUME OF +#10
5% BINDER AND 18% RUBBER(-#80 SIZE)

Figure E2. Density versus Volume of +#10, 5% Binder and 18% CRM.

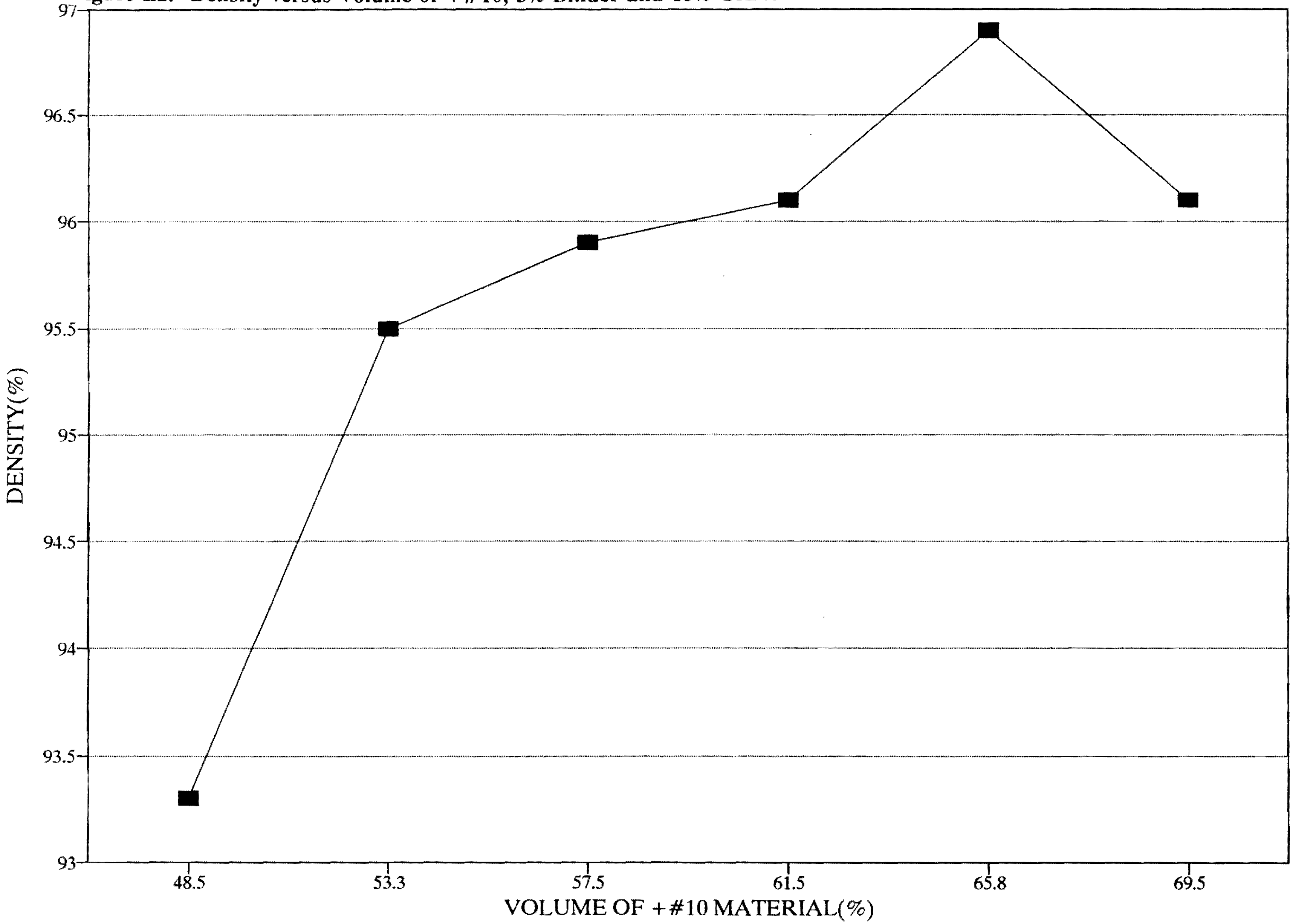


Table E4. Final Gradation For Evaluation Of Mixture Using AAMAS.

Sieve Size Crushed Stone	% Each	Mix Weight(Grams)
1/2"	0.0	0.0
3/8"	1.9	82.7
#4	52.6	2357.7
#10	31.5	3722.7
#40	1.4	3784.7
#80	1.6	3856.0
#200	1.0	3900.7
Passing #200	1.4	3960.5
Sand		
1/2"	0.0	3960.5
3/8"	0.0	3960.5
#4	0.0	3960.5
#10	0.0	3960.5
#40	0.0	3960.5
#80	0.6	3988.2
#200	0.2	3995.2
Passing #200	0.1	4000.0

STEP 3: Static creep test was run on the samples with densities 97$\pm$0.2% according to standard test specification TEX-231-F. The results are tabulated as follows.

Table E5. Summary Of The Static Creep Test Data For 18%[1](Passing #80 size Rubber).

Sample#	1	2	3	Average
Air Voids %	2.8	2.82	2.5	2.8
AC Content[2] %	8.2	8.2	8.2	8.2
Permanent Strain in/in(cm/cm)	8.60×10^{-4}	3.32×10^{-4}	2.72×10^{-4}	4.9×10^{-4}
Slope in/in sec (cm/cm sec)	8.40×10^{-8}	2.50×10^{-8}	2.1×10^{-8}	4.3×10^{-8}
Creep Stiffness psi (Kg/cm^2)	6836.0 (480.64)	10561.0 (742.5)	9659 (679.1)	9019 (634.1)

[1] - By weight of the asphalt content
[2] - By weight of the aggregate

PERFORMANCE EVALUATION OF THE MIXTURE USING AAMAS

Table E6. AAMAS Test Results For *Unconditioned Specimens @41°F.*

Rice Specific Gravity	2.320			
Sample#	3	5	13	Average
Bulk Specific Gravity	2.145	2.17	2.165	2.16
Air Voids,%	7.54	6.47	6.68	6.9
Total Resilient Modulus[1], psi (Kg/cm^2)	[2]	7.26×10^5 (51045)	7.39×10^5 (51959)	7.34×10^5 (51608)
Indirect Tensile Strength, psi (Kg/cm^2)	123.2 (8.66)	143.6 (10.1)	158.04 (11.11)	141.6 (9.96)
Indirect Tensile Strain @Failure, in/in(cm/cm) (10^{-3})	11.9	11.2	11.21	11.4

[1] - Average of the two Axes
[2] - Computer key board was stuck and the data was lost

Table E7. AAMAS Test Results For *Unconditioned Specimens @77°F.*

Rice Specific Gravity	2.320			
Sample#	4	9	11	Average
Bulk Specific Gravity	2.153	2.176	2.160	2.16
Air Voids,%	7.20	6.21	6.90	6.9
Total Resilient Modulus[1], psi (Kg/cm^2)	1.76×10^5 (12375)	2.67×10^5 (18723)	3.73×10^5 (26226)	2.72×10^5 (19124)
Indirect Tensile Strength, psi (Kg/cm^2)	53.21 (3.74)	58.78 (4.13)	56.39 (3.96)	56.13 (3.95)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	14.62	15.29	18.64	16.18

[1] - Average of the two Axes

Table E8. AAMAS Test Results For *Unconditioned Specimens @104°F.*

Rice Specific Gravity	2.320			
Sample#	8	16	17	Average
Bulk Specific Gravity	2.177	2.156	2.151	2.161
Air Voids,%	6.16	7.07	7.28	6.84
Total Resilient Modulus[1], psi (Kg/cm^2)	$1.43x10^5$ (10054)	$1.68x10^5$ (11812)	$1.46x10^5$ (10265)	$1.54x10^5$ (10827)
Indirect Tensile Strength, psi (Kg/cm^2)	24.32 (1.71)	24.00 (1.67)	21.89 (1.54)	23.40 (1.65)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	22.89	21.82	24.11	22.94

[1] - Average of the two Axes

Table E9. AAMAS Test Results For *Moisture Conditioned* Specimens Tested @77°F.

Rice Specific Gravity	2.320			
Sample#	6	12	14	Average
Bulk Specific Gravity	2.150	2.158	2.174	2.161
Air Voids,%	7.33	6.98	6.29	6.87
Degree Of Saturation, %	66.6	70.3	70.2	69.03
Total Resilient Modulus[1], psi (Kg/cm^2)	$3.24x10^5$ (22780)	$3.96x10^5$ (27843)	$4.71x10^5$ (33116)	$3.82x10^5$ (26858)
Indirect Tensile Strength, psi (KG/cm^2)	53.64 (3.78)	59.78 (4.26)	62.47 (4.40)	58.63 (4.12)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	19.66	22.42	20.76	20.95

[1] - Average of the two Axes

Table E10. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-1.**

Rice Specific Gravity	2.320			
Sample#	2	15	18	Average
Bulk Specific Gravity	2.163	2.171	2.156	2.163
Air Voids,%	6.77	6.42	7.07	6.75
Total Resilient Modulus[1], psi (Kg/cm^2)	13.27×10^5 (93318)	14.48×10^5 (101863)	13.55×10^5 (95270)	14.02×10^5 (98575)
Recovery Efficiency	0.72	0.70	0.6	0.67
Indirect Tensile Creep Modulus @3600sec, psi(kg/cm^2)	71340 (5016)	53950 (3793)	53560 (3977)	59615 (4195)

[1] - Average of the two Axes

Table E11. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-2.**

Rice Specific Gravity	2.320			
Sample#	1	7	10	Average
Bulk Specific Gravity	2.169	2.159	2.158	2.162
Air Voids,%	6.51	6.94	6.98	6.81
Total Resilient Modulus[1], psi (Kg/cm^2)	22.98×10^5 (161572)	23.76×10^5 (167057)	25.19×10^5 (177109)	23.97×10^5 (168568)
Indirect Tensile Strength, psi (Kg/cm^2)	193.12 (13.60)	179.97 (12.56)	197.42 (13.88)	190.17 (13.37)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	5.02	4.24	3.47	4.24

[1] - Average of the two Axes

Table E12. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-1.

Rice Specific Gravity	2.320			
Sample#	1a	2	1b	Average
Bulk Specific Gravity	2.27	2.29	2.24	2.28
Air Voids,%	1.73	0.87	2.87	1.15
Total Uniaxial Resilient Modulus, psi(kg/cm^2)	114945 (8082)	95986 (6749)	100351 (7056)	103761 (7295)
Slope Of Compressive Creep Test Curve, b	0.37978	0.09013	0.02435	0.16475
Intercept Of Compressive Creep Test Curve, a	0.0004	0.00177	0.00412	0.00210
Total Permanent Deformation @3600sec, in/in(cm/cm)	0.00912	0.00501	0.00368	0.00593
Compressive Creep Modulus @3600sec, psi(kg/cm^2)	6570 (462)	16137 (1135)	11844 (833)	11517 (810)

Table E13. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-2.

Rice Specific Gravity	2.320			
Sample#	3	5	6	Average
Bulk Specific Gravity	2.29	2.26	2.3	2.28
Air Voids,%	0.87	2.16	0.43	1.15
Unconfined Compressive Strength, psi (Kg/cm^2)	221.5 (15.57)	213.8 (15.03)		217.7 (15.30)
Compressive Strain @Failure, in/in (cm/cm) (10^{-3})	36.5	41.2		38.85

Table E14. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-3.

Rice Specific Gravity	2.320		
Sample#	1	3b	Average
Bulk Specific Gravity	2.265	2.250	2.258
Air Voids,%	2.36	3.02	2.69
Dynamic Resilient Modulus @200thcycle, psi(kg/cm^2)	57300 (4029)	99290 (6981)	78295 (5505)
Slope Of Repetitive Creep Test Curve, b	0.32634	0.32175	0.32405
Intercept Of Repetitive Creep Test Curve, a	0.00146	0.00128	0.00137
Total Permanent Deformation @10000cycles, in/in(cm/cm)	0.01387	0.01484	0.01436

Appendix F

Laboratory Data for 10%CW Mixture

(10% Coarse Rubber, by Weight of Asphalt, via Wet Method)

MIX DESIGN FOR CRUMB RUBBER MODIFIED ASPHALT MIXES
USING TxDOT (TEX-232-F) PROCEDURE
WET METHOD -#10 SIZE RUBBER @10% BY WEIGHT OF ASPHALT

STEP 1: Trial Gradation Weights For Varying Coarse To Fine Fraction

Batch Weight: 4000g Binder Content: 5%(by weight of aggregate)

Table F1. Gradation for Various Fractions of +#10 Size to -#10 Size.

Sieve Size	60/40*		65/35		70/30	
Crushed Stone	% Each	Mix Wt.	% Each	Mix Wt.	% Each	Mix Wt.
1/2"	0	0	0	0	0	0
3/8"	1.3	53.1	1.4	57.5	1.5	61.9
#4	36.5	1512.5	39.5	1638.6	42.6	1764.6
#10	22.2	2400	24	2600	25.9	2800
#40	19.4	3175.9	16.6	3263.8	13.8	3351.6
#80	2.1	3259.8	1.8	3335.5	1.5	3411.3
#200	2	3338.4	1.7	3402.8	1.4	3467.2
Passing #200	4.7	3524.4	4.7	3588.8	4.7	3653.2
Sand 1/2"	0	3524.4	0	3588.8	0	3653.2
3/8"	0	3524.4	0	3588.8	0	3653.2
#4	0	3524.4	0	3588.8	0	3653.2
#10	0	3524.4	0	3588.8	0	3653.2
#40	0.5	3545.4	0.4	3606.7	0.4	3668.1
#80	7.1	3828.5	6.1	3848.9	5	3869.4
#200	3.5	3970	3	3970	2.5	3970
Passing #200	0.8	4000	0.8	4000	0.8	4000
Total	100	4000	100	4000	100	4000

* - Coarse to fine fraction (60% material by weight retained on #10 Sieve)

(Table F1 continued......)

Table F1. Continued.

Sieve Size	75/25*		80/20		85/15	
Crushed Stone	% Each	Mix Wt.	% Each	Mix Wt.	% Each	Mix Wt.
1/2"	0	0	0	0	0	0
3/8"	1.7	66.3	1.8	70.8	1.9	75.2
#4	45.6	1890.7	48.6	2016.7	51.7	2142.8
#10	27.7	3000	29.6	3200	31.4	3400
#40	11	3439.5	8.2	3527.4	5.4	3615.3
#80	1.2	3487	0.9	3562.8	0.6	3638.5
#200	1.1	3531.6	0.8	3596	0.5	3660.4
Passing #200	4.7	3717.6	4.7	3782	4.7	3846.4
Sand 1/2"	0	3717.6	0	3782	0	3846.4
3/8"	0	3717.6	0	3782	0	3846.4
#4	0	3717.6	0	3782	0	3846.4
#10	0	3717.6	0	3782	0	3846.4
#40	0.3	3729.5	0.2	3790.8	0.1	3852.2
#80	4.0	3889.8	3.0	3910.3	2.0	3930.7
#200	2.0	3970	1.5	3970	1.0	3970
Passing #200	0.8	4000	0.8	4000	0.8	4000
Total	100	4000	100	4000	100	4000

* - Coarse to fine fraction (75% material by weight retained on #10 Sieve)

Table F2. Summary Of Densities Of The Trial Batches Described In Table F1.

Coarse/ Fine (By Weight)	Wt. of +#10 Material (%)	Volume of +#10 Material (%)	Density(%)			
			Sample #1	Sample #2	Sample #3	Average
60/40	60	48.8	94.4	93.6	94.0	94.0
65/35	65	53.4	95.6	95.7	95.7	95.7
70/30	70	58.2	96.5	96.6	95.9	96.3
75/25	75	62.3	97.0	97.4	96.9	97.1
80/20	80	65.8	97.5	96.3	96.6	96.8
85/15	85	69.4	94.8	94.8	95.4	95.0

STEP 2: The objective of this step is achieve a relative density of 97±0.2% by trial and error method. With the values from Table 2 a graph was plotted with volume of the +#10 material on the X-axis and percent density on the Y-axis. From the graph a gradation that gave the maximum density was selected. To that mix 5% of +#10 size material is added by volume and binder content is increased to 8.2% by weight of the aggregate. The following is the summary of the different trials with the varying volume fraction of the +#10 size material.

Table F3. Summary Of Trials To Achieve 97±0.2% Density.

Trial #	Volume of +#10 material(%)	Density(%)			
		Sample#1	Sample#2	Sample#3	Average
1	67.5	98.9	98.6	98.6	98.7
2	74.0	93.7	93.9	92.9	93.5
3	69.0	97.7	97.9	98.0	97.9
4	69.5	97.8	97.2	97.8	97.6
5	69.6	96.6	95.8	96.2	96.2
6	70.0	97.5	97.5	97.4	97.5
7	71.0		96.9	97.3	97.1

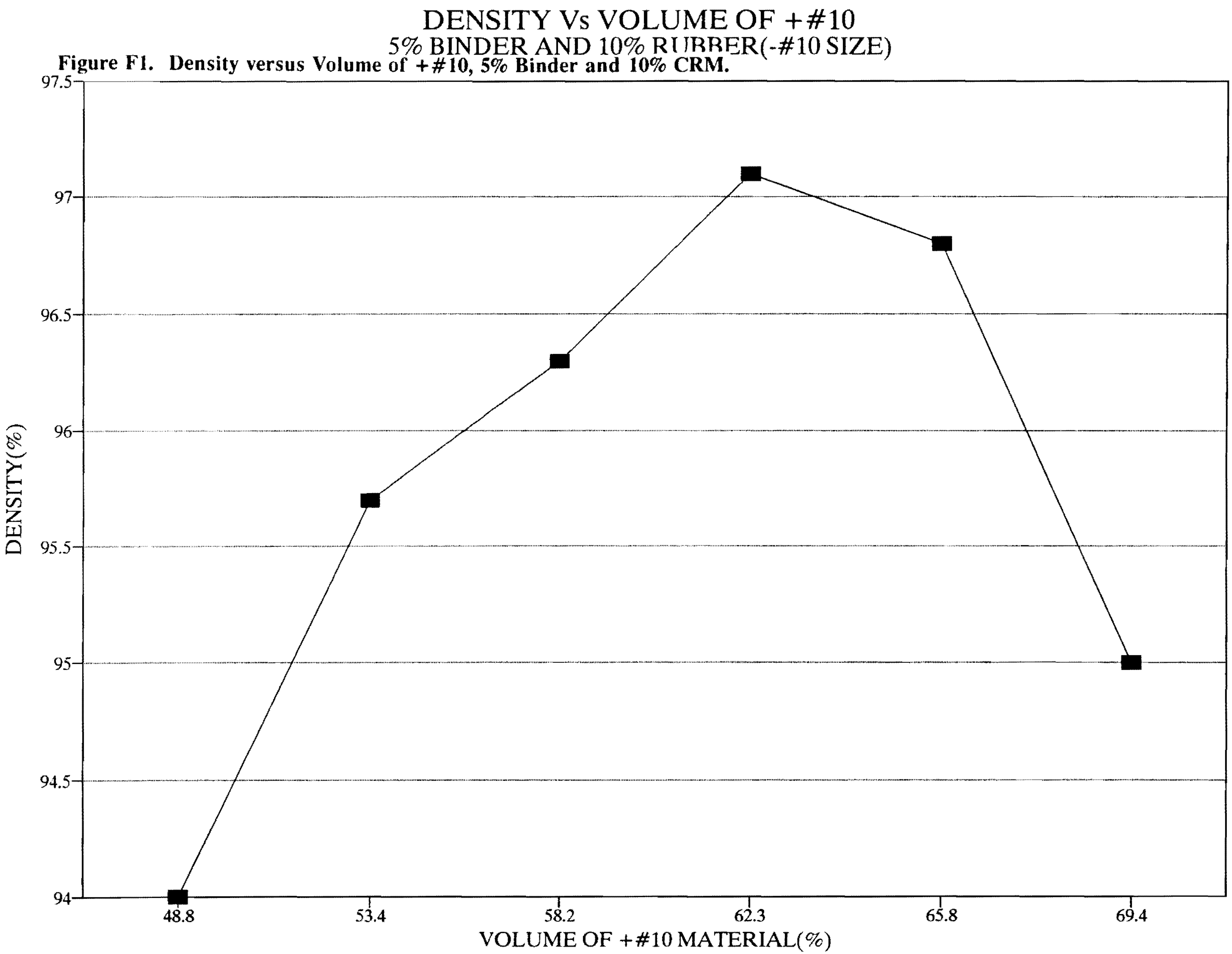


Figure F1. Density versus Volume of +#10, 5% Binder and 10% CRM.

Figure F2. Density versus Volume of +#10, 10% CRM.

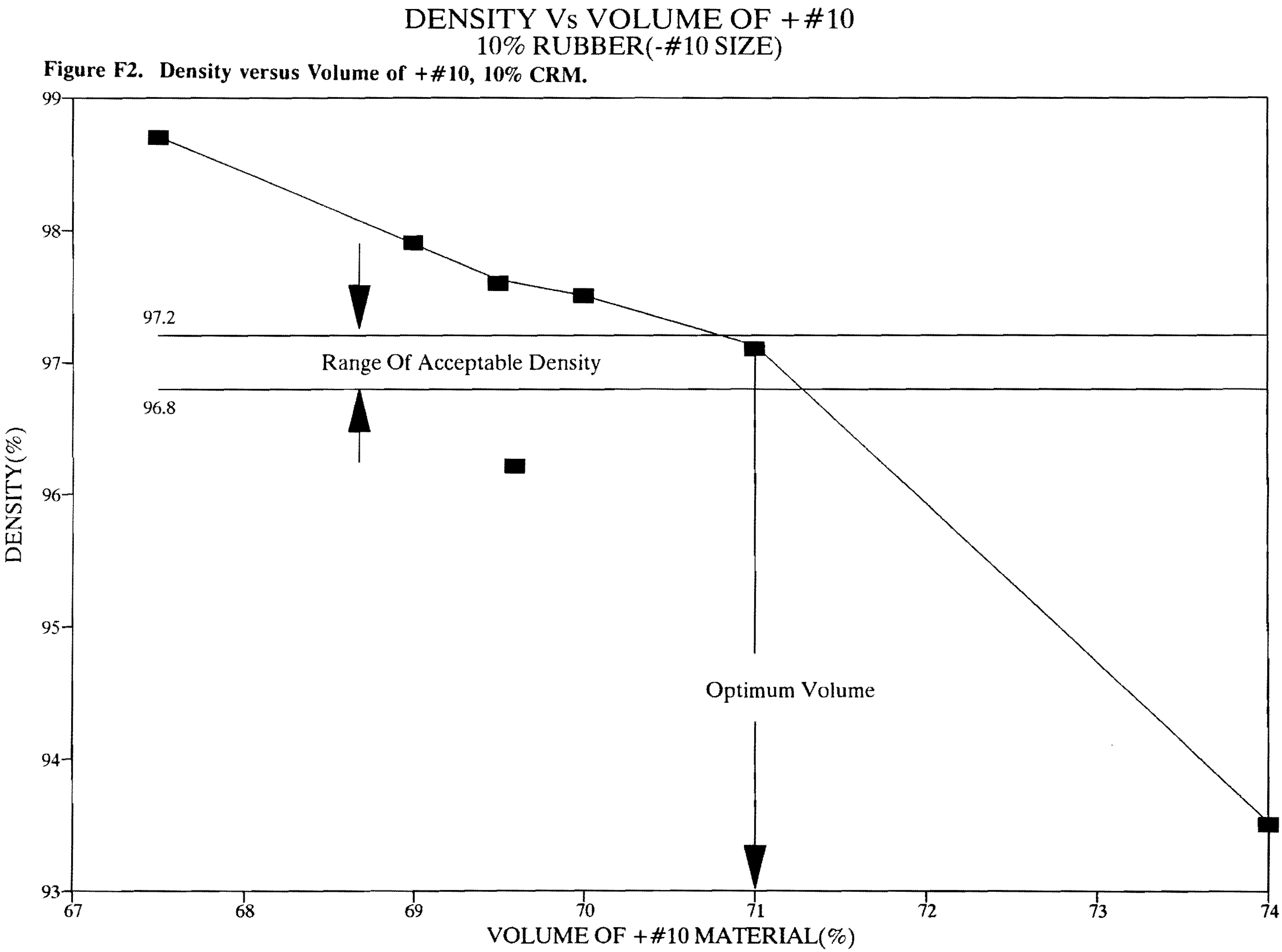


Table F4. Final Gradation For Evaluation Of Mixture Using AAMAS.

Sieve Size Crushed Stone	% Each	Cumulative Mix Weight(Grams)
1/2"	0.0	0.0
3/8"	1.8	79.0
#4	50.2	2251.0
#10	30.1	3554.2
#40	2.3	3653.8
#80	2.6	3768.5
#200	1.7	3840.4
Passing #200	2.2	3936.5
Sand		
1/2"	0.0	3936.5
3/8"	0.0	3936.5
#4	0.0	3936.5
#10	0.0	3936.5
#40	0.0	3936.5
#80	1.0	3981.0
#200	0.3	3992.2
Passing #200	0.2	4000.0

STEP 3: Static creep test was run on the samples with densities 97$\pm$0.2% according to standard test specification TEX-231-F. The results are tabulated as follows.

Table F5. Summary Of The Static Creep Test Data For 10%[1](Passing #80 size Rubber).

Sample#	1	2	3	Average
Air Voids %	2.1	3.1	2.7	2.9
Binder Content[2]	8.2%	8.2%	8.2%	8.2%
Permanent Strain in/in(cm/cm)	5.3×10^{-4}	1.95×10^{-4}	4.1×10^{-4}	3.78×10^{-4}
Slope in/in sec (cm/cm sec)	4.5×10^{-8}	2.5×10^{-8}	2.4×10^{-8}	3.1×10^{-8}
Creep Stiffness psi (Kg/cm^2)	7996 (562.2)	7856 (552.4)	8054 (566.3)	7969 (560.3)

[1] - By weight of the asphalt content
[2] - By weight of the aggregate

PERFORMANCE EVALUATION OF THE MIXTURE USING AAMAS

Table F6. AAMAS Test Results For *Unconditioned Specimens @41°F.*

Rice Specific Gravity	2.311			
Sample#	9	10	11	Average
Bulk Specific Gravity	2.177	2.176	2.175	2.176
Air Voids,%	5.80	5.84	5.88	5.84
Total Resilient Modulus[1], psi (Kg/cm^2)	$10.01x10^5$ (70407)	$10.09x10^5$ (70959)	$8.47x10^5$ (59583)	$9.53x10^5$ (67005)
Indirect Tensile Strength, psi (Kg/cm^2)	94.74 (6.66)	93.13 (6.55)	97.05 (6.82)	94.97 (6.68)
Indirect Tensile Strain @Failure, in/in(cm/cm) (10^{-3})	3.34	2.34	4.33	3.34

[1] - Average of the two Axes

Table F7. AAMAS Test Results For *Unconditioned Specimens @77°F.*

Rice Specific Gravity	2.311			
Sample#	4	13	16	Average
Bulk Specific Gravity	2.177	2.180	2.176	2.178
Air Voids,%	5.80	5.67	5.84	5.77
Total Resilient Modulus[1], psi (Kg/cm^2)	$1.78x10^5$ (12528)	$2.19x10^5$ (15405)	$2.15x10^5$ (15132)	$2.04x10^5$ (14355)
Indirect Tensile Strength, psi (Kg/cm^2)	58.38 (4.10)	63.23 (4.45)	63.28 (4.45)	61.63 (4.33)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	12.01	8.28	8.24	9.51

[1] - Average of the two Axes

Table F8. AAMAS Test Results For *Unconditioned Specimens @104°F.*

Rice Specific Gravity	2.311			
Sample#	6	8	15	Average
Bulk Specific Gravity	2.159	2.178	2.190	2.176
Air Voids,%	6.58	5.76	5.24	5.86
Total Resilient Modulus[1], psi (Kg/cm^2)	63045 (4433)	68965 (4849)	68010 (4782)	66673 (4678)
Indirect Tensile Strength, psi (Kg/cm^2)	16.04 (1.13)	16.42 (1.15)	17.51 (1.23)	16.66 (1.17)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	18.67	17.00	16.94	17.54

[1] - Average of the two Axes

Table F9. AAMAS Test Results For *Moisture Conditioned* Specimens Tested @77°F.

Rice Specific Gravity	2.311			
Sample#	1	2	17	Average
Bulk Specific Gravity	2.170	2.168	2.190	2.176
Air Voids,%	6.10	6.19	5.24	5.84
Degree Of Saturation, %	19.51	32.69	58.91	37.04
Total Resilient Modulus[1], psi (Kg/cm^2)	$1.97x10^5$ (13860)	$1.71x10^5$ (12049)	$2.05x10^5$ (14418)	$1.91x10^5$ (13442)
Indirect Tensile Strength, psi (KG/cm^2)	49.93 (3.52)	58.40 (4.11)	67.42 (4.77)	58.58 (4.12)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	12.59	12.01	11.87	12.16

[1] - Average of the two Axes

Table F10. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-1.**

Rice Specific Gravity	2.311			
Sample#	3	5	7	Average
Bulk Specific Gravity	2.172	2.169	2.164	2.168
Air Voids,%	6.03	6.14	6.30	6.17
Total Resilient Modulus[1], psi (Kg/cm^2)	10.82x10^5 (76100)	10.34x10^5 (72728)	12.03x10^5 (84601)	11.07x10^5 (77810)
Recovery Efficiency	0.48	0.56	0.57	0.54
Indirect Tensile Creep Modulus @3600sec, psi(kg/cm^2)	19746 (1388)	21563 (1516)	26357 (1853)	22555 (1586)

[1] - Average of the two Axes

Table F11. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-2.**

Rice Specific Gravity	2.311			
Sample#	12	14	18	Average
Bulk Specific Gravity	2.185	2.178	2.189	2.184
Air Voids,%	5.50	5.80	5.28	5.50
Total Resilient Modulus[1], psi (Kg/cm^2)	11.13x10^5 (78251)	10.88x10^5 (76468)	11.72x10^5 (82420)	11.24x10^5 (79046)
Indirect Tensile Strength, psi (Kg/cm^2)	116.47 (8.19)	108.56 (7.63)	101.87 (7.16)	108.96 (7.66)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	0.82	1.17	1.22	1.07

[1] - Average of the two Axes

TABLE F12. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-1.

Rice Specific Gravity	2.311			
Sample#	3	4	5	Average
Bulk Specific Gravity	2.30	2.29	2.26	2.28
Air Voids,%	0.48	0.91	2.21	1.20
Total Uniaxial Resilient Modulus, psi(kg/cm^2)	121451 (8539)	98117 (6899)	94864 (6670)	104811 (7369)
Slope Of Compressive Creep Test Curve, b	0.11801	0.03055	0.14618	0.09824
Intercept Of Compressive Creep Test Curve, a	0.00229	0.00474	0.00289	0.00331
Total Permanent Deformation @3600sec, in/in(cm/cm)	0.00604	0.00949	0.00784	0.00777
Compressive Creep Modulus @3600sec, psi(kg/cm^2)	9860 (693)	7536 (530)	6183 (435)	7859 (553)

Table F13. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-2.

Rice Specific Gravity	2.311			
Sample#	1	2	6	Average
Bulk Specific Gravity	2.29	2.27	2.29	2.28
Air Voids,%	0.91	1.77	0.91	1.20
Unconfined Compressive Strength, psi (Kg/cm^2)	188.1 (13.23)	173.1 (12.17)	207.1 (14.56)	189.4 (13.32)
Compressive Strain @Failure, in/in (cm/cm) (10^{-3})	41.9	34.5	34.9	37.1

[1] - Average of the two Axes

Table F14. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-3.

Rice Specific Gravity	2.311		
Sample#	1	2	Average
Bulk Specific Gravity	2.276	2.271	2.274
Air Voids,%	1.54	1.73	1.63
Dynamic Resilient Modulus @200thcycle, psi(kg/cm^2)	146700 (10314)	137300 (9654)	142000 (9984)
Slope Of Repetitive Creep Test Curve, b	0.39633	[1]	0.39633
Intercept Of Repetitive Creep Test Curve, a	0.00079	[1]	0.00079
Total Permanent Deformation @10000sec, in/in(cm/cm)	0.01809	[1]	0.01809

[1] - One of the LVDTs fell off during the experiment, so we had to discard the data

Appendix G

Laboratory Data for 18%CW Mixture

(18% Coarse Rubber, by Weight of Asphalt, via Wet Method)

MIX DESIGN FOR CRUMB RUBBER MODIFIED ASPHALT MIXES USING TxDOT (TEX-232-F) PROCEDURE WET METHOD -#10 SIZE RUBBER @18% BY WEIGHT OF ASPHALT

STEP 1: Trial Gradation Weights For Varying Coarse To Fine Fraction

Batch Weight: 4000g Binder Content: 5%(by weight of aggregate)

Table G1. Gradation for various fractions of +#10 Size to -#10 Size.

Sieve Size	60/40*		65/35		70/30	
Crushed Stone	% Each	Mix Wt.	% Each	Mix Wt.	% Each	Mix Wt.
1/2"	0	0	0	0	0	0
3/8"	1.3	53.1	1.4	57.5	1.5	61.9
#4	36.5	1512.5	39.5	1638.6	42.6	1764.6
#10	22.2	2400	24	2600	25.9	2800
#40	19.4	3175.9	16.6	3263.8	13.8	3351.6
#80	2.1	3259.8	1.8	3335.5	1.5	3411.3
#200	2	3338.4	1.7	3402.8	1.4	3467.2
Passing #200	4.7	3524.4	4.7	3588.8	4.7	3653.2
Sand 1/2"	0	3524.4	0	3588.8	0	3653.2
3/8"	0	3524.4	0	3588.8	0	3653.2
#4	0	3524.4	0	3588.8	0	3653.2
#10	0	3524.4	0	3588.8	0	3653.2
#40	0.5	3545.4	0.4	3606.7	0.4	3668.1
#80	7.1	3828.5	6.1	3848.9	5	3869.4
#200	3.5	3970	3	3970	2.5	3970
Passing #200	0.8	4000	0.8	4000	0.8	4000
Total	100	4000	100	4000	100	4000

* - Coarse to fine fraction (60% material by weight retained on #10 Sieve)

(Table 1 continued......)

Table G1. Continued.

Sieve Size	75/25*		80/20		85/15	
Crushed Stone	% Each	Mix Wt.	% Each	Mix Wt.	% Each	Mix Wt.
1/2"	0	0	0	0	0	0
3/8"	1.7	66.3	1.8	70.8	1.9	75.2
#4	45.6	1890.7	48.6	2016.7	51.7	2142.8
#10	27.7	3000	29.6	3200	31.4	3400
#40	11	3439.5	8.2	3527.4	5.4	3615.3
#80	1.2	3487	0.9	3562.8	0.6	3638.5
#200	1.1	3531.6	0.8	3596	0.5	3660.4
Passing #200	4.7	3717.6	4.7	3782	4.7	3846.4
Sand 1/2"	0	3717.6	0	3782	0	3846.4
3/8"	0	3717.6	0	3782	0	3846.4
#4	0	3717.6	0	3782	0	3846.4
#10	0	3717.6	0	3782	0	3846.4
#40	0.3	3729.5	0.2	3790.8	0.1	3852.2
#80	4.0	3889.8	3.0	3910.3	2.0	3930.7
#200	2.0	3970	1.5	3970	1.0	3970
Passing #200	0.8	4000	0.8	4000	0.8	4000
Total	100	4000	100	4000	100	4000

* - Coarse to fine fraction (75% material by weight retained on #10 Sieve)

Table G2. Summary Of Densities Of The Trial Batches Described In Table G1.

Coarse/ Fine (By Weight)	Wt. of +#10 Material (%)	Volume of +#10 Material (%)	Density(%)			
			Sample #1	Sample #2	Sample #3	Average
60/40	60	47.9	93.1	92.1	93.1	92.8
65/35	65	51.3	89.9	92.2	91.2	91.1
70/30	70	55.9	90.0	91.7	91.7	91.1
75/25	75	61.3	94.1	94.1	93.8	94.0
80/20	80	65.6	92.7	93.7	93.6	93.3
85/15	85	69.8	94.9	94.8	95.1	94.9

STEP 2: The objective of this step is achieve a relative density of 97±0.2% by trial and error method. With the values from Table 2 a graph was plotted with volume of the +#10 material on the X-axis and percent density on the Y-axis. From the graph a gradation that gave the maximum density was selected. To that mix 5% of +#10 size material is added by volume and binder content is increased to 8.2% by weight of the aggregate. The following is the summary of the different trials with the varying volume fraction of the +#10 size material.

Table G3. Summary Of Trials To Achieve 97±0.2% Density.

Trial #	Volume of +#10 material(%)	Density(%)			
		Sample#1	Sample#2	Sample#3	Average
1	66.3	98.8	98.5	98.4	98.6
2	74.0	93.2	93.5	94.6	93.7
3	68.5	96.1	97.0	96.6	96.6
4	68.8	96.2	96.3	95.9	96.1
5	68.0	97.0	96.7	97.1	96.9

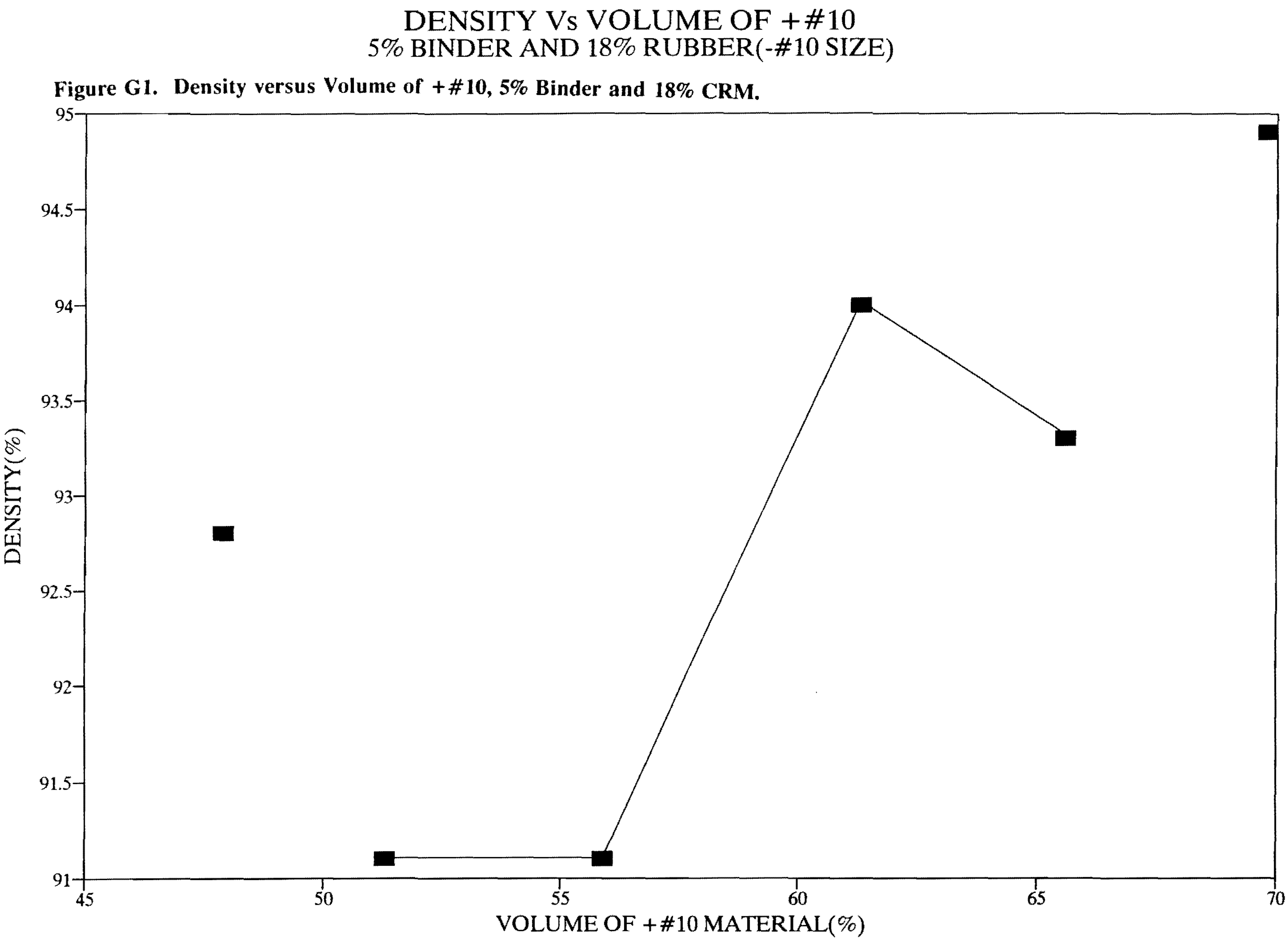


Figure G1. Density versus Volume of +#10, 5% Binder and 18% CRM.

DENSITY Vs VOLUME OF +#10
18% RUBBER(-#10 SIZE)

Figure G2. Density versus Volume of +10, 18% CRM.

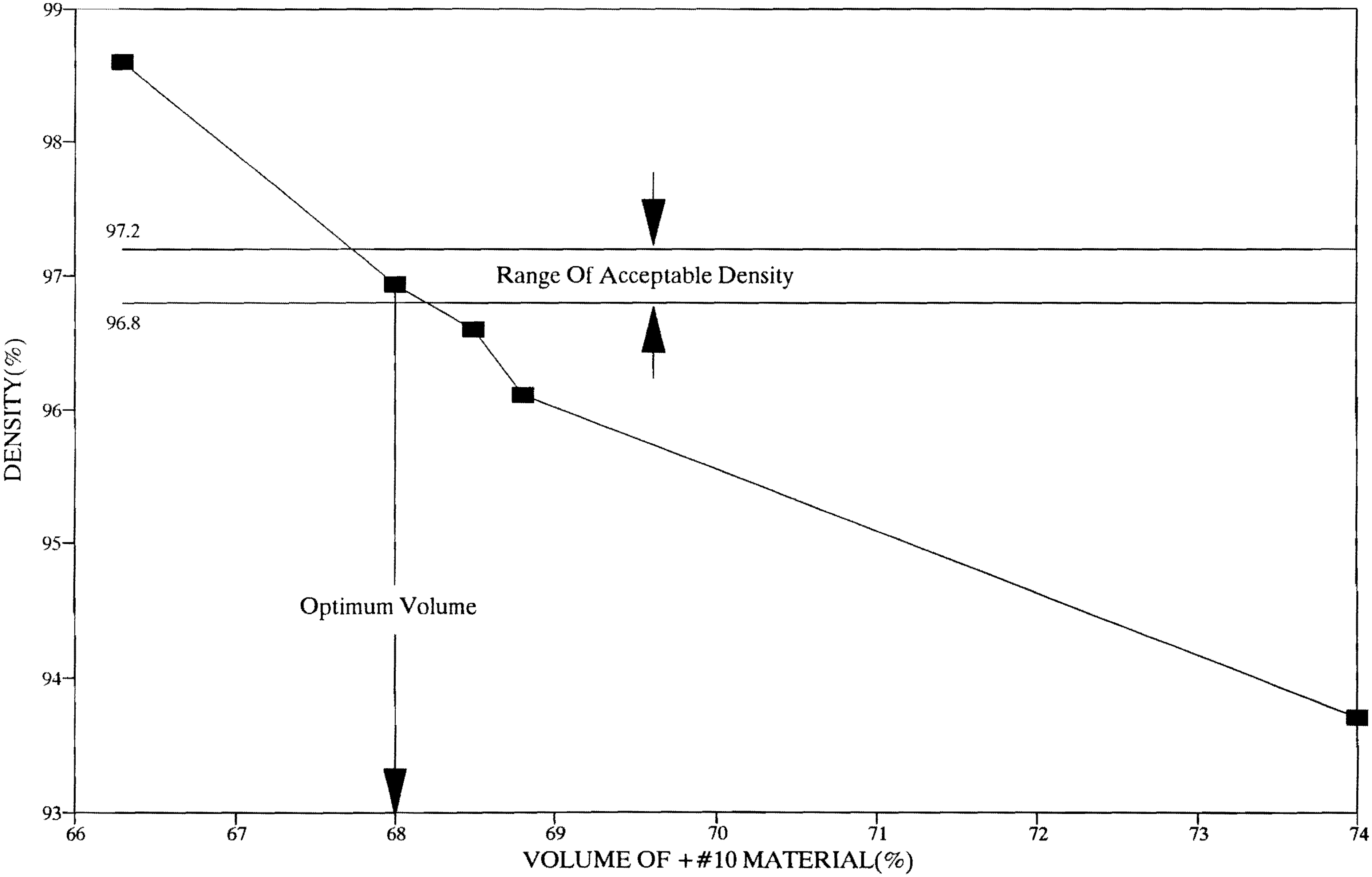


Table G4. Final Gradation For Evaluation Of Mixture Using AAMAS.

Sieve Size	% Each	Cumulative Mix Weight(Grams)
Crushed Stone		
1/2"	0.0	0.0
3/8"	1.7	75.7
#4	48.1	2156.7
#10	28.8	3405.4
#40	3.1	3538.2
#80	3.5	3691.2
#200	2.2	3787.1
Passing #200	3.0	3915.3
Sand		
1/2"	0.0	3915.3
3/8"	0.0	3915.3
#4	0.0	3915.3
#10	0.0	3915.3
#40	0.0	3915.3
#80	1.3	3974.6
#200	0.3	3989.6
Passing #200	0.2	4000.0

STEP 3: Static creep test was run on the samples with densities 97$\pm$0.2% according to standard test specification TEX-231-F. The results are tabulated as follows.

Table G5. Summary Of The Static Creep Test Data For 18%[1](Passing #10 size Rubber).

Sample#	1	2	3	Average
Air Voids %	3.0	3.3	2.9	3.07
Binder Content[2]	8.2%	8.2%	8.2%	8.2%
Permanent Strain in/in(cm/cm)	7.8×10^{-4}	14.2×10^{-4}	2.6×10^{-4}	8.2×10^{-4}
Slope in/in sec (cm/cm sec)	9.2×10^{-8}	17.5×10^{-8}	6.1×10^{-8}	10.9×10^{-8}
Creep Stiffness psi (Kg/cm^2)	5150 (362.1)	5157 (362.6)	4790 (336.8)	5032 (353.8)

[1] - By weight of the asphalt content
[2] - By weight of the aggregate

PERFORMANCE EVALUATION OF THE MIXTURE USING AAMAS

Table G6. AAMAS Test Results For *Unconditioned Specimens @41°F.*

Rice Specific Gravity	2.310			
Sample#	3	14	17	Average
Bulk Specific Gravity	2.180	2.180	2.170	2.177
Air Voids,%	5.63	5.63	6.06	5.77
Total Resilient Modulus[1], psi (Kg/cm^2)	10.78×10^5 (75815)	10.54×10^5 (74086)	10.99×10^5 (77302)	10.77×10^5 (75735)
Indirect Tensile Strength, psi (Kg/cm^2)	81.14 (5.72)	92.01 (6.48)	94.13 (6.62)	89.09 (6.26)
Indirect Tensile Strain @Failure, in/in(cm/cm) (10^{-3})	3.81	3.09	3.06	3.32

[1] - Average of the two Axes

Table G7. AAMAS Test Results For *Unconditioned Specimens @77°F.*

Rice Specific Gravity	2.310			
Sample#	8	10	16	Average
Bulk Specific Gravity	2.180	2.176	2.160	2.172
Air Voids,%	5.63	5.80	6.49	5.97
Total Resilient Modulus[1], psi (Kg/cm^2)	3.38×10^5 (23734)	3.56×10^5 (25041)	3.01×10^5 (21191)	3.32×10^5 (23322)
Indirect Tensile Strength, psi (Kg/cm^2)	79.86 (5.61)	81.35 (5.73)	77.92 (5.48)	79.71 (5.60)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	5.49	4.92	4.40	4.94

[1] - Average of the two Axes

Table G8. AAMAS Test Results For *Unconditioned Specimens @104°F.*

Rice Specific Gravity	2.310			
Sample#	9	12	13	Average
Bulk Specific Gravity	2.186	2.160	2.186	2.177
Air Voids,%	5.37	6.49	5.37	5.74
Total Resilient Modulus[1], psi (Kg/cm^2)	95550 (6718)	111275 (7824)	109507 (7699)	105444 (7414)
Indirect Tensile Strength, psi (Kg/cm^2)	30.52 (2.15)	25.97 (1.83)	29.09 (2.04)	28.53 (2.01)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	11.09	7.94	9.45	9.49

[1] - Average of the two Axes

Table G9. AAMAS Test Results For *Moisture Conditioned* Specimens Tested @77°F.

Rice Specific Gravity	2.310			
Sample#	18	19	20	Average
Bulk Specific Gravity	2.170	2.160	2.189	2.173
Air Voids,%	6.06	6.49	5.24	5.93
Degree Of Saturation, %	32.31	39.99	30.32	34.21
Total Resilient Modulus[1], psi (Kg/cm^2)	$3.03x10^5$ (21335)	$2.97x10^5$ (20849)	$3.50x10^5$ (21581)	$3.17x10^5$ (22255)
Indirect Tensile Strength, psi (KG/cm^2)	75.44 (5.30)	65.09 (4.58)	72.74 (5.11)	71.09 (5.00)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	9.14	6.98	7.42	7.85

[1] - Average of the two Axes

Table G10. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-1.**

Rice Specific Gravity	2.310			
Sample#	7	11	15	Average
Bulk Specific Gravity	2.170	2.190	2.170	2.177
Air Voids,%	6.14	5.24	6.10	5.81
Total Resilient Modulus[1], psi (Kg/cm^2)	$13.79x10^5$ (96973)	$14.69x10^5$ (103273)	$11.07x10^5$ (77359)	$13.16x10^5$ (95535)
Recovery Efficiency	0.64	0.64	0.55	0.61
Indirect Tensile Creep Modulus @3600sec, psi(kg/cm^2)	46549 (3273)	54336 (3820)	22655 (1593)	41180 (2895)

[1] - Average of the two Axes

Table G11. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-2.**

Rice Specific Gravity	2.310			
Sample#	2	4	6	Average
Bulk Specific Gravity	2.190	2.170	2.170	2.177
Air Voids,%	5.24	6.10	6.10	5.81
Total Resilient Modulus[1], psi (Kg/cm^2)	$14.32x10^5$ (100659)	$14.45x10^5$ (101566)	$11.76x10^5$ (82681)	$13.51x10^5$ (94968)
Indirect Tensile Strength, psi (Kg/cm^2)	88.21 (6.20)	88.16 (6.20)	102.97 (7.24)	93.11 (6.55)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	1.47	1.28	1.29	1.35

[1] - Average of the two Axes

Table G12. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-1.

Rice Specific Gravity	2.31		
Sample#	10	6	Average
Bulk Specific Gravity	2.29	2.30	2.295
Air Voids,%	0.90	0.43	0.67
Total Uniaxial Resilient Modulus, psi(kg/cm^2)	117905 (8290)	138349 (9727)	128127 (9009)
Slope Of Compressive Creep Test Curve, b	0.13211	0.22871	0.18041
Intercept Of Compressive Creep Test Curve, a	0.00331	0.00139	0.00235
Total Permanent Strain @3600sec, in/in(cm/cm)	0.00968	0.00902	0.00935
Compressive Creep Modulus @3600sec, psi(kg/cm^2)	6044 (425)	6563 (461)	6304 (443)

Table G13. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-2.

Rice Specific Gravity	2.31			
Sample#	1	3	5	Average
Bulk Specific Gravity	2.27	2.29	2.29	2.28
Air Voids,%	1.73	0.87	0.87	1.15
Unconfined Compressive Strength, psi (Kg/cm^2)	235.7 (16.57)		247.32 (17.40)	241.5 (16.98)
Compressive Strain @Failure, in/in (cm/cm) (10^{-3})	25.7		36.6	31.2

[1] - Average of the two Axes

Table G14. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-3.

Rice Specific Gravity	2.31		
Sample#	1	2	Average
Bulk Specific Gravity	2.292	2.283	2.296
Air Voids,%	0.80	1.20	1.00
Dynamic Resilient Modulus @200thcycle, psi(kg/cm^2)	115900 (8149)	156500 (11004)	136200 (9576)
Slope Of Repetitive Creep Test Curve, b	0.34272	0.36879	0.35576
Intercept Of Repetitive Creep Test Curve, a	0.00132	0.00017	0.00075
Total Permanent Strain @10000cycles, in/in(cm/cm)	0.01619	0.01486	0.01553

Appendix H

Laboratory Data for 18%CD Mixture

(18% Coarse Rubber, by Weight of Asphalt, via Dry Process)

MIX DESIGN FOR CRUMB RUBBER MODIFIED ASPHALT MIXES USING TEXAS DoT (TEX-232-F) PROCEDURE DRY METHOD -#10 SIZE RUBBER @18% BY WEIGHT OF ASPHALT

Table H1. Summary Of Trials To Achieve 97±0.2% Density.

Trial #	Volume of +#10 material(%)	Density(%)			
		Sample#1	Sample#2	Sample#3	Average
1	68.0	95.8	95.0	94.9	95.2
2	63.0	97.7	97.4	97.6	97.6
3	64.3		97.1	97.0	97.1

DENSITY Vs VOLUME OF +#10
18% RUBBER(-#10 SIZE)DRY METHOD

Figure H1. Density versus Volume of +#10.

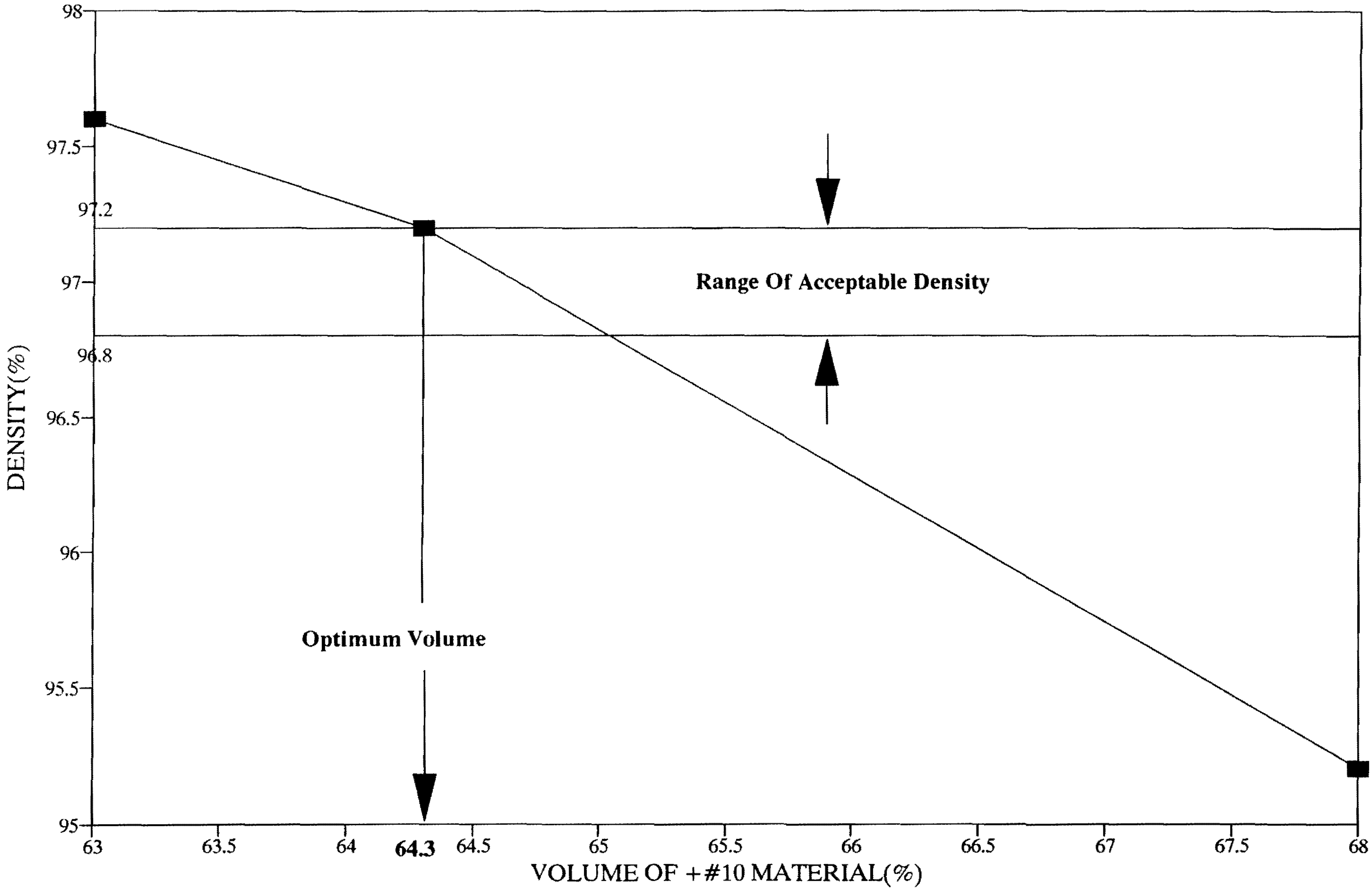


Table H2. Final Gradation For Evaluation Of Mixture Using AAMAS.

Sieve Size Crushed Stone	% Each	Cumulative Mix Weight(Grams)
1/2"	0.0	0.0
3/8"	1.7	71.6
#4	45.5	2040.4
#10	27.3	3221.6
#40	4.0	3395.5
#80	4.6	3595.7
#200	2.9	3721.3
Passing #200	3.9	3889.2
Sand		
1/2"	0.0	3889.2
3/8"	0.0	3889.2
#4	0.0	3889.2
#10	0.0	3889.2
#40	0.0	3889.2
#80	1.7	3966.8
#200	0.5	3986.4
Passing #200	0.3	4000.0

STEP 3: Static creep test was run on the samples with densities 97±0.2% according to standard test specification TEX-231-F. The results are tabulated as follows.

Table H3. Summary Of The Static Creep Test Data For 18%[1](Passing #10 size Rubber).

Sample#	1	2	3	Average
Air Voids %	3.0	3.3	2.9	3.07
AC Content[2]	6.8%	6.8%	6.8%	6.8%
Permanent Strain in/in(cm/cm)	12.9×10^{-4}	7.51×10^{-4}	12.5×10^{-4}	10.9×10^{-4}
Slope in/in sec (cm/cm sec)	19.1×10^{-8}	10.7×10^{-8}	12.8×10^{-8}	14.2×10^{-8}
Creep Stiffness psi (Kg/cm^2)	3659 (257.3)	4584 (322.3)	3633 (255.4)	3959 (278.3)

[1] - By weight of the asphalt content
[2] - By weight of the aggregate

PERFORMANCE EVALUATION OF THE MIXTURE USING AAMAS

Table H4. AAMAS Test Results For *Unconditioned Specimens @41°F.*

Rice Specific Gravity	2.317			
Sample#	6	7	21	Average
Bulk Specific Gravity	2.20	2.20	2.14	2.181
Air Voids,%	5.05	5.10	7.63	5.87
Total Resilient Modulus[1], psi (Kg/cm^2)	12.54×10^5 (88152)	12.50×10^5 (87878)	8.93×10^5 (62813)	11.32×10^5 (79614)
Indirect Tensile Strength, psi (Kg/cm^2)	109.12 (7.67)	115.45 (8.12)	89.72 (6.31)	104.76 (7.37)
Indirect Tensile Strain @Failure, in/in(cm/cm) (10^{-3})	1.98	2.83	2.47	2.43

[1] - Average of the two Axes

Table H5. AAMAS Test Results For *Unconditioned Specimens @77°F.*

Rice Specific Gravity	2.317			
Sample#	18	22	23	Average
Bulk Specific Gravity	2.196	2.178	2.178	2.184
Air Voids,%	5.20	6.00	6.00	5.74
Total Resilient Modulus[1], psi (Kg/cm^2)	3.02×10^5 (21241)	3.07×10^5 (21593)	3.03×10^5 (21288)	3.04×10^5 (21374)
Indirect Tensile Strength, psi (Kg/cm^2)	75.36 (5.30)	70.86 (4.98)	70.96 (4.99)	72.39 (5.10)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	6.70	6.82	5.80	6.44

[1] - Average of the two Axes

Table H6. AAMAS Test Results For *Unconditioned Specimens @104°F.*

Rice Specific Gravity	2.317			
Sample#	8	13	19	Average
Bulk Specific Gravity	2.180	2.200	2.178	2.186
Air Voids,%	5.90	5.00	6.00	5.65
Total Resilient Modulus[1], psi (Kg/cm^2)	114336 (8039)	103944 (7308)	99576 (7002)	105952 (7449)
Indirect Tensile Strength, psi (Kg/cm^2)	28.19 (1.98)	28.92 (2.03)	25.60 (1.80)	28.57 (2.01)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	18.77	12.79	12.73	14.76

[1] - Average of the two Axes

Table H7. AAMAS Test Results For *Moisture Conditioned* Specimens Tested @77°F.

Rice Specific Gravity	2.317			
Sample#	9	15	16	Average
Bulk Specific Gravity	2.176	2.190	2.189	2.185
Air Voids,%	6.10	5.50	5.50	5.70
Degree Of Saturation, %	44.59	49.12	53.01	48.91
Total Resilient Modulus[1], psi (Kg/cm^2)	2.29×10^5 (16127)	2.28×10^5 (16056)	2.17×10^5 (15286)	2.25×10^5 (15823)
Indirect Tensile Strength, psi (KG/cm^2)	68.96 (4.86)	69.07 (4.86)	68.20 (4.80)	68.74 (4.83)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	9.18	9.52	9.92	9.54

[1] - Average of the two Axes

Table H8. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-1.**

Rice Specific Gravity	2.317			
Sample#	10	14	20	Average
Bulk Specific Gravity	2.190	2.186	2.183	2.186
Air Voids,%	5.50	5.70	5.80	5.67
Total Resilient Modulus[1], psi (Kg/cm^2)	$16.00x10^5$ (112476)	$15.57x10^5$ (109445)	$14.07x10^5$ (98909)	$15.21x10^5$ (106943)
Recovery Efficiency	0.67	0.65	0.67	0.66
Indirect Tensile Creep Modulus @3600sec, psi(kg/cm^2)	64070 (4505)	50956 (3583)	50383 (3542)	55136 (3877)

[1] - Average of the two Axes

Table H9. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-2.**

Rice Specific Gravity	2.317			
Sample#	11	12	17	Average
Bulk Specific Gravity	2.180	2.202	2.170	2.184
Air Voids,%	5.80	5.00	6.50	5.74
Total Resilient Modulus[1], psi (Kg/cm^2)	$16.49x10^5$ (115928)	$16.31x10^5$ (114701)	$14.16x10^5$ (99583)	$15.66x10^5$ (110070)
Indirect Tensile Strength, psi (Kg/cm^2)	105.96 (7.45)	115.69 (8.13)	109.32 (7.69)	110.32 (7.76)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	1.41	1.16	0.94	1.14

[1] - Average of the two Axes

Table H10. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-1.

Rice Specific Gravity	2.317			
Sample#	2	4	6	Average
Bulk Specific Gravity	2.261	2.270	2.267	2.266
Air Voids,%	2.4	2.02	2.15	2.20
Total Uniaxial Resilient Modulus, psi(kg/cm^2)	86944 (6113)	106319 (7475)	119129 (8376)	104128 (7321)
Slope Of Compressive Creep Test Curve, b	0.05377	0.03507	0.05694	0.04859
Intercept Of Compressive Creep Test Curve, a	0.00356	0.00388	0.00269	0.00378
Total Permanent Strain @3600sec, in/in(cm/cm)	0.00553	0.00518	0.00433	0.00501
Compressive Creep Modulus @3600sec, psi(kg/cm^2)	10834 (762)	11455 (805)	13659 (960)	11983 (842)

Table H11. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-2.

Rice Specific Gravity	2.317			
Sample#	3	5		Average
Bulk Specific Gravity	2.279	2.250		2.265
Air Voids,%	1.65	2.89		2.27
Unconfined Compressive Strength, psi (Kg/cm^2)	246.3 (17.3)	187.3 (13.2)		231.8 (16.3)
Compressive Strain @Failure, in/in (cm/cm) (10^{-3})	27.9	27.2		27.6

[1] - Average of the two Axes

Table H12 AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-3

Rice Specific Gravity	2.317		
Sample#	1	2	Average
Bulk Specific Gravity	2.262	2.260	2.261
Air Voids,%	2.38	2.54	2.46
Dynamic Resilient Modulus @200thcycle, psi(kg/cm^2)	93280 (6559)	107800 (7579)	100540 (7069)
Slope Of Repetitive Creep Test Curve, b	0.34001	0.31515	0.32758
Intercept Of Repetitive Creep Test Curve, a	0.00127	0.00206	0.00117
Total Permanent Strain @10000cycles, in/in(cm/cm)	0.01662	0.02092	0.01887

Appendix I

Laboratory Data for 18%FD Mixture

(18% Fine Rubber, by Weight of Asphalt, via Dry Process)

MIX DESIGN FOR CRUMB RUBBER MODIFIED ASPHALT MIXES
USING TxDOT (TEX-232-F) PROCEDURE
DRY METHOD -#80 SIZE RUBBER @18% BY WEIGHT OF ASPHALT

Table I1. Summary Of Trials To Achieve 97±0.2% Density.

Trial #	Volume of +#10 material(%)	Density(%)			
		Sample#1	Sample#2	Sample#3	Average
1	74.4	96.5	96.4	95.5	96.1
2	69.0	98.7	97.9	98.4	98.2
3	72.0	97.3	97.9	97.8	97.7
4	73.0	97.4	97.5	96.1	97.0

DENSITY Vs VOLUME OF +#10
18% RUBBER(-#80 SIZE)DRY METHOD

Figure I1. Density versus Volume of +#10.

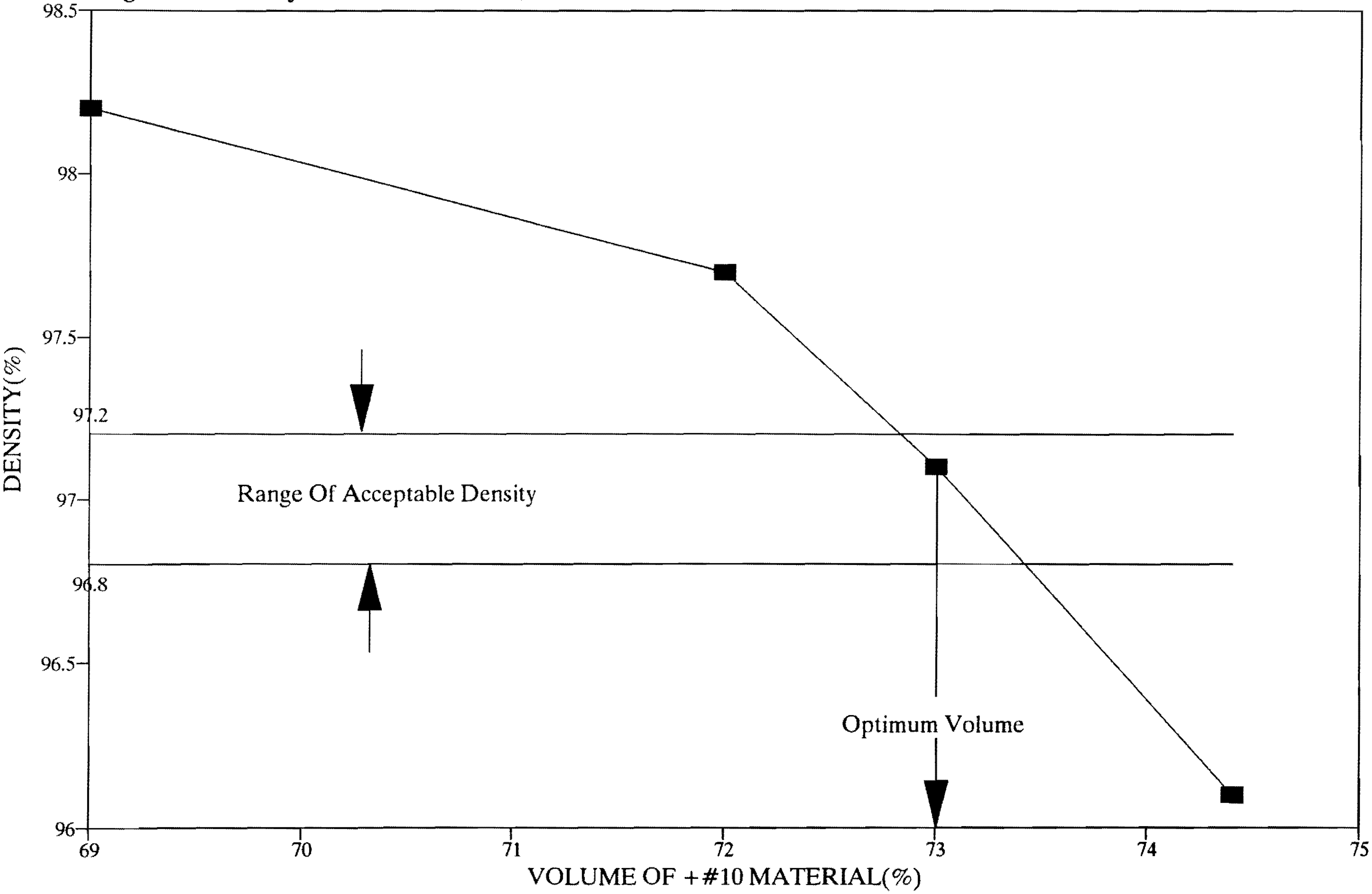


Table I2. Final Gradation For Evaluation Of Mixture Using AAMAS.

Sieve Size Crushed Stone	% Each	Cumulative Mix Weight(Grams)
1/2"	0.0	0.0
3/8"	1.9	81.2
#4	51.6	2313.8
#10	30.9	3653.4
#40	1.8	3730.8
#80	2.1	3820.0
#200	1.3	3875.9
Passing #200	1.7	3950.7
Sand		
1/2"	0.0	3950.7
3/8"	0.0	3950.7
#4	0.0	3950.7
#10	0.0	3950.7
#40	0.0	3950.7
#80	0.8	3985.2
#200	0.2	3993.9
Passing #200	0.1	4000.0

STEP 3: Static creep test was run on the samples with densities 97±0.2% according to standard test specification TEX-231-F. The results are tabulated as follows.

Table 13. Summary Of The Static Creep Test Data For 18%[1](Passing #80 size Rubber).

Sample#	1	2	3	Average
Air Voids %	2.6	2.5	3.9	3.00
AC Content[2]	6.8%	6.8%	6.8%	6.8%
Permanent Strain in/in(cm/cm)	5.1×10^{-4}	7.0×10^{-4}	6.8×10^{-4}	6.3×10^{-4}
Slope in/in sec (cm/cm sec)	7.7×10^{-8}	4.6×10^{-8}	3.3×10^{-8}	5.2×10^{-8}
Creep Stiffness psi (Kg/cm^2)	6093 (428.4)	6472 (455.0)	6222 (437.5)	6262 (440.3)

[1] - By weight of the asphalt content
[2] - By weight of the aggregate

Table I4. AAMAS Test Results For *Unconditioned Specimens @41°F.*

Rice Specific Gravity	2.335			
Sample#	2	3	13	Average
Bulk Specific Gravity	2.159	2.182	2.172	2.171
Air Voids,%	7.54	6.55	6.98	7.02
Total Resilient Modulus[1], psi (Kg/cm^2)	8.53x10^5 (59941)	7.63x10^5 (53616)	7.99x10^5 (56173)	8.05x10^5 (56577)
Indirect Tensile Strength, psi (Kg/cm^2)	71.98 (5.06)	79.68 (5.60)	68.76 (4.83)	73.47 (5.17)
Indirect Tensile Strain @Failure, in/in(cm/cm) (10^{-3})	3.82	2.98	2.39	3.06

[1] - Average of the two Axes

Table I5. AAMAS Test Results For *Unconditioned Specimens @77°F.*

Rice Specific Gravity	2.335			
Sample#	7	12	15	Average
Bulk Specific Gravity	2.166	2.174	2.172	2.171
Air Voids,%	7.24	6.90	6.98	7.04
Total Resilient Modulus[1], psi (Kg/cm^2)	2.02x10^5 (14226)	1.93x10^5 (13564)	1.93x10^5 (13596)	1.96x10^5 (13795)
Indirect Tensile Strength, psi (Kg/cm^2)	62.03 (4.37)	63.64 (4.51)	65.82 (4.63)	63.83 (4.49)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	4.69	7.57	5.44	5.90

[1] - Average of the two Axes

Table 16. AAMAS Test Results For *Unconditioned Specimens @104°F.*

Rice Specific Gravity	2.335			
Sample#	4	14	19	Average
Bulk Specific Gravity	2.165	2.173	2.173	2.170
Air Voids,%	7.28	6.94	6.94	7.05
Total Resilient Modulus[1], psi (Kg/cm^2)	113038 (7948)	99714 (7011)	74715 (5253)	95822 (6737)
Indirect Tensile Strength, psi (Kg/cm^2)	21.54 (1.51)	22.53 (1.58)	18.10 (1.27)	20.72 (1.46)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	9.59	9.16	9.87	9.54

[1] - Average of the two Axes

Table 17. AAMAS Test Results For *Moisture Conditioned* Specimens Tested @77°F.

Rice Specific Gravity	2.335			
Sample#	5	6	8	Average
Bulk Specific Gravity	2.174	2.178	2.161	2.171
Air Voids,%	6.89	6.71	7.45	7.02
Degree Of Saturation, %	29.95	22.59	18.42	23.65
Total Resilient Modulus[1], psi (Kg/cm^2)	$1.61x10^5$ (11291)	$2.00x10^5$ (14067)	$1.80x10^5$ (12629)	$1.80x10^5$ (12662)
Indirect Tensile Strength, psi (KG/cm^2)	50.98 (3.58)	56.33 (3.96)	50.05 (3.52)	52.45 (3.69)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	7.32	6.74	10.10	8.05

[1] - Average of the two Axes

Table 18. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-1.**

Rice Specific Gravity	2.335			
Sample#	11	16	18	Average
Bulk Specific Gravity	2.155	2.181	2.178	2.171
Air Voids,%	7.72	6.57	6.7	7.02
Total Resilient Modulus[1], psi (Kg/cm^2)	$10.47x10^5$ (73591)	$12.09x10^5$ (84975)	$12.79x10^5$ (89917)	$11.78x10^5$ (82828)
Recovery Efficiency	0.52	0.65	0.57	0.58
Indirect Tensile Creep Modulus @3600sec, psi(kg/cm^2)	15249 (1072)	23876 (1679)	27793 (1954)	22306 (1568)

[1] - Average of the two Axes

Table 19. **AAMAS Test Results For *Environmental Aged/Hardened Specimens* Tested @41°F For Set-2.**

Rice Specific Gravity	2.335			
Sample#	9	10	17	Average
Bulk Specific Gravity	2.169	2.167	2.174	2.170
Air Voids,%	7.12	7.20	6.90	7.07
Total Resilient Modulus[1], psi (Kg/cm^2)	$10.04x10^5$ (70580)	$9.93x10^5$ (69800)	$9.19x10^5$ (64641)	$9.72x10^5$ (68340)
Indirect Tensile Strength, psi (Kg/cm^2)	87.47 (6.15)	82.09 (5.77)	81.55 (5.73)	83.70 (5.89)
Indirect Tensile Strain @Failure, in/in (cm/cm) (10^{-3})	1.75	1.72	1.94	1.80

[1] - Average of the two Axes

Table I10. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-1.

Rice Specific Gravity	2.335			
Sample#	6	7	8	Average
Bulk Specific Gravity	2.253	2.244	2.241	2.28
Air Voids,%	3.5	3.88	4.0	1.15
Total Uniaxial Resilient Modulus, psi(kg/cm^2)	113968 (8013)	149224 (10492)	102704 (7221)	121965 (8575)
Slope Of Compressive Creep Test Curve, b	0.02995	[1]	0.11393	0.07194
Intercept Of Compressive Creep Test Curve, a	0.00239	[1]	0.00143	0.00191
Total Permanent Strain @3600sec, in/in(cm/cm)	0.00311		0.00366	0.00338
Compressive Creep Modulus @3600sec, psi(kg/cm^2)	19020 (1337)	11300 (795)[1]	16251 (1143)	17635 (1240)

[1] - One of the LVDT was off the range, had to discard the data

Table I11. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-2.

Rice Specific Gravity	2.335			
Sample#	3	4	5	Average
Bulk Specific Gravity	2.275	2.278	2.24	2.264
Air Voids,%	2.58	2.46	4.00	3.02
Unconfined Compressive Strength, psi (Kg/cm^2)	186.12 (13.09)		246.35 (17.32)	201.24 (14.15)
Compressive Strain @Failure, in/in (cm/cm) (10^{-3})	34.9		39.22	37.02

[1] - Average of the two Axes

Table I12. AAMAS Test Results For *Traffic Densified Samples* Tested @104°F For Set-3.

Rice Specific Gravity	2.335		
Sample#	1	2	Average
Bulk Specific Gravity	2.279	2.286	2.283
Air Voids,%	2.4	2.07	2.24
Dynamic Resilient Modulus @200thcycle, psi(kg/cm^2)	125700 (8838)	146600 (10307)	136150 (9573)
Slope Of Repetitive Creep Test Curve, b	0.41173	0.31818	0.36496
Intercept Of Repetitive Creep Test Curve, a	0.00036	0.00214	0.00120
Total Permanent Strain @10000sec, in/in(cm/cm)	0.00865	0.01797	0.01331

[1] - One of the LVDT was off the range, had to discard the data

Appendix J
Description of AAMAS and Preparation of Samples for Testing

Performance Evaluation

The performance of the mixes that were designed in chapter 3 are evaluated by using Asphalt-Aggregate Mixture Analysis System (AAMAS). AAMAS was developed for designing and evaluating the performance of the asphalt concrete mixes in the laboratory. Two mix design methods namely Hveem and Marshall, with some modifications are commonly used throughout the United States. The design philosophies behind these methodologies are reasonable and served well over the past few decades. These mix design methods were developed several years ago for the traffic volumes and tire pressures representing that period. High traffic volumes and tire pressures increased the need for a system that can not only provide mix design but also evaluate the performance of the mixtures that are designed. AAMAS was developed based on the engineering properties of the mixtures that relate to the performance of the mixes simulating the field conditions.

AAMAS can be broadly divided into two parts. First is the mixture design and the second is the analysis of the mix with the test procedures and evaluating the mix using distress mechanisms. One of the advantages of the AAMAS is that it provides the flexibility of designing the mix using the mix design prescribed in AAMAS or mix design adopted by the agency or the department. The performance evaluation of the mixtures is briefly explained in the following lines.

The evaluation is done by considering performance or the resistance for various forms of distresses the mix encounters in the pavement structure over the design period. There are several analytical models and procedures for designing or evaluating flexible pavements. Most of the models use elastic layer theory or finite element analysis for calculating stresses, strains and deflections etc., for estimating the life of the pavement. After careful evaluation of all these analytical models four forms of distresses were considered for the evaluation of the mixes in conjunction with the pavement structure as a whole. They are fatigue cracking, rutting or permanent deformation, thermal cracking or low temperature cracking and moisture damage. Secondary consideration is given to disintegration, such as ravelling and skid resistance.

The input parameters or properties of the mixes that are considered for evaluation using analytical models are measured by laboratory testing. Five tests were selected for measuring these engineering properties. They are diametral resilient modulus, indirect tensile strength test, gyratory shear strength test and the indirect tensile and uniaxial unconfined compression test. Besides these tests repetitive creep test is added at the request of FHWA officials.

Testing Program

AAMAS testing program can be divided into three distinctive steps.

- Preparation of the samples
- Preconditioning of the samples
- Testing of the samples

Each of these steps are explained in the following pages. Nine different mixes were considered for this testing program and are listed below. Of these three are dense graded mixes and six are crumb rubber modified mixes designed using Tex-232-F.

(1) Type-D control mix (control)

(2) 0.5% Fine rubber by weight of aggregate in control mix (DGF)

(3) 0.5% Coarse rubber by weight of aggregate in control mix (DGC)

(4) 10% Fine rubber by weight of asphalt content - Wet Method (10%FW)

(5) 18% Fine rubber by weight of asphalt content - Wet Method (18%FW)

(6) 10% Coarse rubber by weight of asphalt content - Wet Method (10%CW)

(7) 18% Coarse rubber by weight of asphalt content - Wet Method (18%CW)

(8) 18% Fine rubber by weight of asphalt content - Dry Method (18%FD)

(9) 18% Coarse rubber by weight of asphalt content - Dry Method (18%CW)

Preparation Of The Samples

For the mixes described above, final gradations and optimum asphalt contents were determined in chapter 3 and are used for the preparation of the samples. For each of the mixes described above, twenty seven samples were fabricated for testing. Eighteen of these samples are 4" in diameter and 2" in height. Nine samples are 4" in diameter and 4" in height. The step by step procedures for fabrication of these samples are described below.

(1). Blending of these of these mixes was done in accordance with Test method Tex-205-F for dense graded mixes and necessary modifications for mixing were done for CRM mixes in accordance with Tex-232-F.

(2). After mixing, these mixes were kept in a forced draft oven at approximately 275°F for three hours before molding. This is done to simulate the plant hardening of asphalt and absorption of the asphalt by the aggregates.

(3). Fabrication of the samples was done using a California Kneading compactor. This is because 4 inch samples cannot be molded using Texas gyratory compactor and also because of the nature of the mixes that were considered. It is believed that for open graded mixes, compacting using kneading compactor would simulate the field conditions better than Texas gyratory compactor and Marshall hammer.

(4) All the 2" samples were molded for an air void content of 5 to 8% to simulate the initial condition of the mix immediately after placing in the field. All 2" samples are compacted in a single lift. After compacting the samples are left in the mold for two hours to avoid any rebound of the rubber particles. The target air void content was achieved by trial and error method. For all the mixes 20 tamps @250 psi were applied to accomplish a semi compacted condition so that mix will not be unduly disturbed when the full load is applied. The compaction effort required to achieve 5 to 8% air voids are given in table J.1.

(5). All 4 inch are molded for air voids at refusal. Due to the unavailability of the equipment prescribed in the AAMAS for molding refusal samples in AAMAS was not followed. Due to this it was decided to use the California kneading compactor to mold the refusal samples. A mix which has air voids between 5 to 8% immediately after placing and compacting the mix, due to the wheel loads the mix densifies and reaches air voids of approximately 3%. This reduction in air voids is called as rutting. A mix reaches air voids less than or equal to 3% then the said to be at the refusal. So it was decided to mold all the refusal to less than or equal to 3% air voids. Compaction energies required to achieve the refusal state are approximately same for control, DGF and 18% FD. Compaction energy @refusal is the same for all the wet mixes. Coarse rubber when added dry, required less compaction effort to reach refusal compared to the fine rubber added dry. This is the case when coarse rubber is to the open graded mix or to the dense graded mix. CRM mixes are not only easily compactible but also easily compressible than dense graded mixes. This is because of the higher asphalt contents, openness of the gradation and partly due to the presence of rubber. All 4 inch samples were fabricated in two lifts of two inches. For each lift 20 blows @250 psi were applied to accomplish a semi compacted condition so that mix will not be unduly disturbed when the full load is applied. The compaction required to refusal i.e., less than or equal to 3% air voids are given in table J.2.

Table J.1. Compaction Efforts For Different Mixes To Achieve Air Voids Between 5 and 8%.

Mix	Compaction Effort
1. Control	55 blows @350 psi and a levelling load of 10000 lb
2. DGF	20 blows @325 psi and a levelling load of 9000 lb
3. DGC	20 blows @300 psi and a levelling load of 9000 lb
4. 10% FW	60 blows @325 psi and a levelling load of 10000 lb
5. 18% FW	40 blows @300 psi and a levelling load of 10000 lb
6. 10% CW	50 blows @325 psi and a levelling load of 9000 lb
7. 18% CW	50 blows @325 psi and a levelling load of 9000 lb
8. 18% FD	90 blows @300 psi and a levelling load of 9000 lb
9. 18% CD	20 blows @300 psi and a levelling load of 8500 lb

(6). All the samples were extracted from the molds and were properly labeled.

(7). Bulk specific gravity and air voids were calculated according to the Test Method Tex-207-F.

(8). Thickness and diameter of each sample was determined using a digital Vernier Calipers.

(9). The diametral samples were sorted into six subsets of three samples each so that each subset has approximately equal air voids.

(10). Three subsets were labeled as unconditioned and to be tested at 41°F, 77°F and 104°F respectively. These specimens were kept in plastic bags. The plastic bags were sealed and stored at room temperature. One subset was labeled for moisture conditioning. The remaining two subsets were labeled for environmental aging.

Table J.2. Compaction Effort To Achieve $\leq 3\%$ Air Voids.

Mix	Compaction Effort		
	Lift 1	Lift 2	Levelling Load
1. Control	160 blows @600psi	320 blows @600psi	15000lb
2. DGF	150 blows @600psi	350 blows @600psi	15000lb
3. DGC	100 blows @550psi	180 blows @550psi	10000lb
4. 10% FW	150 blows @550psi	300 blows @550psi	12000lb
5. 18% FW	150 blows @550psi	300 blows @550psi	12000lb
6. 10% CW	150 blows @550psi	300 blows @550psi	12000lb
7. 18% CW	150 blows @550psi	300 blows @550psi	12000lb
8. 18% FD	160 blows @600psi	320 blows @600psi	15000lb
9. 18% CD	75 blows @350psi	125 blows@350psi	10000lb

Preconditioning Of The Test Specimens

Three subsets were preconditioned to simulate the field conditions of aging of asphalt and damage induced due to moisture and conditioning of samples. These two are explained here briefly in the following paragraphs.

Moisture Conditioning

Moisture conditioning is done to simulate the damage to the mix due to the moisture over the life of the asphalt concrete mix. The presence of moisture in the mix leads to stripping of the aggregate and adversely affects the strength characteristics of the mix. Three diametral samples (2 inch samples) or one subset was used for the moisture damage

evaluation. The laboratory simulation of long term(10 to 20 years) effect is described here.
1. Three specimens were placed on individual spacers in a vacuum container. The container was filled with distilled water at room temperature water and such that the specimens have at least one centimeter water above them. A vacuum of 26 inches was applied to the specimens for 15 minutes and specimens were left in the container for 30 minutes.
2. Bulk specific gravity was measured according to the Test Method Tex-207-F. Saturated surface dry bulk specific gravity was calculated and compared with specific gravity before applying vacuum. The specimens were immediately returned to the vacuum container.
3. The degree of saturation was determined by comparing the volume of absorbed water with the volume of the air voids before applying vacuum. Vacuum was applied to the specimens two more times if they did not reach a saturation value of 55%. After three attempts if the specimens did not reach a saturation value of 55% then the specimens were tightly covered with a plastic film and were placed in a plastic bag containing 0.3 oz of water and the plastic bag was sealed. The saturation values for all the mixes are shown in table J.3
4. The plastic bag was kept in a freezer @0 ± 5.4°F for 16 hours. After 16 hours the specimens were transferred into a water bath @140±1.8°F for 24 hours. After 24 hours the specimens were removed and were placed in a water bath already at 77°F for 2 hours. Then the test specimens were tested for resilient modulus and indirect tensile strength and strain.

Table J.3 Degree Of Saturation.

Mix	Degree Of Saturation (%)
1. Control	59.63
2. DGF	47.68
3. DGC	45.92
4. 10% FW	35.85
5. 18% FW	69.03
6. 10% CW	37.04
7. 18% CW	34.21
8. 18% FD	23.65
9. 18% CD	48.91

Temperature Conditioning (Accelerated Aging)

Asphaltic concrete mixture properties are time temperature dependent. Asphalt reacts with atmospheric oxygen which stiffens or hardens the asphalt. This is a very slow process, but as temperature increases the reaction rate increases exponentially. Aging of the asphalt alters the physical properties of the mix considerably. However at lower temperatures if the mix is too stiff the stresses induced due to temperature differential will be released through the development of the crack. This condition is undesirable. Two subsets were temperature conditioned. The laboratory method for simulating long term environmental and temperature loading is described below.

Two subsets of three diametral specimens were placed in the forced draft oven set at temperature of 140°F. These specimens were heated for approximately 48 hours. After initial aging, the temperature of the forced draft oven was elevated to 225°F. These specimens were aged for 5 more days. After 5 days six specimens were placed in temperature cabinet set at temperature 41°F and were stored for 12 hours prior to testing.

Testing of the Samples

Five tests were selected as tools for measuring the properties of the mix. The conditioned and unconditioned samples were tested as follows

Unconditioned Diametral Specimens

Three sets of three specimens each were tested at three different temperatures. They are 41°F, 77°F, and 104°F. These specimens were kept at their respective test temperatures 12 hours prior to the testing. Resilient modulus test (a nondestructive test) was performed on these specimens. Resilient Modulus test was conducted according to the Standard Test Method ASTM D4123. Total resilient modulus is the average of the two axes perpendicular to each other of the same sample. Indirect tensile test was conducted on the axis which was found to have higher resilient deformation. Indirect tensile test was conducted on the same specimens at their respective temperatures at a loading rate of 2 in/min. All the values of these two tests were tabulated in Appendixes A through I and results were discussed in Chapter 4.

Moisture Conditioned Specimens

Resilient modulus was performed on the samples that were moisture conditioned as described previously. This test was performed according to standard test method ASTM D4123 at test temperature 77°F. Total resilient modulus and total and instantaneous resilient deformations were recorded. The indirect tensile test was conducted as described previously. All the results were tabulated in Appendix A through I and the results were discussed in chapter 4. Moisture damage evaluation was done by comparing indirect tensile strains before and after moisture conditioning.

Temperature Conditioned Samples

Resilient modulus and Indirect tension were conducted according to the standard procedures described previously for the first subset of temperature conditioned samples. These tests were performed at 41°F. The results were tabulated in appendix and are discussed in chapter 4. For the second subset, a indirect tensile creep was conducted at 41°F. The scope of this test is to find the creep recovery efficiency of the specimens after a fixed duration after applying the static load. The creep recovery efficiency values were tabulated and discussed in chapter 4.

Traffic Densified Specimens

Three subsets of traffic densified samples were tested. Unconfined compression test was performed on subset one at test temperature 104°F. The unconfined compressive strength and compressive strain at failure were recorded and tabulated. These results are discussed in chapter 4. A uniaxial resilient modulus test was performed on the specimens in subset two at 104°F. After determining the resilient modulus, a uniaxial unconfined compressive static creep test was performed at 104°F. A stress level of 60 psi was used for all the mixes. Creep stiffness and creep strain were calculated at various time intervals during the test. This test is valuable tool for evaluating the rutting or permanent deformation characteristics of the mix. The results are tabulated and discussed in detail in chapter 4. A dynamic loading creep test was conducted on the third subset. This test was also performed at 104°F. In order to compare the results dynamic creep test was also performed at stress level of 60 psi. All the results are tabulated in Appendix A through I. A detailed discussion and analysis is done in chapter 4.