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16. Abstract  This is the fourth report for research project 0-1250, <i>The South Central Superpave Center</i> . This study was initiated and sponsored by the Texas Department of Transportation (TxDOT) to evaluate the uniaxial static creep test for Superpave mixes. The research period for the project was scheduled for 1 year, from September 1998 through August 1999. This report presents the results, findings, conclusions, and recommendations based on the laboratory study and testing of asphalt mixtures.			
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**DEVELOPMENT AND USE OF STATIC CREEP TEST TO  
EVALUATE RUT RESISTANCE OF SUPERPAVE MIXES**

by

Weng On Tam, Mansour Solaimanian, and Thomas W. Kennedy

Research Report Number 1250-4

Research Project 0-1250

The South Central Superpave Center

Conducted for the

**TEXAS DEPARTMENT OF TRANSPORTATION**

in cooperation with the

**U.S. DEPARTMENT OF TRANSPORTATION**

**FEDERAL HIGHWAY ADMINISTRATION**

by the

**CENTER FOR TRANSPORTATION RESEARCH**

Bureau of Engineering Research

**THE UNIVERSITY OF TEXAS AT AUSTIN**

January 2000



## **IMPLEMENTATION RECOMMENDATIONS**

Because of the lack of practical differences between mixes, no attempt was made to develop acceptance criteria. However, it is believed that the work done in this project will contribute to the growing knowledge and experience on static creep testing as a means of evaluating rut resistance of asphalt concrete mixes.

In implementing the results of this research effort, adequate care must be taken to identify the context of the recommendations and adapt them to each unique set of circumstances. In order to extend the applicability of these conclusions, additional research is required to verify the findings for a broader range of materials (asphalt and aggregates). Only then can the findings be applied in a broader sense.

One underlying comment throughout the discussion of results and the conclusions is the absence of practical differences in the results, even in the presence of statistical differences. One concern in the current test setup is the fairly large variability mentioned in Chapter 6. With a rather large variation between the three replicates, it is hard to say that one mean is very different from another. One possible solution would be to increase the stress level at which the test is run. Some other research efforts, such as the Asphalt Aggregate Mixture Analysis System (AAMAS), have used a stress level of 207 kPa, and this stress level should be investigated in the future.

## **DISCLAIMERS**

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 views or policies of either the Federal Highway Administration (FHWA) or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation.

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Research performed in cooperation with TxDOT and the U.S. Department of Transportation,  
Federal Highway Administration.

## **PREFACE**

This is the fourth report for research project 0-1250, *The South Central Superpave Center*. This study was initiated and sponsored by the Texas Department of Transportation (TxDOT) to evaluate the uniaxial static creep test for Superpave mixes. The research period for the project was scheduled for one year, from September 1998 through August 1999. This report presents the results, findings, conclusions, and recommendations based on the laboratory study and testing of asphalt mixtures.

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## SUMMARY

From October 1987 through March 1993, a \$50 million strategic highway research program (SHRP) project was conducted to develop new ways to specify, test, and design asphalt paving materials. The results of this research effort are collectively referred to as *Superpave*.

Currently, the Level I Superpave mix design procedure is used to design asphalt concrete mixes. This procedure is a volumetric mix design procedure. Though the use of Superpave mix design procedures is becoming more and more common, it has always been felt that there was a need for a strength test to validate the volumetric mix design procedure. A good strength test would serve to “calm the fears” of concerned agencies and contractors.

The static creep test is one such test that could be used to validate the Superpave volumetric mix design procedures. The overall objective of this research effort is to investigate the sensitivity of the static creep test to changes in various mix parameters. These parameters are the specimen diameter, specimen preparation method (molded versus saw-cut), asphalt content, and gradation. In addition, the Superpave shear tester (SST) will be evaluated to determine if it can also be used to run this static creep test. If possible, some guidelines will be presented to assist in the development of acceptance criteria for Superpave mixes.

For the study on the effect of diameter and sample preparation, only the effect of slope is of significance from a practical standpoint. However, there were practical differences in the slope between the two material testing devices. There was also a trend of decreasing stiffness, increasing permanent strain, and decreasing slope with increasing asphalt content at 40 °C (104 °F). However, most of the differences were not of practical significance. The results were less consistent at 64 °C (147 °F).

Because of the lack of practical differences between mixes, no attempt was made to develop acceptance criteria. However, it is believed that the work done in this project will contribute to the growing knowledge and experience on static creep testing as a means of evaluating rut resistance of asphalt concrete mixes.



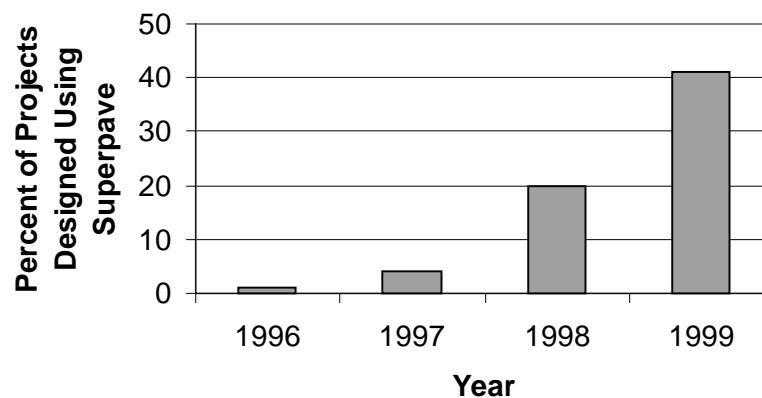
## CHAPTER 1. INTRODUCTION

### 1.1. BACKGROUND

In 1987, Congress established the Strategic Highway Research Program (SHRP) to sponsor several coordinated research projects that were directed at improving the performance and durability of roads in the United States. From October 1987 through March 1993, one \$50 million SHRP project was conducted to develop new ways to specify, test, and design asphalt paving materials. The results of this research effort are collectively referred to as *Superpave* (Kennedy et al. 1994). A product that resulted from this project is the Superpave shear tester (SST). This material testing device is discussed in greater detail in Chapter 3.

The past four years have seen a steady increase in the use of Superpave technology. Forty-seven agencies expect to fully implement the Superpave binder specification by 2000, with two planning to complete implementation after 2000 and three with undetermined plans. Meanwhile, twenty-eight agencies are on target to fully implement the volumetric mix design procedures by 2000. Of the remaining survey respondents, twelve plan to implement Superpave mix designs after 2000, and twelve have undetermined plans (Dunn 1999).

The percentages of hot-mix asphalt projects designed using the Superpave system over the past four paving seasons are shown in Figure 1.1. From 1996 to 1999, the percent of projects designed using the Superpave system increased from 1 percent to 41 percent. Based on the survey, the Superpave lead states team expects that by 2001, 82 percent of all hot-mix asphalt projects will be designed using the Superpave procedures (Dunn 1999).



*Figure 1.1: Proportion of Hot-Mix Asphalt Projects Designed Using the Superpave System (Dunn, 1999)*

## **1.2 MOTIVATION**

Currently, Level I Superpave mix design procedures are used to design asphalt concrete mixes. This level is a volumetric mix design procedure (Cominsky et al. 1994). Although use of the Superpave mix design procedures is becoming more and more common, it has always been felt that there is a need for a strength test to validate the effectiveness of the volumetric mix design procedure. It has been suggested that volumetric properties alone may not be adequate by themselves to ensure good performance for high-volume roads (Brown et al. 1999). A strength test would serve to “calm the fears” of agencies and contractors who are accustomed to the Marshall and Hveem stability tests from the Marshall and Hveem mix design procedures (Roberts et al. 1994).

The Texas Department of Transportation (TxDOT) uses the static creep test as an integral part of its mix design for a special asphalt mixture called coarse matrix high binder (CMHB). The test results are compared with pass/fail criteria involving creep stiffness, permanent strain, and slope of the creep curve. This static creep test is one test that could possibly be used to validate the effectiveness of the Superpave volumetric mix design procedures.

TxDOT is investigating the possibility of using the static creep test as a strength test for Superpave mixes. To accomplish this objective, the South Central Superpave Center conducted a research project to investigate the static creep behavior of Superpave mixes. The Superpave shear tester (SST) was evaluated as well to determine if it could produce test results similar to those of TxDOT’s static creep machine (an EnduraTEC Systems [ETS] device) and also if the SST could be used for running this test.

## **1.3 OBJECTIVE AND METHODOLOGY**

The overall objective of this research effort was to evaluate the possibility of using a static creep test similar to that currently used by TxDOT to help differentiate between good-performing and poor-performing Superpave mixes. To achieve this objective, TxDOT’s current static creep test setup was modified to accommodate Superpave gyratory specimens. In addition, the following factors were evaluated:

- the effect of specimen diameter and compaction method (Phase I),
- the effect of using two different machines, TxDOT’s current static creep tester and the SST (Phase I),
- the effect of sample preparation method (molded to the correct height as compared to saw-cutting a regular Superpave gyratory specimen) (Phase II), and
- the effect of temperature and asphalt content (Phase III).

### ***1.3.1 Phase I: Modifications to TxDOT's Static Creep Test and the Superpave Shear Tester***

TxDOT's static creep tester was designed to test 100 mm (3.9 in.) test specimens compacted using the Texas gyratory compactor and needed to be modified to test specimens compacted using the Superpave gyratory compactor (SGC). On the other hand, the SST was designed to test 150 mm (5.9 in.) test specimens compacted using the SGC. However, the built-in software in the SST was developed to run a range of Superpave performance tests, but not a uniaxial static creep test. Accordingly, modifications were made to the current software to develop the capability to run such a test on the SST. All these modifications were carried out as Phase I of this research effort.

### ***1.3.2 Phase II: Evaluating the Effect of Sample Diameters and Testing Machines***

One concern with using the static creep test on Superpave gyratory-compacted samples is the differences in compaction machines and specimen diameters. In this research effort, the effect of diameter and the effect of compaction method were not considered separately. Their individual effects were confounded. The reason for this decision lies in the fact that production models of the Texas gyratory compactor (TGC) can compact only 100 mm (3.9 in.) diameter specimens, and production models of the Superpave gyratory compactor (in its regular setup) can compact only 150 mm (5.9 in.) diameter specimens. It is highly unlikely that either of these gyratory compactors will be used widely to compact specimens to a different diameter on a production scale. The cost associated with acquiring modified equipment to investigate these two factors separately was not justifiable. Henceforth, whenever the effect of diameter is discussed, it should be noted that the 100 mm (3.9 in.) diameter specimens were compacted using the TGC and that the 150 mm (5.9 in.) specimens were compacted using the SGC.

However, it is important to recognize that there are differences in these two compaction methods. Besides the difference in the specimen diameter, the pressure applied during compaction is also different. In the Texas gyratory compactor, an initial pressure of 345 kPa (50 psi) is applied for most of the compaction process, but compaction ends with a one-time final pressure of 17,238 kPa (2,500 psi) to level the specimen before the pressure is released and the specimen extracted. The TGC uses a 5.8 degree angle of gyration. This procedure is outlined in Texas Test Method Tex-206-F (Texas Department of Transportation 1991). The SGC uses a constant pressure of 600 kPa (87 psi), a 1.25 angle of gyration, and a gyration rate of 30 gyrations per minute. This procedure is outlined in the AASHTO TP4, PP19 procedure (American Association of State Highway and Transportation Officials 1998).

In addition, the static creep test was also run on the SST to evaluate the possibility of running this static creep test on a similar materials testing device.

### ***1.3.3 Phase III: Evaluating the Effect of Sample Preparation Methods***

During SHRP, several test methods were developed for different performance tests. Specimens tested on the SST and the IDT (indirect tension tester, another performance-testing machine developed during SHRP) were saw-cut from the midsection of a regular Superpave gyratory-compacted specimen. These saw-cut specimens were approximately 50 mm (2.0 in.) in height.

In recent years, some members of the asphalt industry have started to compact specimens to a height of 50 mm (1.96 in.) in the SGC, instead of compacting specimens in the traditional manner [to approximately 200 mm (7.9 in.)] and then saw-cutting out the midsection of that specimen. This method of preparing specimens reduces material waste and leads to substantial savings in sample preparation time (especially in the sieving of aggregates). In this document, test specimens prepared in this manner are referred to as *molded* specimens.

Currently, work is also underway at the Asphalt Institute to evaluate the effect of sample preparation on the results of two commonly used performance tests run on the SST. They are the frequency sweep and the repeated shear at constant height tests. There are no available published results from this study at this time.

In order to conform with the specimen height used in TxDOT's procedure, the saw-cut specimens used in this research effort were cut to a height of approximately 51 mm (2.0 in.), instead of 50 mm (1.96), and henceforth are referred to as *cut* specimens.

### ***1.3.4 Phase IV: Evaluating the Effect of Temperature and Asphalt Content***

With the evolution of SHRP and the development of the Superpave system, there has been a move towards testing asphalt mixes at temperatures that are likely to be experienced in the field. This clearly can be seen in the new asphalt binder grading system — performance grading. The Superpave performance grading system requires asphalt binders to be tested at temperatures to which the binder will be exposed in the field. For example, in central Texas, the average 7-day high temperature is 64 °C (147.2 °F) to 70 °C (158 °F). Consequently, asphalt binders used in central Texas are tested at 64 °C or 70 °C to determine if they are adequate for use in this region. However, many creep tests, such as the static creep test, are run at 40 °C (104 °F) to determine the potential for rutting. In an effort to evaluate the effect of running the mix tests at pavement temperatures, the static creep test was run at both 40 °C and 64 °C.

In order to evaluate the effect of asphalt content, three mixes were compacted with varying asphalt contents so as to determine the sensitivity of these mixes to asphalt content changes.

### ***1.3.5 Phases V and VI: Evaluation of Good versus Poor Mixes***

In order to develop acceptance criteria for Superpave mixes, it is important to evaluate a few good-performing and poor-performing pavements in the field. With the help

of TxDOT materials engineers in the Austin, San Antonio, New Braunfels, and Waco Districts, the mix design of four good-performing pavements was selected to represent good mix designs.

Mixes that were not performing well in the field were not selected to represent poor-performing mixes. A pavement's inadequate performance could be a result of poor underlying layers, poor construction practices, or to a number of other factors. Therefore, poor-performing pavements were not selected to represent poor mix designs. Instead, good-performing mix designs were modified to create the poor-performing mixes. This modification was achieved by compacting the laboratory specimens with 1 percent more asphalt than optimum. Having 1 percent more asphalt over optimum is believed to lead to a significantly greater susceptibility to rutting.

## **1.4 CONTRIBUTIONS TO INDUSTRY**

The main contribution to the asphalt paving industry is the development of a static creep test for Superpave mixes. In addition, evaluation of the effect of compaction method and specimen diameter on static creep results will help TxDOT determine the applicability of its CMHB acceptance criteria to Superpave mixtures. The evaluation of the SST will assist state agencies and research institutions to evaluate the SST's ability to conduct a comparable static creep test. The evaluation of temperatures will also be useful in determining if a test at 64 °C (147.2 °F), instead of at 40 °C (104 °F), will better differentiate between a good-performing and poor-performing mix. Last, but not least, the evaluation of the sample preparation method (molded to the correct height as compared to saw-cutting a regular Superpave gyratory specimen) will help determine if the time and materials savings in using molded specimens accurately replicate results obtained from regular specimens that have been saw-cut. It should be noted that while saw-cut Superpave gyratory specimens have traditionally been tested on the SST, the practice does not mean that this sample preparation method is superior to molding specimens to the correct height using the SGC.

## **1.5 SCOPE**

This research project focused on evaluating various factors that could influence static creep results, as mentioned in Section 1.3. This report is organized as follows:

Chapter 1 discusses issues surrounding the need for a strength test to validate Superpave Level I mix design procedures. A brief overview of the objectives and the research methodology is given. The research effort evaluated four factors considered in developing acceptance criteria. They are diameter and compaction method, testing machines, sample preparation method, and temperature and asphalt content.

Chapter 2 provides a theoretical overview of the creep behavior of asphalt concrete mixes and discusses current static creep test criteria. First, the strain rate equation and the generalized form used to illustrate creep in asphalt concrete mixes are discussed. Second, material properties evaluated in creep tests and available creep test criteria are reviewed.

Chapter 3 discusses the development, current test procedures, and acceptance criteria for TxDOT's static creep test. It also discusses modifications made to TxDOT's static creep machine and the SST in order to achieve the objectives of this research effort.

Chapter 4 explains why the mix designs and materials used in this research effort were chosen. This explanation includes a presentation of the material properties and how the mixes were selected for this study.

Chapter 5 shows the experiment designs for each part of this research effort. It also discusses the proposed approach to developing static creep acceptance criteria. Chapter 6 discusses the results from Phase II through Phase V of this research effort. Finally, Chapter 7 presents the conclusions from this research effort and makes recommendations for developing acceptance criteria for Superpave mixes.

## CHAPTER 2. BACKGROUND

### 2.1 CREEP BEHAVIOR OF ASPHALT CONCRETE MIXES

Creep is the time-dependent deformation resulting from a constant applied stress. The deformation of soil involves time-dependent rearrangement of materials. Therefore creep in soil and other particulate matter can be explained as a rate process. The basis of the rate process theory is that atoms, molecules, and particles participating in a time-dependent flow process, called *flow units*, are constrained from relative movement by energy barriers separating equilibrium positions. The displacement of flow units to new positions requires the introduction of sufficient activation energy to overcome the barrier (Mitchell 1976).

#### 2.1.1 Strain Rate Equation

Of the total number of activated flow units at any instant, some may have sufficient energy to overcome the energy barrier and pass over to the next equilibrium position. Others, with insufficient activation energy to surmount the barrier, return to their original position. For each unit that successfully crosses the barrier, there will be a displacement. The component of this displacement in a given direction, multiplied by the number of successful jumps per unit time, gives the rate of movement per unit time. Strain rate is this movement expressed as a displacement per unit time. Mitchell (1976) explains that this shear rate in a particular medium is influenced by a number of factors, as explained by the equation:

$$\varepsilon = 2X (kT/h) \exp(-\Delta F/RT) \sinh(f\lambda/2kT) \quad (\text{Eq 2.1})$$

where	$\varepsilon$	=	strain rate
	$\Delta F$	=	activation energy,
	$T$	=	absolute temperature (°K),
	$k$	=	Boltzman constant,
	$h$	=	Planck constant,
	$f$	=	force,
	$\lambda$	=	distance between successive equilibrium positions,
	$X$	=	proportion of successful barrier crossings, and
	$R$	=	universal gas constant.

Equation 2.1 shows that the strain rate is directly proportional to temperature. As temperature increases, the rate of strain increases. If  $(f\lambda/2kT) < 1$ , then  $\sinh(f\lambda/2kT) \approx (f\lambda/2kT)$ , and rate is directly proportional to force,  $f$ . This is the case for ordinary Newtonian fluids. If  $(f\lambda/2kT) > 1$ , Equation 2.1 is a reasonable first approximation of the rate process that explains creep of asphalt concrete mixtures.

Hill (1973) presented a pseudo-theoretical physical model that attempted to explain the relative displacement of a pair of adjacent aggregate particles in a viscoelastic matrix. He assumed:

- a thin binder film and no hydrostatic effects in the binder,
- deformation is solely due to shear in the binder and volume change is negligible, and
- under the influence of a constant stress, the rate of relative displacement of a pair of adjacent particles is determined by the decreasing thickness of the binder film and by the viscosity of the binder.

Based on these assumptions, Hill characterized the strain in asphalt mixtures using (Equation 2.2),

$$\epsilon_{\text{mix}}/F_y = 2^q [(1 + \sigma t/3\eta F_x)^{1/2q} - 1] \quad (\text{Eq 2.2})$$

where  $1/F_x$  and  $1/F_y$  are constants,  $q$  is an integer greater than 1 corresponding to the number of “Chinese boxes” used in the model determined by the aggregate gradation, and  $\sigma t/3\eta F_x$  is the viscous component of strain.

Other work by Shell researchers (Van der Loo 1974 and 1978) has led to the development of a formula (Equation 2.3) that related rut depth ( $\Delta H$ ), thickness of asphalt layer ( $H$ ), stiffness of the mix ( $S_m$ ), average compressive stress within the asphalt layer ( $\sigma_{\text{tire}}$ ), dynamic versus static deformation correction factor ( $C_m$ ), and the stress distribution factor ( $Z$ ). For asphalt,  $C_m$  is in the range of 1.2 and 1.6.

$$\Delta H = H \cdot \sigma_{\text{tire}}/S_m \cdot C_m \cdot Z \quad (\text{Eq 2.3})$$

Alterations were made to the Shell rutting prediction model based on research conducted for Texas Department of Transportation (TxDOT) Research Project 7-2474 (Mahboub and Little 1988). The original Shell model for rutting prediction assumes a linear relationship between the applied stress and the accumulated strain. However, this relationship need not be linear. As such, a modified Shell equation was proposed (Equation 2.4),

$$h = H \cdot (Z\sigma_{tire}/\sigma_{lab})^{1.61} \epsilon_{VP}(t) \quad (\text{Eq 2.4})$$

where  $h$  is the calculated rut depth,  $H$  is the asphalt layer thickness,  $Z$  is the vertical stress distribution factor,  $\sigma_{tire}$  is the average contact pressure,  $\sigma_{lab}$  is the stress level at which the creep test is run, and  $\epsilon_{VP}(t)$  is the viscoplastic trend in the mixture measured in the creep test.

Yandell (1971), Kinder (1986), Lai and Anderson (1973), and Tseng and Litton (1986) have proposed using some form of the power law to approximate the accumulation of permanent deformation as a function of time or number of load cycles.

### 2.1.2 Influence of Creep Stress Intensity on Strain Rate

A schematic representation of the influence of creep stress intensity on strain rate at some given time after stress application is shown in Figure 2.1. At low stresses, creep rates are small and of little practical importance. The curve shape is compatible with the hyperbolic sine function predicted by the rate process in Equation 2.1. In the midrange of stresses, a nearly linear relationship is found between logarithm of strain rate and stress. This is also predicted in Equation 2.1, when the argument of the hyperbolic sine is greater than 1. At stresses approaching the strength of the material, the strain rates become very large, indicating the onset of failure (Mitchell 1976).

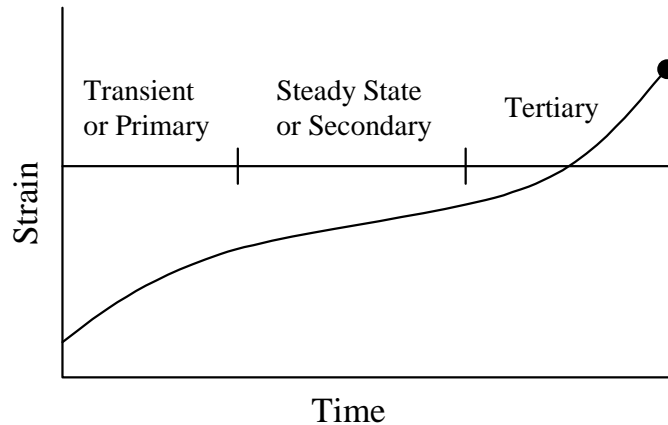


Figure 2.1: Influence of Creep Stress Intensity on Strain Rate

### 2.1.3 Various Stages of Creep

Figure 2.2 illustrates another generalized form used to show the various stages of creep. This figure provides the generalized shape for the strain (or deformation) as a function of time. The first stage, transient or primary creep, consists of a relatively high initial strain rate that decreases with time. The second stage, steady state or secondary creep, consists of a

constant strain rate (Finnie and Heller 1959). The third state, tertiary creep, consists of acceleration in strain rate leading to failure.



*Figure 2.2: Stages of Creep*

The relationship between creep strain and the logarithm of time may be linear, concave upward, or concave downward. However, a linear relationship is often assumed as a practical approximation because of its simplicity in analysis and ease of use. There is no fundamental *law* of behavior that dictates one or the other.

## **2.2 ASPHALT CREEP TESTS**

Uniaxial creep tests have been used to evaluate rutting susceptibility of asphalt concrete mixes for a long time. The use of these tests is a result of the test's relative simplicity and because of its logical connection with permanent deformation in asphalt mixes. However, as with all other laboratory tests, one major problem with the laboratory creep tests is the difficulty in relating laboratory results with actual field performance.

Van der Loo (1989) analyzed the relationship between rutting in the field and creep (static and dynamic) testing in the laboratory. After analyzing the use of results from laboratory-prepared specimens to predict rutting behavior, Van der Loo concluded "it may be that the main purpose of laboratory test methods must be limited to the ranking of materials rather than the prediction of rut depth."

Though creep tests on laboratory-prepared specimens may not be able to accurately predict rut depth, some work has been done to develop acceptance criteria for asphalt concrete mixes based on creep results.

### ***2.2.1 Evaluation of Creep Test Data***

The total strain at failure after a period of loading is often used as a criterion for accepting or rejecting mixes. Typically, the strain after 1 hour of loading is used. Such was the time period used in the asphalt aggregate mixture analysis system (AAMAS, a mixture analysis system developed prior to, and which was to serve as a starting point for, the Strategic Highway Research Program [SHRP]). A minimum creep modulus after 1 hour of loading was recommended in AAMAS (Von Quintus et al. 1991). Another commonly used criterion is the creep stiffness. This is calculated by dividing the applied stress by the resulting strain at a specific time period. This is often also calculated after 1 hour of loading.

Two other criteria are the permanent strain and the slope. The permanent strain is the residual strain remaining after a sample is allowed to recover the elastic portion of the strain. For permanent strain criteria, a fixed-loading period is followed by a fixed-recovery period. TxDOT's test method Tex-231-F is one such example (Texas Department of Transportation 1995). Slope is defined as the gradient of the strain versus time curve. A maximum slope is specified for the end of the loading period to determine whether the sample is in the secondary creep phase or in the tertiary creep phase of creep. The secondary creep phase has a relatively small slope compared to the slope in the tertiary creep phase. Tertiary creep is an indication that a sample is about to fail and may not be acceptable.

### ***2.2.2 Comparison of Different Creep Modulus Criteria***

The AAMAS creep test has a loading period of 1 hour and a recovery time of 1 hour. The 1-hour test period is relatively popular because it is long enough to be applicable to the loading conditions during which rutting occurs, yet short enough to be practical.

The minimum creep modulus values at 1 hour of loading from the creep modulus charts developed by Mahboub and Little (1988) are as shown in Table 2.1. These values are based on testing conducted at 40 °C (104 °F) under uniaxial compressive stresses that are representative of actual pavement conditions.

Researchers have also developed similar criteria from other creep tests. Khedr (1986) recommends a minimum creep modulus of 137.9 MPa (20 ksi) after 1 hour of testing under an applied pressure of 207 kPa (30 psi). Table 2.2 shows the range of acceptable creep modulus after 1 hour of loading at 40 °C (104 °F) under applied pressures of 103 kPa (14.9 psi) suggested by Kronfuss et al. (1984). Kronfuss et al. also established an upper limit for creep modulus at 46.5 MPa (6.7 ksi) because they believe that higher creep moduli could lead to mixes susceptible to load-induced fatigue or thermal effects. However, this criterion was established for cooler climates and should be interpreted with caution before applying it to other environmental conditions. Viljoen and Meadows (1981) developed a minimum creep modulus criterion of 82.7 MPa (12 ksi) after 100 minutes of loading at 40 °C (104 °F) under an applied pressure of 207 kPa (30 psi).

*Table 2.1: Minimum Creep Modulus Values from Mahboub and Little (1988)*

Pavement Category	Rut Potential	Min. Creep Modulus (MPa)
Asphalt Concrete Over Rigid Base	Low	69
	Moderate	34.5
Full-Depth Asphalt Concrete (Intermediate Layers)	Low	55.2
	Moderate	20.7
Full-Depth Asphalt Concrete (Lower Layers)	Low	27.6
	Moderate	17.2
Surface Asphalt Concrete Layers	Low	55.2
	Moderate	27.6

*Table 2.2: Acceptable Creep Modulus from Kronfuss et al. (1984)*

Traffic Intensity Level	Range of Acceptable Creep Modulus (MPa)
Low	$\geq 20.7$
Moderate	20.7 – 31.0
High	31.0 – 45.6

Sousa et al. (1991) documented one of the most comprehensive studies on the effects of mix variables on compressive creep properties of asphalt concrete mixtures. The test temperature was 40 °C (104 °F), the applied vertical pressure was 207 kPa (30 psi), and the loading time was 1 hour. The parameters considered in this evaluation included the creep modulus or permanent strain following a specific period of loading, time to reach a critical level of strain (time to rupture), and slope of the strain-versus-time curve in a designated region, such as the steady-state region. Mixture variables considered in this study include asphalt cement type, asphalt content, aggregate type, air void content, compaction temperature, and stress level. A review of this data indicates that compressive creep is sensitive to all of these factors.

However, the levels selected for each of the variables need to be evaluated carefully before a general conclusion for all possible levels for each factor is reached. Overall, a creep modulus greater than 69 Mpa (10 ksi) under realistic conditions indicates a mix that has low sensitivity to rutting. Creep moduli in the range of 41.4 (6 ksi) to 69 MPa (10 ksi) indicate that a mix is moderately to highly sensitive to rutting.

Little et al. (1993) also conducted a study on 100 mixtures to develop critical creep values of slope for the steady-state portion of the creep-versus-time curve and permanent strain after 1 hour. The researcher concluded that creep test parameters that have been shown to relate rationally to permanent deformation potential are strain at 1 hour of loading, concomitant creep stiffness at 1 hour, and the log-log slope of the steady-state portion of the creep strain versus time curve.

## CHAPTER 3. TESTING EQUIPMENT: CURRENT SETUP AND MODIFICATIONS

### 3.1 TXDOT STATIC CREEP TEST (TEST METHOD TEX-231-F)

The Texas static creep test was initially developed as a strength test for crumb rubber modified (CRM) asphalt hot mix. This work was completed prior to 1993 (Tahmoressi 1993). TxDOT uses the static creep test to determine resistance to permanent deformation of bituminous mixtures at temperatures and loads similar to those experienced by these materials in the field (Texas Department of Transportation 1994).

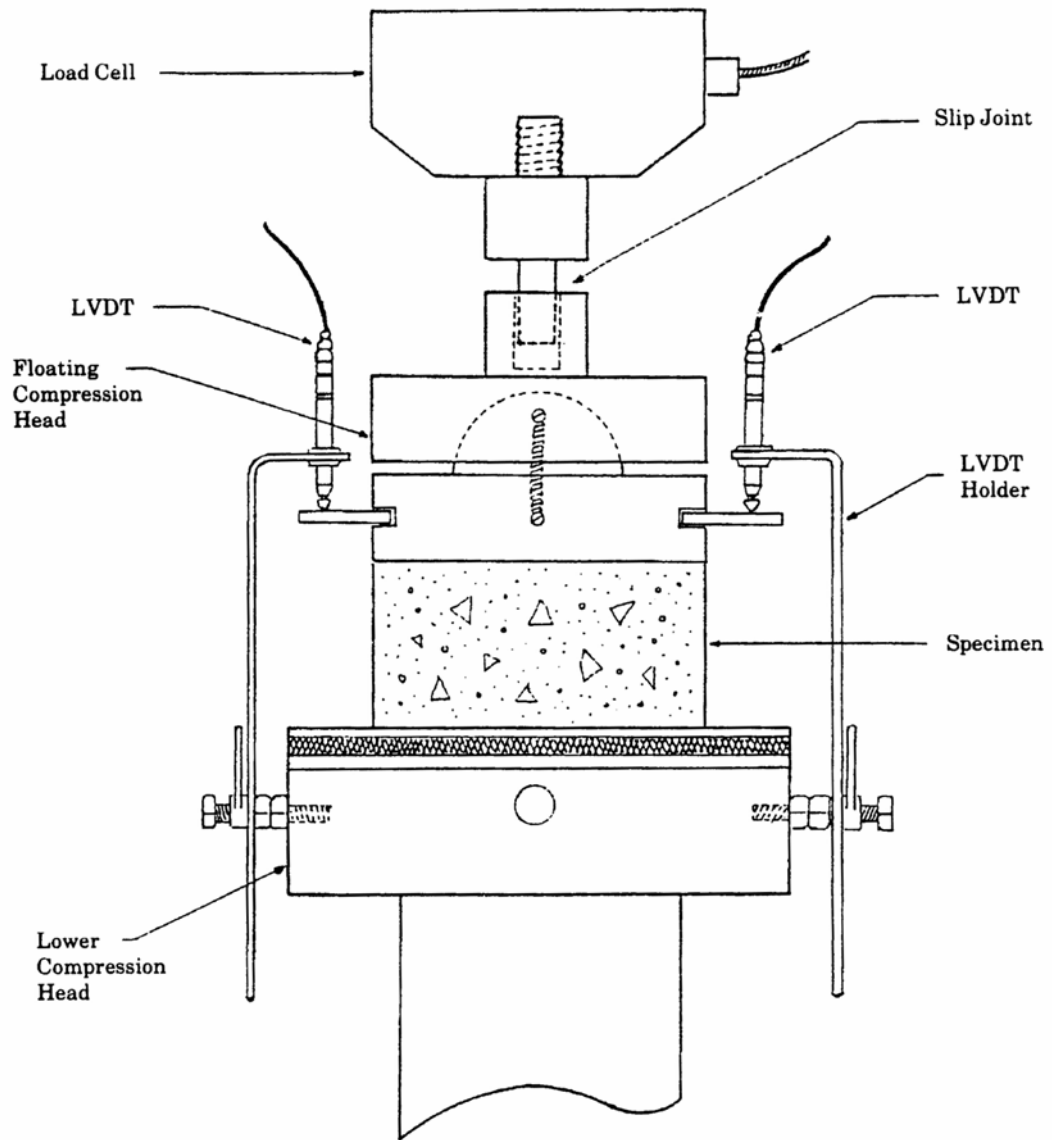
#### *3.1.1 Equipment Requirements*

The loading press must be capable of applying a constant axial load, up to 2224 N, over a specified time period. It must also be capable of measuring the axial load to an accuracy of plus or minus 1 percent of the applied axial load. The temperature control system must be capable of controlling temperature within the range of 40 °C (104 °F) to 60 °C (140 °F). The temperature must be held to within 1 °C (33.8 °F) of the specified test temperature.

Vertical deformations must be measured with a linear variable displacement transducer (LVDT). Two LVDTs, placed diametrically opposite each other, must be used to measure this deformation. The resolution on each LVDT must be better than 0.0025 mm (0.000098 in.). Smooth-loading platens must be used to minimize the effects of friction on the ends of the sample. The upper-load platen must be of the same diameter as the sample being tested to provide for positive centering of the specimen under load. The upper platen must be of the floating compression type to account for minor deviations in a specimen's surface (Texas Department of Transportation 1994). Figure 3.1 shows a picture of the test setup, and Figure 3.2 shows a schematic diagram of the test setup.



Figure 3.1: Static Creep Testing Apparatus

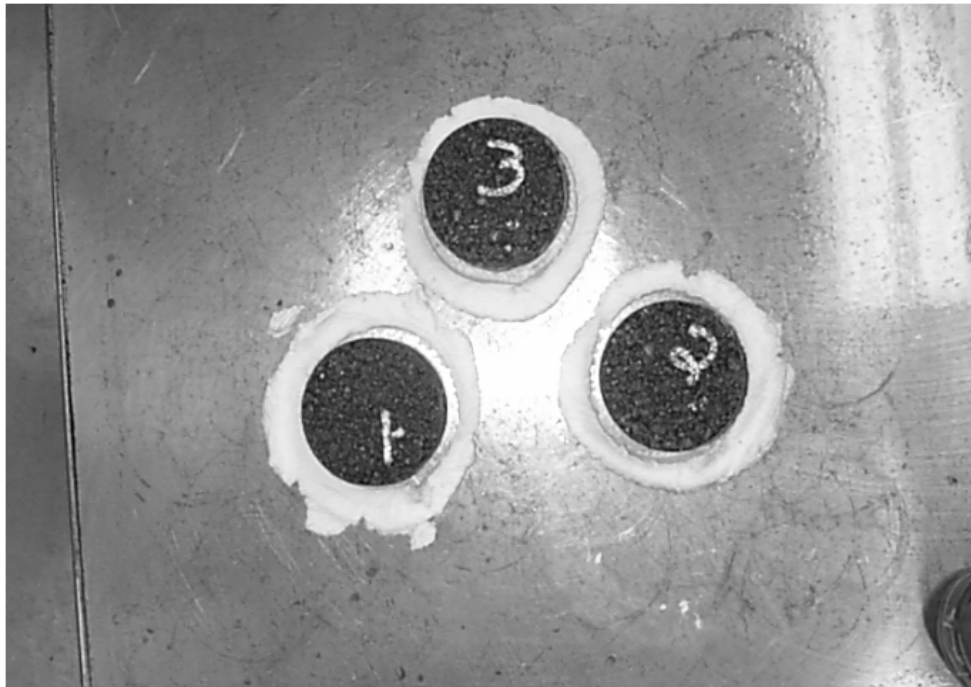


*Figure 3.2: Schematic Diagram of the Static Creep Setup  
(Texas Department of Transportation 1994)*

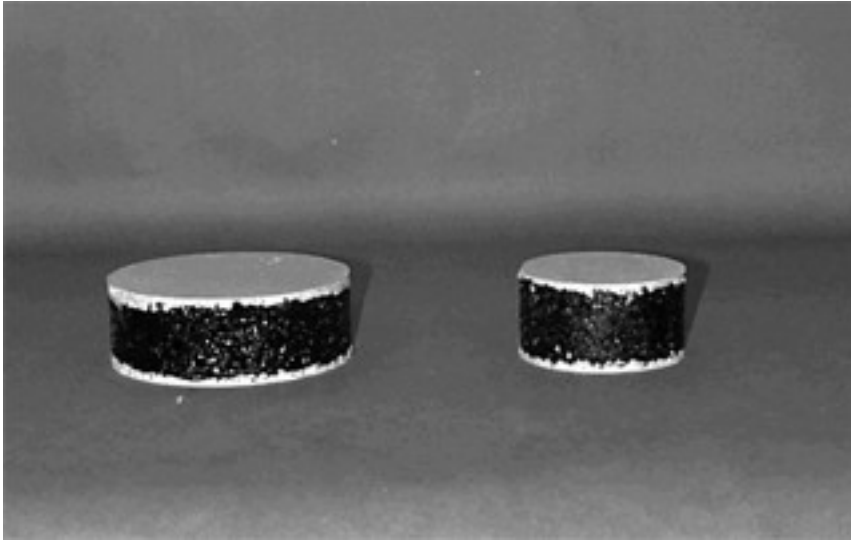
### ***3.1.2 Sample Preparation***

TxDOT static creep equipment is designed to test specimens compacted with the Texas gyratory compactor. This compaction method is described in Test Method Tex-205-F, “Laboratory Method of Mixing Bituminous Mixtures” (Texas Department of Transportation 1994).

Texas gyratory specimens are compacted to a diameter of 101 mm (3.98 in.). For the static creep test, they are compacted to a height of 51 (2 in.)  $\pm$  2 mm (0.08 in.). Prior to testing, the specimens are leveled with hydrocal, a capping compound that is a combination of gypsum cement and hydrostone, each contributing 50 percent by volume. The two components are mixed with water to an appropriate consistency, with the specimens then placed on top of the compound and leveled with a bubble level. Figure 3.3 shows three test specimens after one side has been capped. Once the hydrocal has dried, the specimen is turned over and the process is repeated to cap the other end of the specimen (Texas Department of Transportation 1994). Figure 3.4 shows a 100 mm (3.9 in.) specimen and a 150 mm (5.9 in.) specimen after both ends have been capped.



*Figure 3.3: Static Creep Specimens with One Side Capped*



*Figure 3.4: 150 mm and 100 mm Diameter Specimens after Capping*

### **3.1.3 Test Procedure**

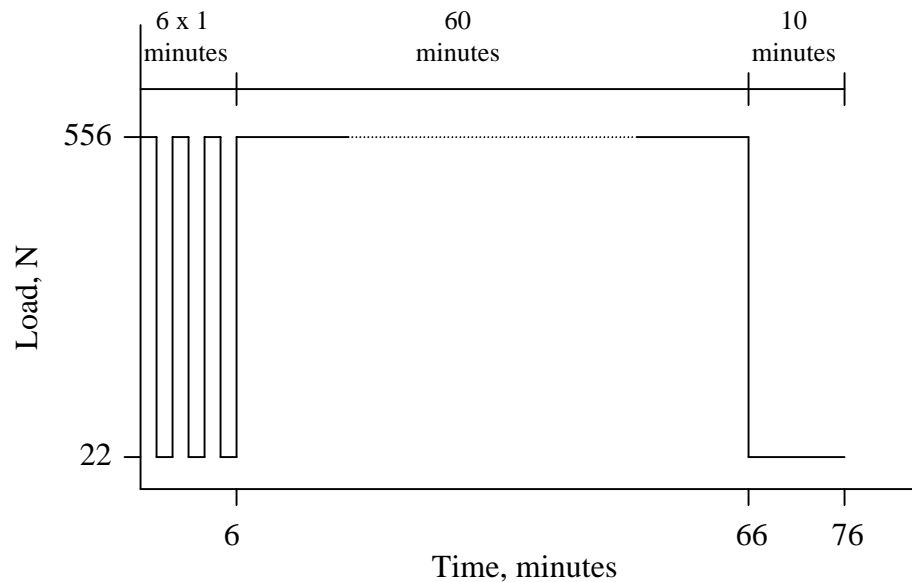
The test specimens are placed into a controlled temperature chamber maintained at 40 °C (104 °F) for 3 to 5 hours prior to the start of the test to bring the specimens to the test temperature. Following this preparation, the specimen is mounted as shown in Figure 3.5.



**Figure 3.5: Static Creep Test Setup**

To achieve more uniform contact between the loading platens and the specimen, three preloading cycles are applied prior to the test. Each cycle consists of a 556 N load applied for 1 minute, followed by a 1-minute rest period. After the preloading cycles, a 556 N load is applied to the specimen for 1 hour. At the end of 1 hour, the load is removed and the specimen allowed to rebound for 10 minutes (Texas Department of Transportation 1994).

Studies conducted by TxDOT in 1995 indicated that applying a 22 N load for the 1-minute rest period eliminated the possibility of a dynamic load hitting the specimen during loading because the loading head would no longer float during the 1-minute rest period. In addition, applying a 22 N load during the recovery phase of the test (when the 556 N load is removed) improved the repeatability of the permanent strain readings. As such, 1 N loads are applied during these *rest* or *recovery* periods of the test. Figure 3.6 shows the load applied during the test (Texas Department of Transportation 1995).



*Figure 3.6: Loading Pattern for Static Creep Test*

#### **3.1.4 Calculation of Material Properties**

Three material properties are measured from the static creep test: stiffness, permanent strain, and slope of the steady-state curve. The procedure to calculate required material properties is described in Texas Test Method Tex-231-F (Texas Department of

Transportation 1994). To begin, the average deformation is calculated by averaging the two LVDT readings. The strain is then calculated by dividing the average deformation by the specimen thickness (height). This is shown in Equation 3.1.

$$\text{Strain (mm/mm)} = \text{Average Deformation} / \text{Thickness} \quad (\text{Eq 3.1})$$

Next, strain is plotted against time for each of the three specimens. Figure 3.7 shows an example of the typical shape of the measured responses from each of the two LVDTs. Figure 3.8 shows the shape of a typical strain (averaged from the two LVDTs) versus time plot for one specimen. From this plot, the total strain (mm/mm), the permanent strain (mm/mm), and the slope of the steady-state curve (mm/mm/sec) are obtained. The creep stiffness is calculated using Equation 3.2.

$$\text{Creep Stiffness} = \text{Applied Pressure} / \text{Total Strain} \quad (\text{Eq 3.2})$$

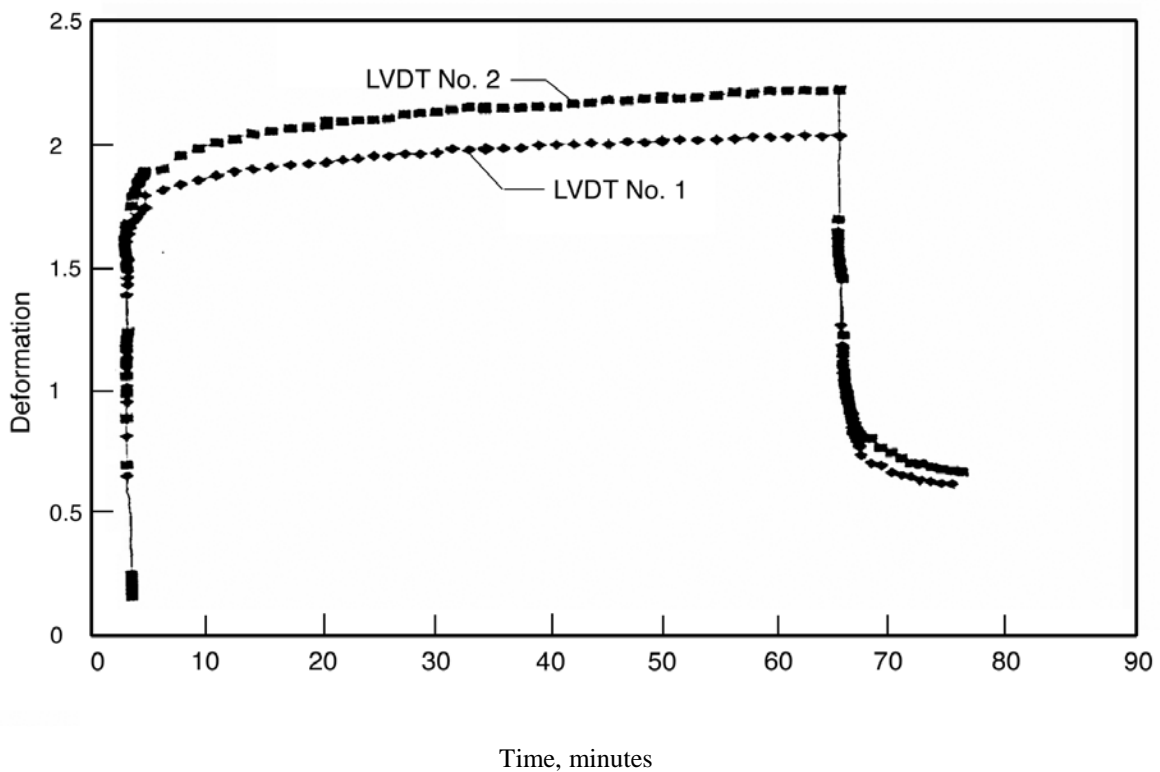
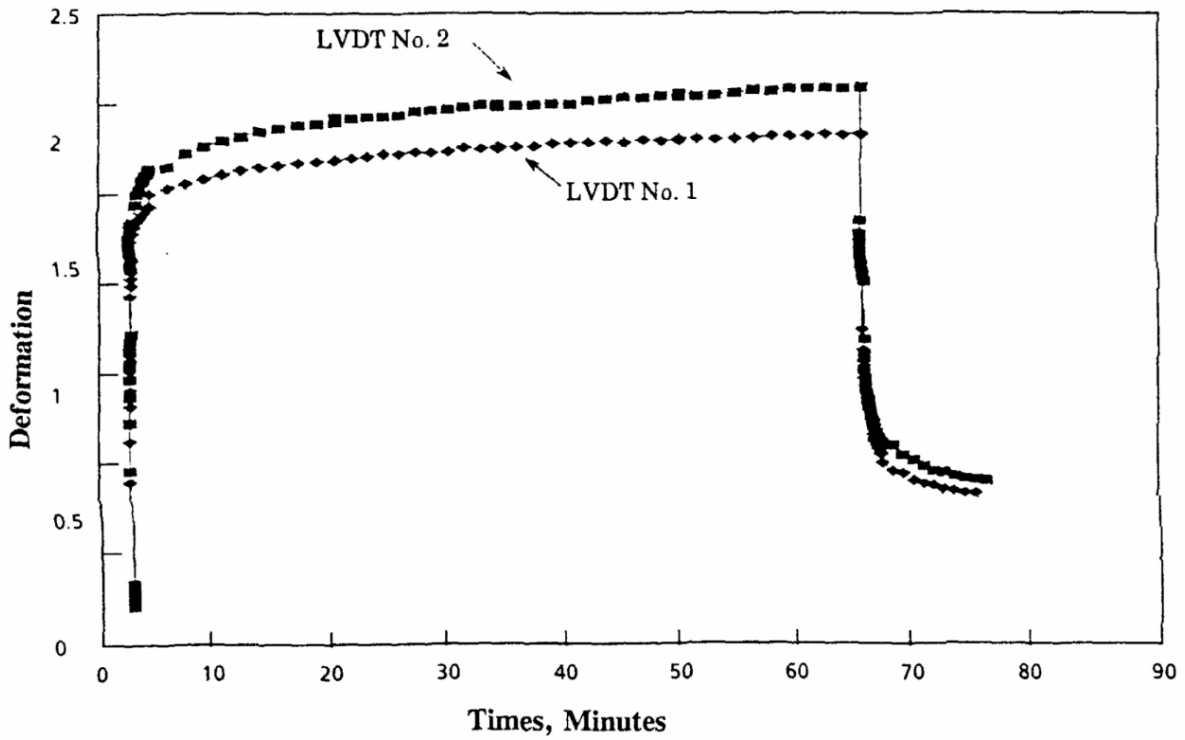


Figure 3.7: Sample Shape of the Response Measured by the Two LVDTs (Texas Department of Transportation 1994)



*Figure 3.8: Relationship between strain versus time for a typical static creep test (Texas Department of Transportation 1994)*



## CHAPTER 4. MIX DESIGN AND MATERIAL SELECTION

In this research effort, several sources for aggregates and asphalts were used. Among the aggregates used were limestone rock from Colorado Materials and Capitol Aggregates, sandstone from Delta Materials, and field sand from Odell-Greer Construction. Coincidentally, the three types of asphalts used on all of these projects were supplied by Texas Fuel and Asphalt (TF&A) in Corpus Christi, Texas.

### 4.1 SELECTION OF BASE MIX DESIGNS

BIH-35 (New Braunfels) mix design was chosen as the primary mix design. This mix was selected for a number of reasons. For one, the BIH-35 mix design is a coarse (aggregate gradation passes below the restricted zone) Superpave mix that is performing well in the field. Choosing BIH-35 as the primary mix design will shed more light on the behavior of Superpave mixes. In addition, the mix design was conducted at the South Central Superpave Center (SCSC). The aggregates (from Colorado Materials) were readily available at the SCSC, and the staff had a fair amount of experience with this aggregate source because Colorado Materials aggregates are used as the laboratory standard. The AC-20 asphalt that was used was also readily available from TF&A.

Figure 4.1 shows the aggregate gradation for BIH-35 (Mix A), as well as for two other finer mixes. The intermediate mix (Mix B) has a gradation that passes through the restricted zone. The fine mix (Mix C) passes over the restricted zone. Mixes B and C were developed to evaluate the effect of having additional fines in a mix. These three mixes will henceforth be referred to as the base mixes. All three base mixes are composed of aggregates from Colorado Materials and asphalt binder from TF&A. In Figure 4.1, the black solid squares represent boundary limits that the gradations must meet to satisfy Superpave 19 mm (0.75 in.) mix requirements.

It should be noted that the restricted zone resides along the maximum density gradation between the intermediate size [either 4.75 mm (0.19 in.) or 2.36 mm (.09 in.)] and the 0.3 mm (0.01 in.) size. It forms a area through which aggregate gradations are not permitted to pass under the Superpave system. Gradations that pass through this restricted zone, violating Superpave design requirements, are often called *humped gradations*, a term that denotes the noticeable hump in the gradation curve. In most cases, a humped gradation indicates a mixture that possesses too much fine sand in relation to total sand. This type of gradation can result in tender mix behavior that manifests itself by being difficult to compact during construction and more susceptible to permanent deformation. Gradations that violate this zone also possess weak aggregate skeletons that depend too much on asphalt binder stiffness to achieve shear strength and can be sensitive to asphalt content (McGennis et al. 1995).

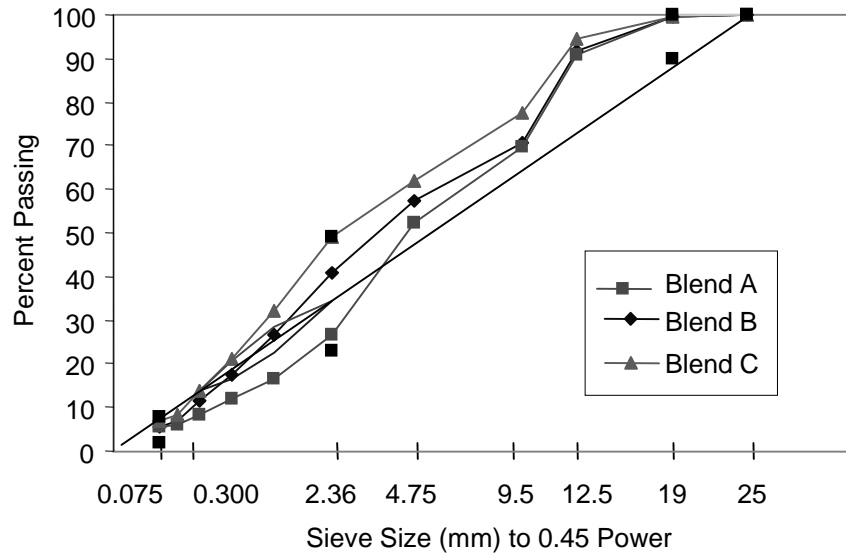


Figure 4.1: Gradation of Three Base Mixes

#### 4.1.1 Aggregates and Asphalt Used in the Base Mixes

The contractor of the BIH-35 project, Colorado Materials, provided the aggregates used in the three base mixes. The component aggregate stockpiles were C-Rock, D-Rock, F-Rock, and washed screenings. Table 4.1 shows the gradation of these four component aggregate stockpiles.

Table 4.1: Aggregate Gradation for Colorado Materials Stockpiles

Size, mm	Percent Passing			
	Colo. Matls. C-Rock	Colo. Matls. D-Rock	Colo. Matls. F-Rock	Colo. Matls. Washed Scrn.
25	100.0	100.0	100.0	100.0
19	98.7	100.0	100.0	100.0
12.5	70.0	99.3	100.0	100.0
9.5	7.9	70.1	100.0	100.0
4.75	2.7	6.9	73.6	98.4
2.36	2.0	3.9	13.0	81.5
1.18	1.9	3.0	4.6	53.0
0.6	1.7	2.8	4.0	34.0
0.3	1.6	2.5	3.5	21.5
0.15	1.5	2.3	3.3	12.0
0.075	1.4	1.9	3.0	10.0

In trying to replicate the mix that is in place on IH-35 in New Braunfels, the research team developed the design of Mix A based on aggregate gradations and asphalt content from a QC/QA testing summary provided by the New Braunfels District office. The aggregate gradation is shown in Figure 4.1; the asphalt content is 5.1 percent of the total mass of the mix. As per the original design for BIH-35, Mix A was designed based on a design gyration level ( $N_{\text{design}}$ ) of 96 gyrations. The two other mixes (Mix B and Mix C) were designed based on a design gyration level ( $N_{\text{design}}$ ) of 68 gyrations. Mixes B and C both have an asphalt content of 4.8 percent by total mass of the mix.

The asphalt binder used in the BIH-35 project is an AC-20 supplied by TF&A. Table 4.2 shows the asphalt binder properties for this AC-20. This asphalt was graded as a PG 64-22 using the Superpave performance graded classification system. The same asphalt was used in Mixes B and C, so that the effect of having greater quantities of fines would not be confounded with the type of asphalt.

*Table 4.2: TF&A AC-20 Asphalt Binder Properties*

Property	Test Result	Specification
Unaged Properties: $G^*/\sin \delta$ @ 64 °C	1.56 kPa	1.00 kPa min.
RTFO-Aged Properties: $G^*/\sin \delta$ @ 64 °C	3.28 kPa	2.20 kPa min.
PAV-Aged Properties: $G^*\sin \delta$ @ 25 °C S @ -12 °C m @ -12 °C	3677 kPa 228 MPa 0.357	5000 kPa max. 300 MPa max. 0.300 min.

## 4.2 ADDITIONAL MIX DESIGNS

In addition to the three base mix designs, the mix designs for three other mixes that are performing well in the field were selected for evaluation. Once again, the TxDOT district offices assisted in the selection of these mixes. One of the mixes is a Superpave 12.5 mm (0.49 in.) mix laid down on IH-35 in Waco, Texas. The other two are coarse matrix high binder (CMHB) mixes. The first one is a CMHB-C (coarse CMHB) laid down on IH-35 in Austin, Texas. The second one is a CMHB-F (fine CMHB) laid down on FM 1103 in New Braunfels, Texas.

### *4.2.1 Superpave 12.5 mm Mix in Waco, Texas*

The Superpave 12.5 mm (0.49 in.) mix was laid down on IH-35 in Waco, Texas. The field gradation and the laboratory gradation used in this research effort for this mix are shown in Figure 4.2; the asphalt content is 4.9 percent of the total mass of the mix. In Figure

4.2, the black solid squares represent boundary limits that the gradations must meet to satisfy Superpave 12.5 mm (0.49 in.) mix requirements.

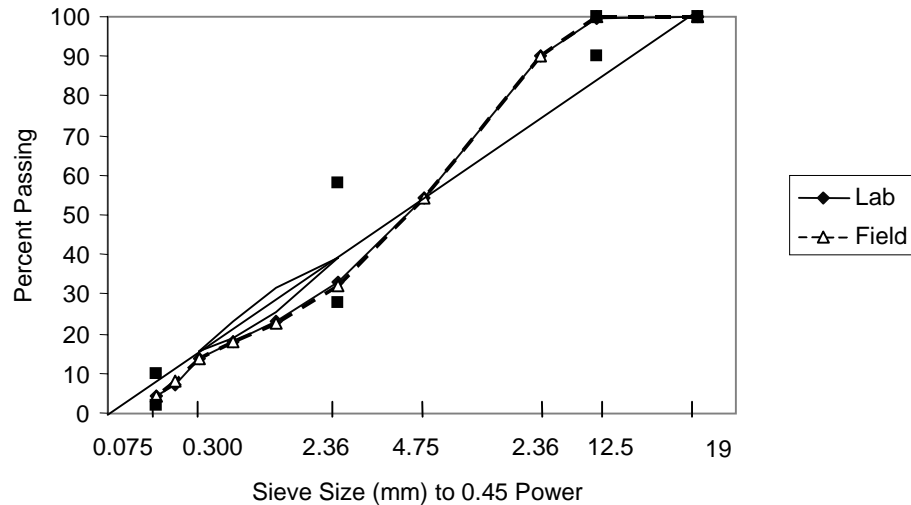


Figure 4.2: Aggregate Gradation for Superpave 12.5 mm (0.49 in.) Mix in Waco, Texas

Delta Materials, Capitol Aggregates, and Odell-Greer supplied the aggregates for this mix. The gradations for the aggregate stockpiles are shown in Table 4.3. TF&A supplied the PG 76-22 asphalt binder. Table 4.4 shows the properties of this asphalt binder.

Table 4.3: Aggregate Gradation of Superpave 12.5 mm (0.49 in.) Mix in Waco, Texas

Size, mm	Percent Passing			
	Delta Matls. D-Rock	Cap. Aggr. F-Rock	Cap. Aggr. Screenings	Odell-Greer Field Sand
19	100.0	100.0	100.0	100.0
12.5	99.0	100.0	100.0	100.0
9.5	77.0	100.0	100.0	100.0
4.75	14.0	69.0	100.0	100.0
2.36	5.0	16.0	87.0	100.0
1.18	3.0	6.0	59.0	100.0
0.6	3.0	4.0	39.0	100.0
0.3	3.0	3.0	23.0	89.0
0.15	3.0	3.0	11.0	34.0
0.0075	3.0	3.0	6.0	11.0

Table 4.4: TF&A PG 76-22 Asphalt Binder Properties

Property	Test Result	Specification
Unaged Properties: G*/sin $\delta$ @ 76 °C	1.32 kPa	1.00 kPa min.
RTFO-Aged Properties: G*/sin $\delta$ @ 76 °C	2.80 kPa	2.20 kPa min.
PAV-Aged Properties: G* sin $\delta$ @ 31 °C S @ -12 °C m @ -12 °C	1676 kPa 202 MPa 0.301	5000 kPa max. 300 MPa max. 0.300 min.

#### 4.2.2 CMHB-C Mix in Austin, Texas

The CMHB-C mix was laid down on IH-35 in Austin, Texas. The field gradation and the laboratory gradation replicated in this research effort for this mix are shown in Figure 4.3; the asphalt content is 5.3 percent of the total mass of the mix. In Figure 4.3, the black circles represent boundary limits that the gradations must meet to satisfy CMHB-C requirements.

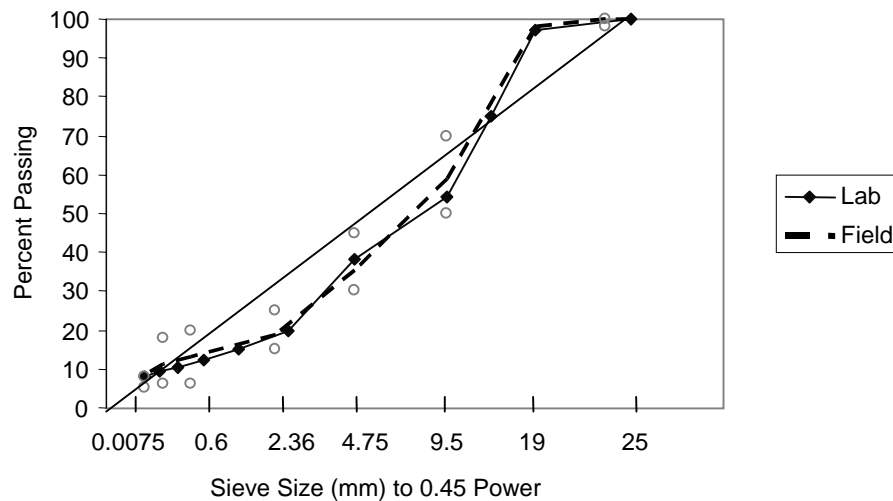


Figure 4.3: Aggregate Gradation for CMHB-C Mix on IH 35 in Austin

Delta Materials and Capitol Aggregates supplied the aggregates for this mix. The gradations for the aggregate stockpiles are shown in Table 4.5. TF&A supplied the AC-45P asphalt binder. Table 4.6 shows the properties of this asphalt binder. This asphalt was graded as a PG 70-22 using the Superpave performance graded classification system. It should be noted that this asphalt binder barely missed being graded out as a PG 70-28 because its m-value is 0.001 lower than the required 0.300.

*Table 4.5: Aggregate Gradation of CMHB-C Mix on IH 35 in Austin*

Size, mm	Percent Passing			
	Delta Matls. C-Rock	Cap. Aggr. D-Rock	Cap. Aggr. F-Rock	Cap Aggr. Scalpings
25	100.0	100.0	100.0	100.0
22.4	100.0	100.0	100.0	100.0
16.0	94.0	100.0	100.0	100.0
9.5	5.0	76.0	100.0	100.0
4.75	2.0	7.0	58.0	100.0
2.00	1.0	2.0	7.0	85.0
0.425	0.0	0.0	3.0	55.0
0.180	0.0	0.0	2.0	46.0
0.0075	0.0	0.0	0.7	39.0

*Table 4.6: TF&A AC-45P Asphalt Binder Properties*

Property	Test Result	Specification
Unaged Properties: G*/sin $\delta$ @ 70 °C	2.30 kPa	1.00 kPa min.
RTFO-Aged Properties: G*/sin $\delta$ @ 70 °C	3.39 kPa	2.20 kPa min.
PAV-Aged Properties: G* <sub>sin<math>\delta</math></sub> @ 28 °C S @ -12 °C m @ -12 °C	1855 kPa 141 MPa 0.361	5000 kPa max. 300 MPa max. 0.300 min.

#### ***4.2.3 CMHB-F Mix in New Braunfels, Texas***

The CMHB-F mix was laid down on FM 1103 in New Braunfels, Texas. The field gradation and the laboratory gradation replicated in this research effort for this mix are shown in Figure 4.4; the asphalt content is 6.5 percent of the total mass of the mix. The black circles represent boundary limits that the gradations must meet to satisfy CMHB-F requirements.

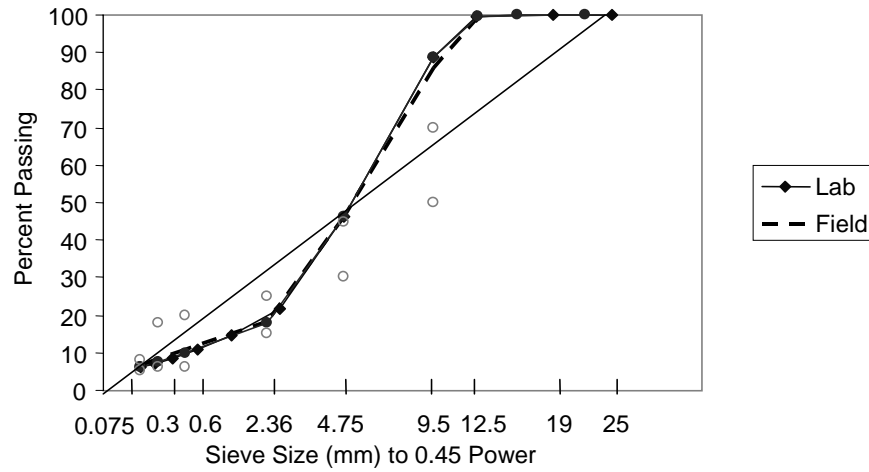


Figure 4.4: Aggregate Gradation for CMHB-F Mix on FM 1103 in New Braunfels, Texas

Colorado Materials supplied the aggregates for this mix. The gradations for the aggregate stockpiles are shown in Table 4.7. TF&A supplied the AC-20 asphalt binder. This is the same asphalt binder as the one used in the BIH-35 mix in New Braunfels. Table 4.2 shows the properties of this asphalt binder.

Table 4.7: Aggregate Gradation of CMHB-F Mix on IH 35 in Austin

Size, mm	Percent Passing		
	Colorado Materials D-Rock	Colorado Materials F-Rock	Colorado Materials Screenings
22.4	100.0	100.0	100.0
16.0	100.0	100.0	100.0
12.5	99.1	100.0	100.0
9.5	73.9	100.0	100.0
4.75	7.3	68.8	100.0
2.00	7.3	7.5	88.8
0.425	2.8	3.8	46.9
0.180	2.6	3.1	34.4
0.075	2.3	2.9	26.1

### 4.3 LIMITATIONS AND ASSUMPTIONS

As with any effort to replicate mixes that are already laid in the field, there are limitations to the extent to which the mixes can be reproduced. It should be noted that the goal of this research effort was to replicate what was in the field in terms of aggregate type, aggregate gradation, asphalt type, and asphalt content. However, designs are based on specimens compacted in the laboratory. Therefore, the difference between compaction

method in the laboratory and compaction method in the field is taken into account. Compaction devices used in the design stage (i.e., the Texas gyratory compactor or the SGC) adequately prepared the mixes for this research effort. It should be noted that all mixes were compacted to an air void content of 4, plus or minus 0.5 percent, in order to ensure consistency within the different mixes as well as between different mixes.

It is impossible to use exactly the same aggregates and asphalt used in the field projects unless some of the materials are stored during construction for future use (as was done on the Westrack project). Therefore, the original aggregate and asphalt producers were contracted to acquire materials that were as close to the materials used in the actual projects. This use of the original producers is the best that could be done.

However, the gradations and asphalt contents used in this research effort are based on the data collected and recorded in the QC/QA summary sheets. For this reason, the replicated mixes actually represent the field mixes and not those originally designed in the laboratory. This approach was chosen because it is common for the contractor to adjust the mix design during production, and the good-performing mixes were selected based on their performance in the field and not because of the performance of the laboratory-compacted specimens used in the design phase.

Volumetric data for the samples tested from each of the mix designs are shown in Appendix A.

## CHAPTER 5. EXPERIMENT DESIGN

The overall research effort is divided into six phases. The first phase involves modifications to TxDOT's static creep tester and the SST. The next four phases involve the evaluation of different test parameters. The experiment designs for these four phases will be presented separately in the subsequent sections. The last phase, Phase VI, will discuss recommendations for developing acceptance criteria, if possible. It should be noted that the numbers shown in the subsequent tables represent the number of replicates used for that combination of factors.

### 5.1 PHASE II: EVALUATING THE EFFECT OF SAMPLE DIAMETERS AND TESTING MACHINES

In Phase II, the effect of sample diameters and testing machines was evaluated. The three base mixes (A, B, and C) discussed in Chapter 4 were used in this phase. These mixes were molded to a diameter of 100 mm (3.9 in.) using the Texas gyratory compactor (TGC) and molded to a diameter of 150 mm (5.9 in.) using the Superpave gyratory compactor (SGC). The specimen heights were kept constant at approximately 51 mm (2 in.). In Phase II, all the tests were conducted at 40 °C (104 °F) on *molded* specimens. Table 5.1 shows this experiment design. Three replicates were used for each combination of levels.

Table 5.1: Experiment Design for Phase II

Static Creep Machine	Diameter (mm)	Mix		
		A	B	C
TxDOT (TGC)	100	3	3	3
	150	3	3	3
SST (SGC)	100	3	3	3
	150	3	3	3

### 5.2 PHASE III: EVALUATING THE EFFECT OF SAMPLE PREPARATION

In Phase III, the effect of sample preparation is evaluated. The same three base mixes used in Phase II (A, B, and C) were used in this phase. The *cut* specimens were prepared by saw-cutting a 51 mm (2 in.) section out of the middle of a regular SGC-compacted specimen. The *molded* specimens were molded in the SGC directly to the required height of 51 mm (2 in.). Phase III, all tests were conducted at 40 °C (104 °F). Table 5.2 shows this experiment design. Three replicates were used for each combination of levels.

Table 5.2: Experiment Design for Phase III

Static Creep Machine	Preparation	Mix		
		A	B	C
TxDOT	Molded	3	3	3
	Cut	3	3	3
SST	Molded	3	3	3
	Cut	3	3	3

### 5.3 PHASE IV: EVALUATING THE EFFECT OF ASPHALT CONTENT AND TEMPERATURE

In Phase IV, the effect of sample preparation was evaluated. The same three base mixes used in Phases I and II (A, B, and C) were used in this phase. In Phase IV, all the tests were conducted on the TxDOT static creep tester at 40 °C (104 °F) and 64 °C (147.2 °F) on *molded* specimens. Table 5.3 shows this experiment design. Three replicates were used for each combination of levels.

Table 5.3: Experiment Design for Phase IV

Asphalt Content	Temperature (°C)	Mix		
		A	B	C
Optimum	40	3	3	3
	64	3	3	3
Optimum + 0.5%	40	3	3	3
	64	3	3	3
Optimum + 1.0%	40	3	3	3
	64	3	3	3

In addition to the test matrix shown in Table 5.3, the BIH-35 mix (A) was also molded at an asphalt content of 0.5 percent less than optimum.

### 5.4 PHASE V: EVALUATING GOOD AND POOR MIXES

In Phase V, good-performing and poor-performing mixes in the Central Texas region were selected for this evaluation. Details of the reasons why these mixes were selected are discussed in Chapter 4. In Phase V, all tests were conducted at 40 °C and 64 °C on *molded*

specimens. Table 5.4 shows this experiment design. Three replicates were used for each combination of levels.

*Table 5.4: Experiment Design for Phase V*

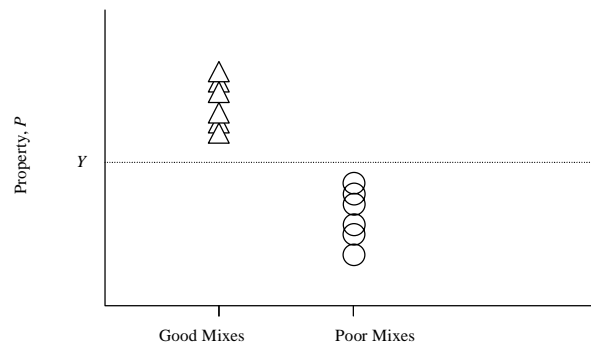
Type of Mix	Temperature (°C)	Good Mix	Poor Mix
Superpave 12.5 mm	40	3	3
	64	3	3
Superpave 19.0 mm	40	3	3
	64	3	3
CMHB-C	40	3	3
	64	3	3
CMHB-F	40	3	3
	64	3	3

## 5.5 PHASE VI: DEVELOPMENT OF ACCEPTANCE CRITERIA

The first phase of this experiment focused on evaluating the effect of specimen diameter (compounded with compaction method) and testing device. The second phase of this experiment focused on evaluating two specimen-preparation methods for SGC-compacted specimens and testing device. Although part of the overall picture, these two phases do not contribute directly to the development of acceptance criteria.

The third phase of this experiment focused on evaluating the effect of asphalt content and testing temperature. The fourth phase focused on differentiating the mix properties of good and poor mixes. Results from these two phases, coupled with engineering judgment and with experience obtained from previous research efforts, will be used to develop some kind of acceptance criteria.

Figure 5.1 shows an example of how acceptance criteria may be developed. In this example, mix property value ( $P$ ) is plotted for good-performing and poor-performing mixes. Accepting the fact that good-performing mixes have higher  $P$  values than poor-performing mixes, a particular  $P$  value can be selected to delineate between good and poor mixes. In this example, a value  $A$  is selected to delineate between good-performing and poor-performing mixes.



*Figure 5.1: Example of a Method to Develop Acceptance Criteria*

If a clear distinction cannot be made between good-performing and poor-performing mixes, then engineering judgment will be used to determine what appropriate guidelines can be recommended for developing acceptance criteria.

## CHAPTER 6. DISCUSSION OF RESULTS

The modifications to TxDOT's static creep test and the SST (Phase I) are discussed in Chapter 3. The results for Phases II through V are discussed in this chapter. For these four phases, three properties were evaluated — stiffness, permanent strain, and slope. Tests results from all the tests can be found in Appendix A.

Based on the variability within replicates in this study, practical significance was determined to be differences greater than 20,000 kPa (2,900 psi) for stiffness, differences greater than  $0.1 \times 10^{-3}$  mm/mm for permanent strain, and differences greater than  $1.0 \times 10^{-8}$  mm/mm/sec for slope.

### 6.1 PHASE II RESULTS: SAMPLE DIAMETER AND TESTING MACHINE EVALUATION

In Phase II of this research effort, the effect of specimen diameter and testing machine was evaluated. The two specimen diameters used were 100 mm (3.9 in.) and 150 mm (5.9 in.). The two testing machines used were TxDOT's static creep tester and the SST. Three replicates for each combination of levels were tested. It should be noted that the specimens were all molded specimens and that they were all tested at 40 °C (104 °F).

Table 6.1 shows the levels at which the main effects or interactions are significant for stiffness, permanent deformation, and slope.

*Table 6.1: Statistical Analysis for Phase II*

Factor/Interaction	p-value		
	Stiffness	Perm. Strain	Slope
Mix	0.0032	0.0087	0.0327
Diameter	0.0061	< 0.0001	< 0.0001
Machine	0.0185	0.5819	0.2241
Mix*Diameter	0.0004	0.0067	0.0261
Mix*Machine	0.0322	0.5867	0.4908
Machine*Diameter	0.0021	0.8879	0.6462
Three-Factor Interaction	0.0073	0.3496	0.4354

For stiffness, all main effects and interactions are statistically significant at an  $\alpha$  level of 5 percent. Even at an  $\alpha$  level of 1 percent, only machine (main effect) and the interaction between machine and mix are not statistically significant. Figure 6.1 shows the column chart and line graphs for stiffness. Figure 6.2 shows the interaction between machine and mix. The significant interaction between the three factors does not allow for any observable trends in stiffness. From a practical standpoint, the observed stiffness for all the combinations, except Mix B and Mix C at 150 mm (5.9 in.) run on the SST, is essentially the same because the range of stiffness within three replicates is about 10,000 (1,450) to 20,000 kPa (2,900 psi).

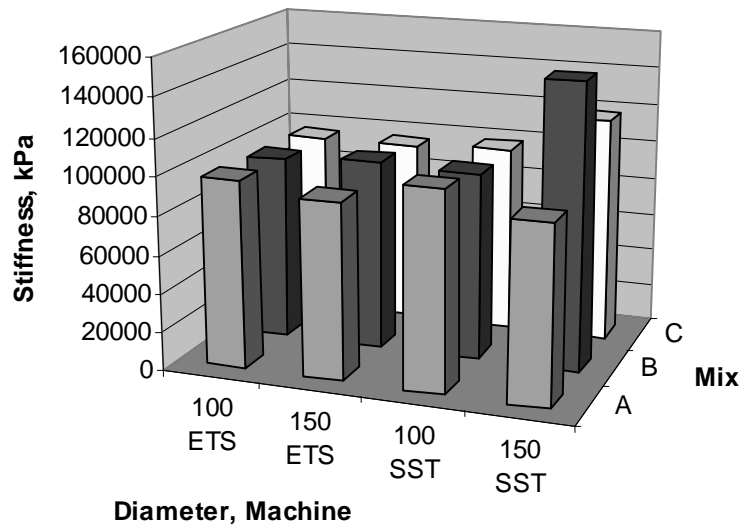


Figure 6.1: Phase II Column Chart for Stiffness (ETS denotes TxDOT's EnduraTEC Systems [ETS] device)

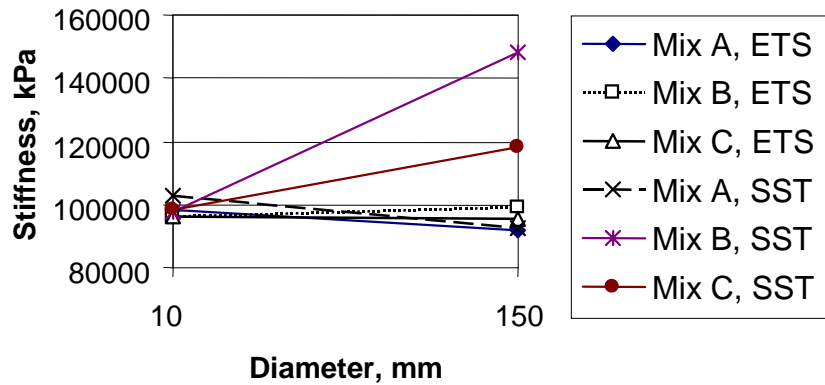


Figure 6.2: Phase II Line Graph for Stiffness (ETS denotes TxDOT's EnduraTEC Systems [ETS] device)

Figures 6.3 and 6.4 show the column chart for permanent strain and slope, respectively. In both cases, the diameter, mix, and interaction between diameter and mix are significant at an  $\alpha$  level of 5 percent. The 100 mm (3.9 in.) specimens have higher permanent strain and higher slope than the corresponding 150 mm (5.9 in.) specimens. This disparity can be due to boundary effects having a greater effect on the 150 mm (5.9 in.) specimens (aspect ratio of 0.33) than on the 100 mm (3.9 in.) specimens (aspect ratio of 0.50). Figures 6.5 and 6.6 show the interaction between diameter and mix for permanent strain and slope, respectively. An interesting trend was observed in permanent strain and slope: For 100 mm (3.9 in.) specimens, Mix B had the highest permanent strain, while for 150 mm specimens, Mix B had the lowest permanent strain. For 100 mm (3.9 in.) specimens, Mix A had the lowest slope, while for 150 mm (5.9 in.) specimens, Mix B had the lowest slope. These trends are due to the interaction between mix and diameter. From a practical standpoint, in half of the combinations, there are practical differences between specimens of different diameters for both the permanent strain and the diameter.

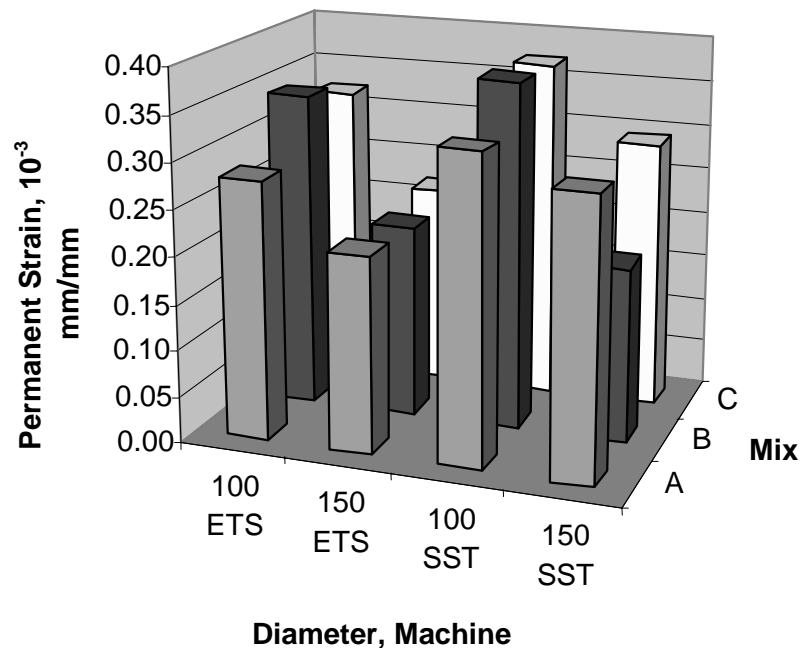


Figure 6.3: Phase II Column Plot for Permanent Strain (ETS denotes TxDOT's EnduraTEC Systems [ETS] device)

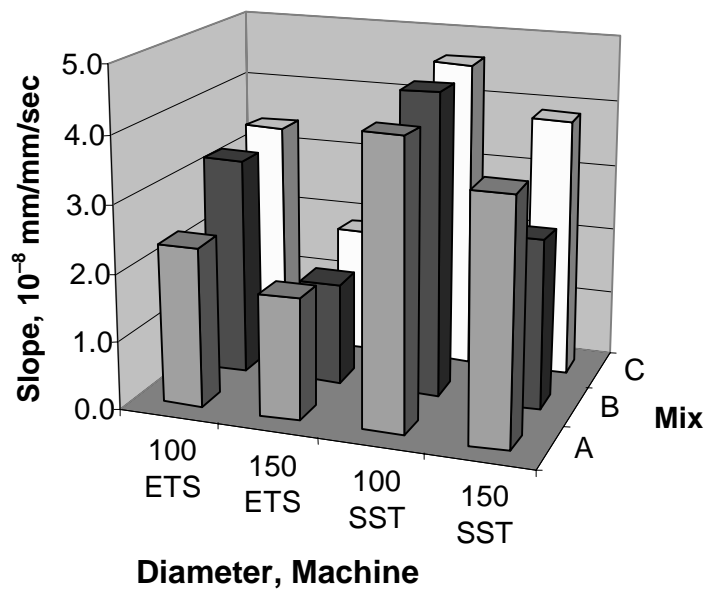


Figure 6.4: Phase II Column Plot for Slope (ETS denotes TxDOT's EnduraTEC Systems [ETS] device)

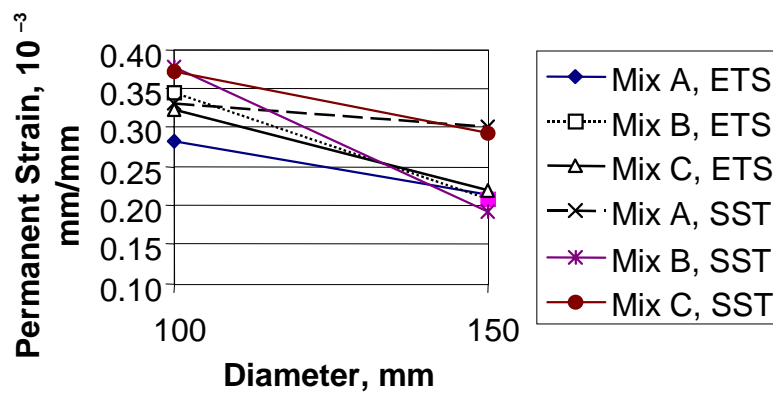


Figure 6.5: Phase II Interaction between Mix and Diameter for Permanent Strain (ETS denotes TxDOT's EnduraTEC Systems [ETS] device)

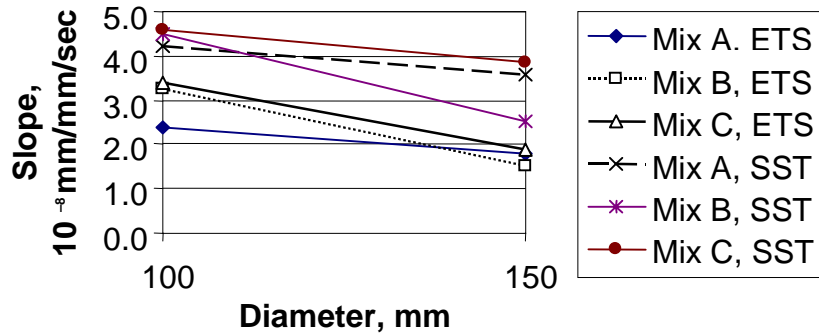


Figure 6.6: Phase II Interaction between Mix and Diameter for Slope (ETS denotes TxDOT's EnduraTEC Systems [ETS] device)

## 6.2 PHASE III RESULTS: SAMPLE PREPARATION AND TESTING MACHINE EVALUATION

In Phase III of this research effort, the effect of specimen preparation and testing machine was evaluated. The two preparation methods are *molded* and *cut*, as discussed earlier in Chapter 1. The two testing machines used were TxDOT's static creep tester and the SST. Three replicates for each combination of levels were tested. It should be noted that all the specimens were 100 mm (3.9 in.) in diameter, and they were all tested at 40 °C (104 °F).

Table 6.2 shows the level at which the main effects or interactions are significant for stiffness, permanent deformation, and slope.

Table 6.2: Statistical Analysis for Phase III

Factor/Interaction	p-value		
	Stiffness	Perm. Strain	Slope
Mix	0.0007	0.0250	0.0210
Sample Preparation	0.1227	0.6350	0.4305
Machine	< 0.0001	0.0200	< 0.0001
Mix*Sample Preparation	0.0196	0.7666	0.5736
Mix*Machine	0.0006	0.1073	0.2669
Machine*Sample Prep.	0.9410	0.5514	0.2344
Three-Factor Interaction	0.2165	0.2668	0.7402

For stiffness, permanent strain, and slope, testing machine and mix (main effects) are statistically significant at an  $\alpha$  level of 5 percent. For stiffness, the interaction between testing machine and mix is also statistically significant at an  $\alpha$  level of 5 percent. Figures 6.7, 6.8, and 6.9 show the column charts for stiffness, permanent strain, and slope, respectively.

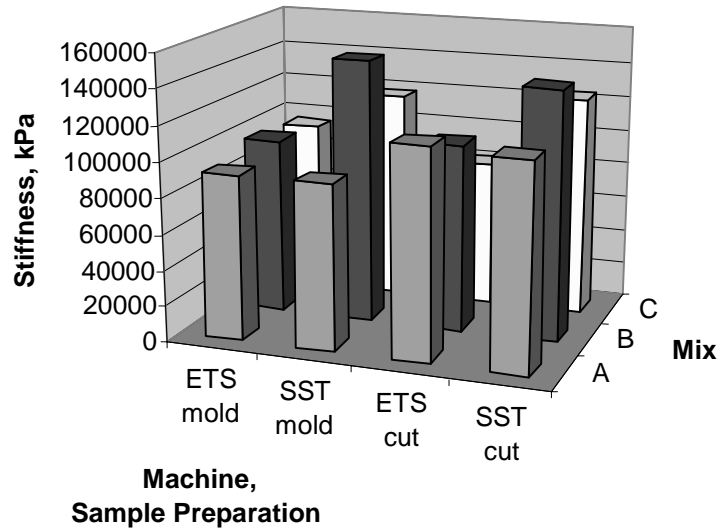


Figure 6.7: Phase III Column Chart for Stiffness (ETS denotes TxDOT's EnduraTEC Systems [ETS] device)

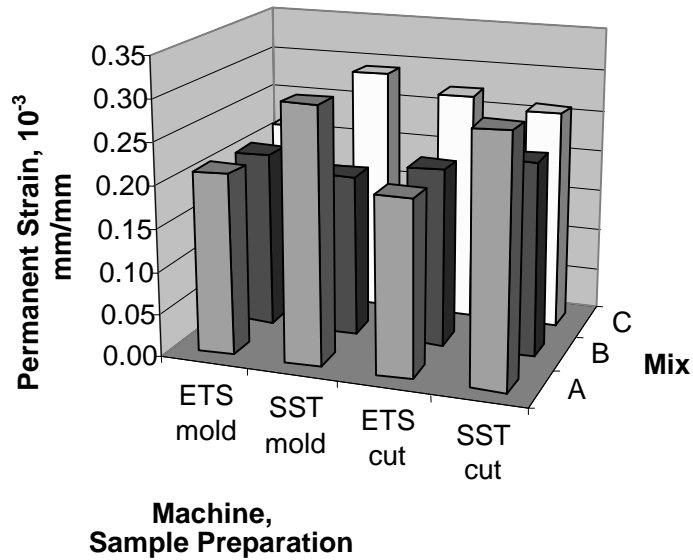


Figure 6.8: Phase III Column Chart for Permanent Strain (ETS denotes TxDOT's EnduraTEC Systems [ETS] device)

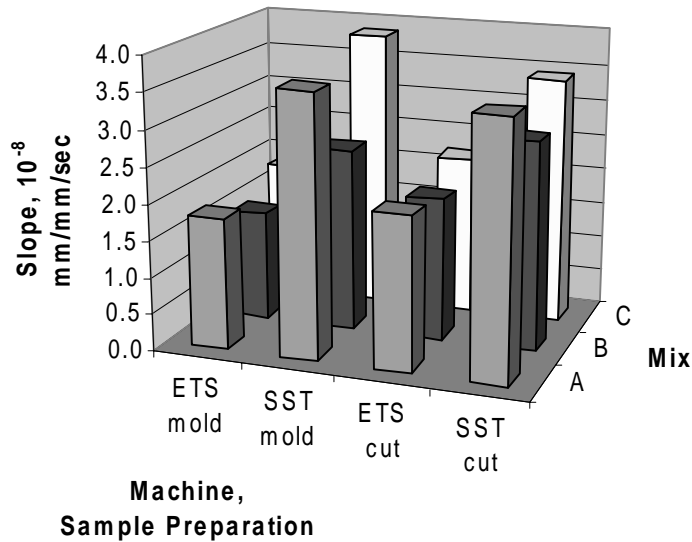


Figure 6.9: Phase III Column Chart for Slope (ETS denotes TxDOT's EnduraTEC Systems [ETS] device)

With the SST, Mix B has the greatest stiffness, followed by Mix C, and then Mix A. With the EnduraTEC Systems (ETS) device, there does not appear to be a clear trend. From a practical standpoint, there is little difference in stiffness among the mix, machine, or sample preparation method. The only practical difference is between the ETS and the SST for Mix B. Mix A and Mix C exhibit this difference, with such difference explained by the interaction between mix and sample preparation.

For permanent strain, there are no clear trends, and there does not appear to be a practical significance for any of the main effects.

Slope results with the SST are consistently higher than those with the ETS, and these differences are of practical significance. There is little practical significance among mixes, though Mix B consistently exhibits lower slope values for all combinations of machine and sample preparation.

### 6.3 PHASE IV RESULTS: ASPHALT CONTENT AND TEMPERATURE EVALUATION

In Phase IV of this research effort, the effect of asphalt content and temperature was evaluated. The three asphalt contents used were optimum, optimum plus 0.5 percent, and optimum plus 1 percent. In addition, Mix A (the primary mix) was also tested at optimum minus 0.5 percent. The two temperatures used were 40 °C (104 °F) and 64 °C (147 °F).

Three replicates for each combination of levels were tested. It should be noted that all the specimens were molded and that they were all 150 mm (5.9 in.) in diameter.

Table 6.3 shows the level at which the main effects or interactions are significant for stiffness, permanent deformation, and slope.

*Table 6.3: Statistical Analysis for Phase IV*

Factor/Interaction	p-value		
	Stiffness	Perm. Strain	Slope
Mix	0.9626	0.7665	0.4580
Asphalt Content	0.0222	0.0482	0.0001
Temperature	0.0803	0.7628	< 0.0001
Mix*Asphalt Content	0.5711	0.8800	0.4153
Mix*Temperature	0.8539	0.6970	0.3487
Temperature*Asphalt Content	0.1054	0.0591	0.0001
Three-Factor Interaction	0.6948	0.8982	0.2660

In this Phase IV, asphalt content (main effect) is statistically significant at an  $\alpha$  level of 5 percent for stiffness, permanent strain and slope. Asphalt content and temperature are also statistically significant at an  $\alpha$  level of 0.01 percent for slope. Figures 6.10, 6.11, and 6.12 show the column charts for stiffness, permanent strain, and slope, respectively.

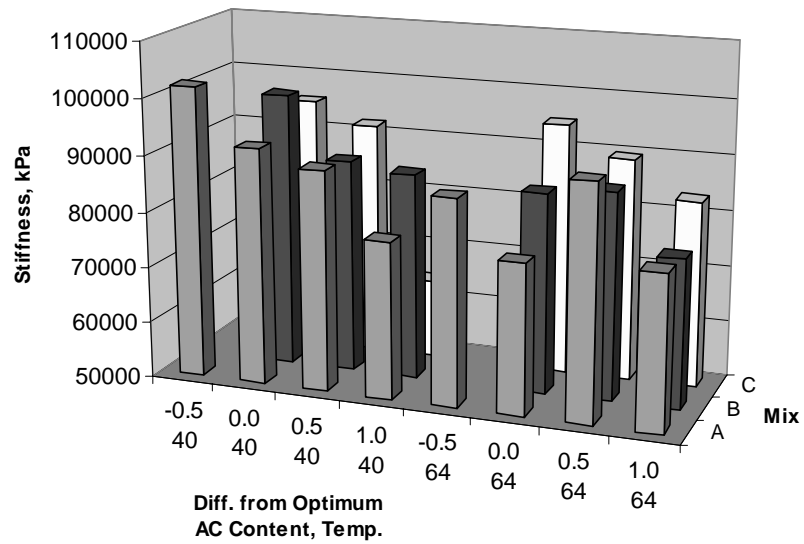


Figure 6.10: Phase IV Column Chart for Stiffness

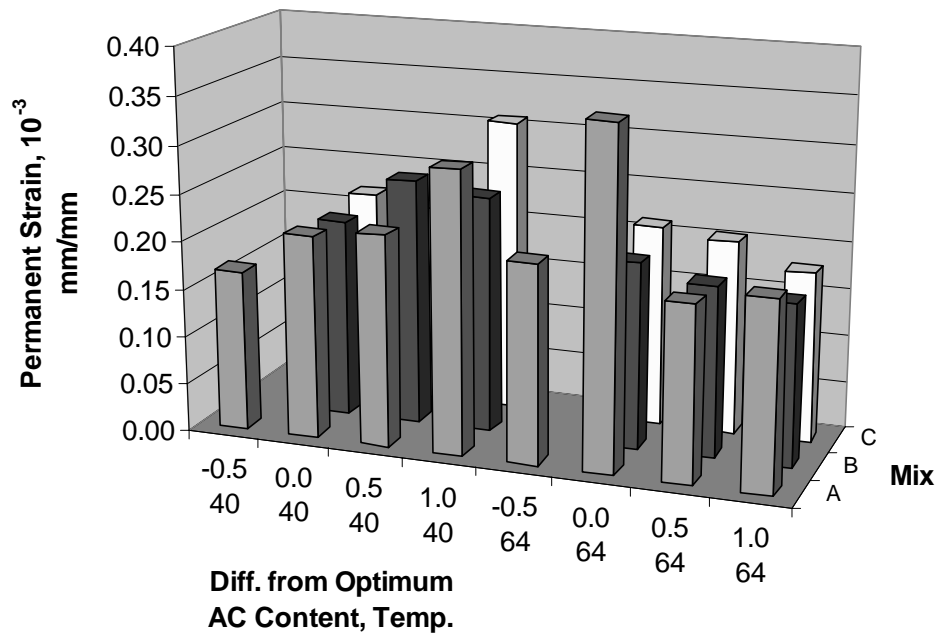


Figure 6.11: Phase IV Column Chart for Permanent Strain

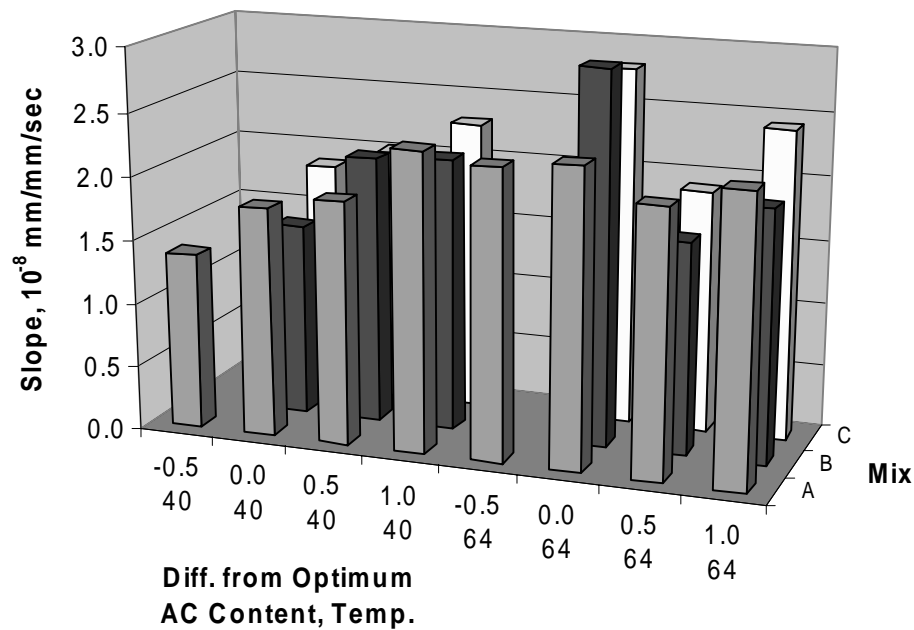


Figure 6.12: Phase IV Column Chart for Slope

There is a clear trend of decreasing stiffness, increasing permanent strain, and increasing slope with increasing asphalt content at 40 °C (104 °F). This finding is to be expected. However, the magnitude with which the responses change with asphalt content varies for Mix A, Mix B, and Mix C. The trends for stiffness, permanent strain, and slope can be seen more clearly in Figures 6.13, 6.14, and 6.15, respectively. Some of the “kinks” in the plots for 64 °C (147 °F) do not conform with the expected results, but are not entirely uncommon in a sensitivity analysis of this nature.

Mixes A, B, and C showed a 15, 12, and 33 percent decrease in stiffness, respectively, at 40 °C (104 °F). This finding shows that Mix C is more sensitive to changes in asphalt content. At 64 °C (147 °F), the decreases were -1, 11, and 12 percent for Mixes A, B, and C, respectively. It should be noted that only the decrease in stiffness for Mix C at 40 °C (104 °F) is of practical significance.

At 40 °C (104 °F), Mixes A, B, and C showed a 38, 19, and 41 percent increase in permanent strain, respectively. This shows that Mixes A and C are more sensitive to changes in asphalt content. At 64 °C (147 °F), decreases in permanent strain of 44, 13, and 16 percent were observed for Mixes A, B, and C, respectively. It is interesting to observe increases in permanent strain with a 1 percent increase in asphalt content at 40 °C (104 °F) and decreases in permanent strain with a 1 percent increase in asphalt content at 64 °C (147 °F). It should be noted that the changes in permanent strain are not of practical significance.

At 40 °C (104 °F), Mixes A, B, and C showed a 30, 41, and 23 percent increase in slope, respectively. This finding shows that the slopes of all the mixes are fairly sensitive to changes in asphalt content. At 64 °C (147 °F), decreases in slope of 3, 32, and 13 percent were observed for Mixes A, B, and C, respectively. It is interesting to observe increases in slope with a 1 percent increase in asphalt content at 40 °C (104 °F) and decreases in slope with a 1 percent increase in asphalt content at 64 °C (147 °F). It should be noted that the changes in slope are not of practical significance.

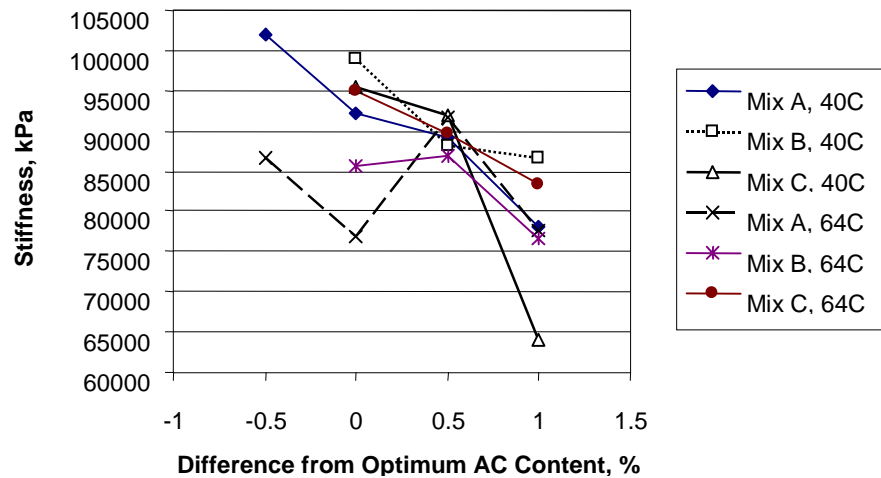


Figure 6.13: Phase IV Interaction between Mix and Temperature for Stiffness

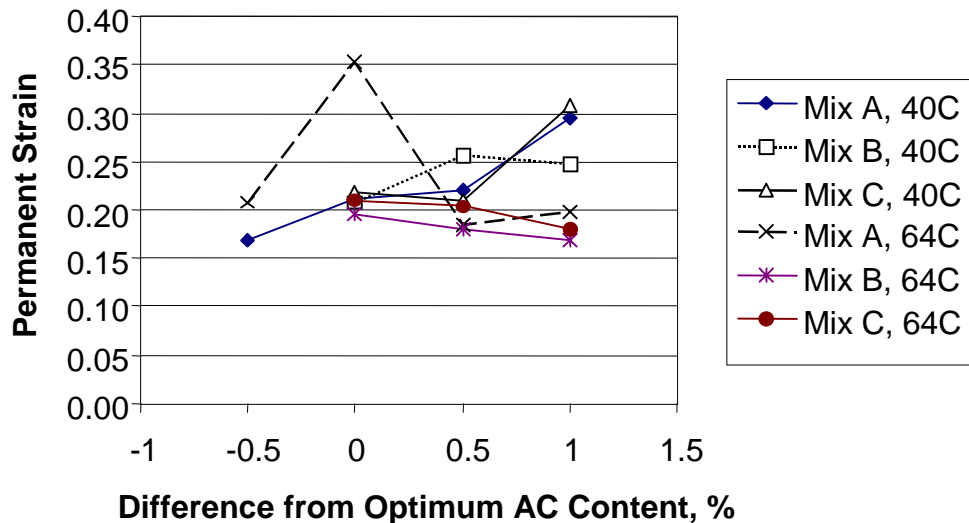


Figure 6.14: Phase IV Interaction between Mix and Temperature for Permanent Strain

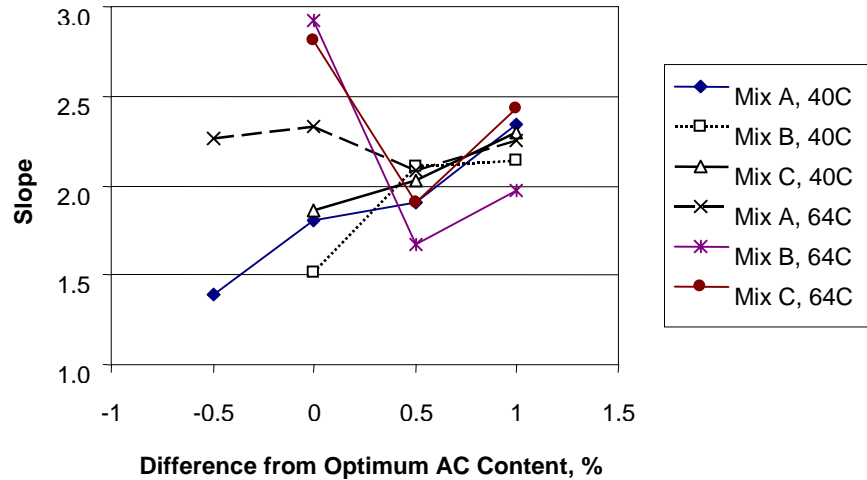


Figure 6.15: Phase IV Interaction between Mix and Temperature for Slope

#### 6.4 PHASE V RESULTS: GOOD VERSUS POOR MIX EVALUATION

In Phase V of this research effort, the effect of good and poor mix was evaluated. Three replicates for each combination of levels were tested. It should be noted that all the specimens were 150 mm (5.9 in.) in diameter and that they were all tested at 40 °C (104 °F) and 64 °C (147 °F). Table 6.4 shows the level at which the main effects or interactions are significant for stiffness, permanent deformation, and slope.

Table 6.4: Statistical Analysis for Phase V

Factor/Interaction	p-value		
	Stiffness	Perm. Strain	Slope
Mix	0.1675	0.9644	0.0077
Asphalt Content	0.2009	0.0591	0.0009
Temperature	0.6269	0.0009	0.1292
Mix*Asphalt Content	0.3032	0.5729	0.1646
Mix*Temperature	0.2502	0.9750	0.0185
Temperature*Asphalt Content	0.4012	0.0622	0.0013
Three-Factor Interaction	0.5766	0.4883	0.1388

For Phase V, temperature is significant at an  $\alpha$  level of 0.09 percent for permanent strain, and mix and asphalt content are significant (main effects) at an  $\alpha$  level of 1 percent.

Figures 6.16, 6.17, and 6.18 show the column charts for stiffness, permanent strain, and slope, respectively.

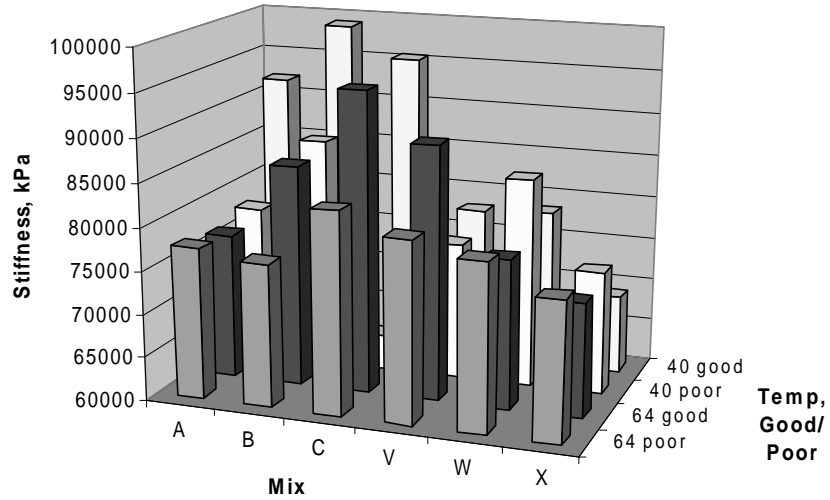


Figure 6.16: Phase V Column Chart for Stiffness

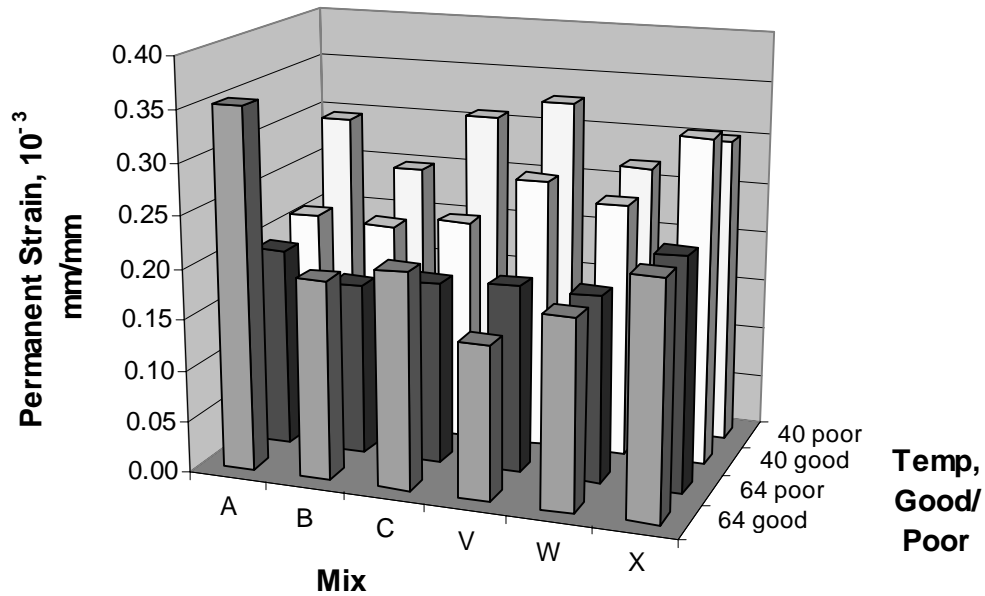


Figure 6.17: Phase V Column Chart for Permanent Strain

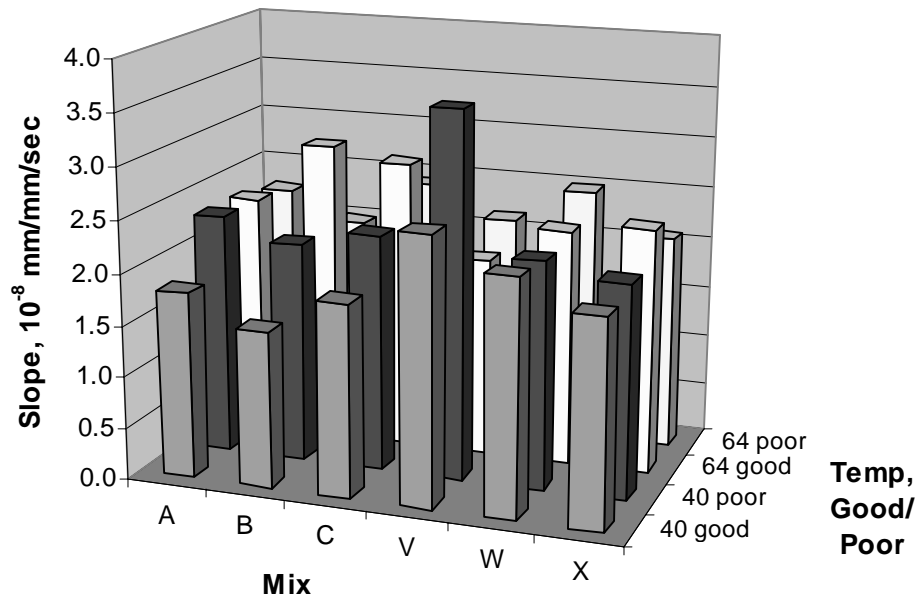


Figure 6.18: Phase V Column Chart for Slope

At 40 °C (104 °F), Mix A and Mix V (Superpave mixes) have a lower stiffness, higher permanent strain, and higher slope for the *poor* mixes compared to the corresponding *good* mix. Mixes B and C (intermediate and fine mixes from the sensitivity analysis in Phase IV) also show the same trends as those for mixes A and V. These results are expected. However, Mix W and Mix X (CMHB mixes) have a higher stiffness and practically similar permanent strain and slope for the *poor* mixes as compared to the corresponding *good* mix. This result is unexpected. For the good mixes, the Superpave mixes have higher stiffnesses than the CMHB-C mixes, though there are no clear trends in the permanent strain and slope. It should be noted, however, that mix stiffnesses are highly dependent on aggregate and asphalt type and quality, which differ between mixes in this study. No conclusions can or should be made regarding a comparison between Superpave and CMHB mixes based on this study. It should also be noted that the differences between *good* and *poor* mixes are not of practical significance, except for Mix C.

At 64 °C (147 °F), the trends are less obvious and the differences observed are less significant than those observed at 40 °C (104 °F). This finding is surprising. At higher temperatures, the asphalt binder is less stiff and is more likely to deform. One possible explanation for this result is the low stress level used in this test — 69 kPa (10 psi). AAMAS and other studies have used a stress level of 207 kPa (30 psi) for unconfined uniaxial static creep tests. It is possible that a higher stress level will increase the difference in the responses for this static creep test.

## **6.5 PHASE VI RESULTS: DEVELOPMENT OF ACCEPTANCE CRITERIA**

As mentioned in Chapter 5, an effort was made to develop acceptance criteria for Superpave mixes, if possible. However, the results from this research effort have not shown practical differences in the measured responses. Consequently, clearly defined acceptance criteria cannot be developed based on the findings presented herein. However, the lessons learned from this research effort are discussed in Chapter 7, and recommendations are made for future efforts to develop acceptance criteria.



## CHAPTER 7. CONCLUSIONS AND RECOMMENDATIONS

This research effort sought to answer a number of questions regarding the static creep behavior of Superpave mixes. Phase I involved modifications to the current TxDOT static creep tester and to the SST. Conclusions from Phases II through V are listed below and are followed by a discussion of the overall conclusions and recommendations for future research.

### 7.1 CONCLUSIONS FROM PHASES II THROUGH V

#### *Conclusions from Phase II*

All main effects and interactions were statistically significant for stiffness at an  $\alpha$  level of 5 percent. However, there were no practical differences in stiffness.

Mix and diameter (main effects) and the interaction between mix and diameter for permanent strain were statistically significant at an  $\alpha$  level of 5 percent. The differences owing to diameter — differences resulting from the interaction between diameter and mix — were of practical significance in some cases but not in other cases. The differences resulting from machine type were not of practical significance.

Mix and diameter (main effects) and the interaction between mix and diameter for slope were statistically significant at an  $\alpha$  level of 5 percent. The differences resulting from diameter were significant in some cases and not significant in other cases. This finding was a result of the interaction between diameter and mix. The differences resulting from machine type were of practical significance in most cases.

#### *Conclusions from Phase III*

Mix and machine (main effects) were statistically significant for stiffness, permanent strain, and slope at an  $\alpha$  level of 5 percent. However, there were no practical differences for either of the two statistically significant main effects for both stiffness and permanent strain. For slope, the SST gave slope values that were consistently higher and of practical significance. There was no practical difference between slope values between sample preparation methods.

The interaction between mix and sample preparation and mix and machine was also statistically significant at an  $\alpha$  level of 5 percent.

#### *Conclusions from Phase IV*

Asphalt content was statistically significant for stiffness, permanent strain, and slope at an  $\alpha$  level of 5 percent. At 40 °C (104 °F), there was a clear trend of decreasing stiffness, increasing permanent strain, and increasing slope with increasing asphalt content. However, only one mix showed differences of practical significance. At 64 °C (147 °F) there was a decrease in stiffness, permanent strain, and slope from optimum to optimum plus 1 percent asphalt. However, several “kinks” in the data were observed at the optimum plus 0.5 percent asphalt content.

Temperature and the interaction between asphalt content and temperature were also significant at an  $\alpha$  level of 5 percent for slope.

### ***Conclusions from Phase V***

Temperature was statistically significant for permanent strain, and asphalt content was statistically significant for slope at an  $\alpha$  level of 5 percent. At 40 °C (104 °F), the *good* Superpave mixes and the two other base mixes have higher stiffnesses, lower permanent strains, and lower slopes than the corresponding *poor* mixes. The two coarse matrix high binder (CMHB) mixes exhibited the opposite behavior. Overall, these differences were not of practical significance. At 64 °C (147 °F), the *good* mixes exhibited similar (from a practical standpoint) if not higher stiffnesses, lower permanent strains, and lower slopes than the *poor* mixes.

## **7.2 CONCLUSIONS AND RECOMMENDATIONS**

In the study of diameters and testing machines, only the effect of machine on slope was of practical significance in almost all cases. Based on the limited scope of this study, it is concluded that the SGC specimens molded to 51 mm (2 in.) in height exhibit creep properties similar to those of a Texas gyratory compactor specimen compacted to the same height. However, there were practical differences in slope values between specimens tested on the ETS and on the SST.

In the study on sample preparation and testing machine, only the effect of machine on slope was consistently of practical significance. Based on the limited scope of this study, it could be concluded that sample preparation methods (*molded* or *cut*) do not affect creep properties of SGC-compacted mixes from a practical standpoint. However, there were practical differences in slope values between specimens tested on the ETS and on the SST.

In the study on asphalt content and temperature, there was a consistent decrease in stiffness, increase in permanent strain, and increase in slope with increasing asphalt content for tests run at 40 °C (104 °F). However, most of the differences were of no practical significance. The differences observed at 64 °C (147 °F) were even less significant and much more inconsistent in terms of whether higher values were observed at 40 °C (104 °F) or 64 °C (147 °F).

In the study of *good* versus *poor* mixes, it was observed that *poor* Superpave mixes exhibited lower stiffnesses, higher permanent strains, and higher slopes as compared with the corresponding *good* mixes. The intermediate and fine mixes from the sensitivity analysis in Phase IV also show the same trends. However, the *poor* CMHB mixes exhibited higher stiffnesses and practically similar permanent strains and slopes for the *poor* mixes as compared to those in the corresponding *good* mixes. This finding could be interpreted to mean that the asphalt content used in the field may not be the optimum for resisting permanent deformation. However, in actual construction, many other factors play into the choice of asphalt content.

### 7.3 FUTURE RESEARCH

As suggested in the conclusions and recommendations, agencies are cautioned against blindly accepting the conclusions of this research effort. Adequate care must be taken to identify the context of the recommendations and adapt them to each unique set of circumstances. In order to extend the applicability of these conclusions, additional research is required to verify the findings for a broader range of materials (asphalt and aggregates). Only then can the findings be applied in a broader sense.

One underlying comment throughout the discussion of results and conclusions is the absence of practical differences in the results, even in the presence of statistical differences. A concern in the current test setup is the fairly large variability mentioned in Chapter 6. With a rather large variation between the three replicates, it is hard to say that one mean is very different from another. One possible solution would be to increase the stress level at which the test is run. Some other methods, including the Asphalt Aggregate Mixture Analysis System (AAMAS), have used a stress level of 207 kPa (30 psi), and this stress level should be investigated in the future. Such a stress level will make the test more of a “torture test” and possibly help to better differentiate between mixes.

Another factor of interest is the aspect ratio. Owing to limitations in the testing equipment, the height of the test specimen was maintained at approximately 51 mm (2 in.), even though the diameter was increased from 100 mm (3.9 in.) to 150 mm (5.9 in.). This height reduced the aspect ratio of the test specimen from 0.5 to 0.33. Further research can be conducted where the aspect ratio is kept constant while the diameter of the specimen is increased. For a 150 mm (5.9 in.) diameter specimen, a specimen height of 75 mm (3.0 in.) is recommended to maintain a constant aspect ratio.

Owing to the lack of practical differences between the mixes, no attempt was made to develop acceptance criteria. However, it is hoped that the work performed in this project will contribute to the growing knowledge regarding the use of the static creep test as a means of evaluating the rut resistance of asphalt concrete mixes.



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## Appendix A

### Summary of Volumetric Properties and Static Creep Test Results

Sensitivity Analysis  
4 inch samples of SC (L) Blend  
ETS

<b>Common Volumetric Properties</b>	C1	C2	C3	C4	C5	Ave.	Std. Dev.	COV, %
Percent Asphalt, Pa	4.8	4.8	4.8	4.8	4.8			
Specific Gravity of Binder, Gb	1.031	1.031	1.031	1.031	1.031			
Stone Bulk Specific Gravity, Gsb	2.534	2.534	2.534	2.534	2.534			
Stone Effective Specific Gravity, Gse	2.634	2.634	2.634	2.634	2.634			
Voids in Mineral Aggregate, VMA	11.5	11.3	12.1	11.7	11.9	11.7	0.32	2.7
Voids Filled with Asphalt, VFA	58.4	57.5	60.2	59.0	59.5	58.9	1.03	1.8
Percent Absorbed Asphalt, Pba	1.5	1.5	1.5	1.5	1.5			
Percent Effective Asphalt, Pbe	3.3	3.3	3.3	3.3	3.3			
Percent Passing 0.075 mm sieve	6.9	6.9	6.9	6.9	6.9			
Dust Proportion	2.07	2.07	2.07	2.07	2.07			

**Test Data**

	C1	C2	C3	C4	C5	Ave.	Std. Dev.	COV, %
Mix Bulk Specific Gravity, Gmb	2.355	2.361	2.341	2.350	2.346	2.351	0.008	0.3
Ave. Max. Theo. Density, Gmm	2.451	2.451	2.451	2.451	2.451	2.451	N/A	N/A
Percent Density	96.1	96.3	95.5	95.9	95.7	95.9	0.32	0.3
Percent Air	3.9	3.7	4.5	4.1	4.3	4.1	0.32	7.7
Ave. Specimen Ht., mm	51.143	50.595	51.505	50.821	51.455	51.104	0.395	0.8
Ave. Specimen Ht., in.	2.014	1.992	2.028	2.001	2.026	2.012	0.016	0.8
Total Strain, (mm/mm)x1000	0.680	0.629	0.721	0.688	0.743	0.692	0.044	6.3
*Creep Stiffness, kPa	101279	108331	95699	100474	92048	99566	6166	6.2
*Permanent Strain, (mm/mm)x1000	0.34	0.29	0.31	0.27	0.39	0.32	0.05	14.7
Recovery Efficiency, %	50.5	53.9	57.6	60.2	48.0	54.0	4.99	9.2
*Slope of SS curve, (mm/mm/sec)xE8	3.7	3.5	3.6	2.7	3.9	3.5	0.46	13.2
Ave. Strain Difference, (mm/mm)xE6	133	134	135	95	264	152	65	42.5
Max. Strain Difference, (mm/mm)xE6	208	192	208	167	366	228	79	34.5

Sensitivity Analysis  
4 inch samples of SC (L) Blend  
MTS

<b><u>Common Volumetric Properties</u></b>	C6	C7	C8	C9	C10	Ave.	Std. Dev.	COV, %
Percent Asphalt, Pa	4.8	4.8	4.8	4.8	4.8			
Specific Gravity of Binder, Gb	1.031	1.031	1.031	1.031	1.031			
Stone Bulk Specific Gravity, Gsb	2.534	2.534	2.534	2.534	2.534			
Stone Effective Specific Gravity, Gse	2.634	2.634	2.634	2.634	2.634			
Voids in Mineral Aggregate, VMA	12.1	12.0	11.3	11.8	11.6	11.8	0.32	2.7
Voids Filled with Asphalt, VFA	60.2	59.9	57.5	59.4	58.5	59.1	1.10	1.9
Percent Absorbed Asphalt, Pba	1.5	1.5	1.5	1.5	1.5			
Percent Effective Asphalt, Pbe	3.3	3.3	3.3	3.3	3.3			
Percent Passing 0.075 mm sieve	6.9	6.9	6.9	6.9	6.9			
Dust Proportion	2.07	2.07	2.07	2.07	2.07			

**Test Data**

	C6	C7	C8	C9	C10	Ave.	Std. Dev.	COV, %
Mix Bulk Specific Gravity, Gmb	2.341	2.343	2.361	2.347	2.354	2.349	0.008	0.4
Ave. Max. Theo. Density, Gmm	2.451	2.451	2.451	2.451	2.451	2.451	N/A	N/A
Percent Density	95.5	95.6	96.3	95.8	96.0	95.8	0.34	0.4
Percent Air	4.5	4.4	3.7	4.2	4.0	4.2	0.34	8.1
Ave. Specimen Ht., mm	51.723	51.036	51.728	50.709	51.064	51.252	0.454	0.9
Ave. Specimen Ht., in.	2.036	2.009	2.037	1.996	2.010	2.018	0.018	0.9
Total Strain, (mm/mm)x1000	0.854	0.919	0.956	1.207	0.932	0.974	0.136	14.0
*Creep Stiffness, kPa	78539	73459	70223	56035	72624	70176	8466	12.1
*Permanent Strain, (mm/mm)x1000	0.32	0.35	0.42	0.49	0.41	0.40	0.07	16.6
Recovery Efficiency, %	63.1	61.4	56.3	59.4	56.5	59.3	2.99	5.0
*Slope of SS curve, (mm/mm/sec)xE8	3.1	3.1	3.1	3.0	3.7	3.2	0.28	8.8
Ave. Strain Difference, (mm/mm)xE6	255	379	172	571	254	326	156	47.7
Max. Strain Difference, (mm/mm)xE6	383	540	301	816	298	468	218	46.6

Sensitivity Analysis  
4 inch samples of SC (L) Blend  
SST

<b>Common Volumetric Properties</b>	C11	C12	C13	C14	C15	Ave.	Std. Dev.	COV, %
Percent Asphalt, Pa	4.8	4.8	4.8	4.8	4.8			
Specific Gravity of Binder, Gb	1.031	1.031	1.031	1.031	1.031			
Stone Bulk Specific Gravity, Gsb	2.534	2.534	2.534	2.534	2.534			
Stone Effective Specific Gravity, Gse	2.634	2.634	2.634	2.634	2.634			
Voids in Mineral Aggregate, VMA	11.4	12.0	12.0	11.7	11.7	11.8	0.25	2.1
Voids Filled with Asphalt, VFA	58.1	60.0	59.9	59.0	59.0	59.2	0.78	1.3
Percent Absorbed Asphalt, Pba	1.5	1.5	1.5	1.5	1.5			
Percent Effective Asphalt, Pbe	3.3	3.3	3.3	3.3	3.3			
Percent Passing 0.075 mm sieve	6.9	6.9	6.9	6.9	6.9			
Dust Proportion	2.07	2.07	2.07	2.07	2.07			

**Test Data**

	C11	C12	C13	C14	C15	Ave.	Std. Dev.	COV, %
Mix Bulk Specific Gravity, Gmb	2.357	2.342	2.343	2.350	2.350	2.348	0.006	0.3
Ave. Max. Theo. Density, Gmm	2.451	2.451	2.451	2.451	2.451	2.451	N/A	N/A
Percent Density	96.2	95.6	95.6	95.9	95.9	95.8	0.25	0.3
Percent Air	3.8	4.4	4.4	4.1	4.1	4.2	0.25	6.0
Ave. Specimen Ht., mm	51.239	51.398	51.317	51.055	51.493	51.300	0.166	0.3
Ave. Specimen Ht., in.	2.017	2.024	2.020	2.010	2.027	2.020	0.007	0.3
Total Strain, (mm/mm)x1000	0.715	0.677	0.749		0.699	0.710	0.030	4.3
*Creep Stiffness, kPa	98094	101427	94805		98134	98115	2703	2.8
*Permanent Strain, (mm/mm)x1000	0.34	0.37	0.40		0.34	0.36	0.03	7.9
Recovery Efficiency, %	52.4	45.3	46.6		51.4	48.9	3.49	7.1
*Slope of SS curve, (mm/mm/sec)xE8	5.3	4.2	4.9		4.7	4.8	0.46	9.6
Ave. Strain Difference, (mm/mm)xE6	6	14	18		14	13	5	38.7
Max. Strain Difference, (mm/mm)xE6	48	48	71		95	66	22	34.3

\*No data could be read from the output file for C14.

Sensitivity Analysis  
6 inch samples of SA (J) Blend  
ETS (saw-cut)

<b>Common Volumetric Properties</b>	A1	A2	A3			Ave.	Std. Dev.	COV, %
Percent Asphalt, Pa	5.1	5.1	5.1					
Specific Gravity of Binder, Gb	1.031	1.031	1.031					
Stone Bulk Specific Gravity, Gsb	2.545	2.545	2.545					
Stone Effective Specific Gravity, Gse	2.605	2.605	2.605					
Voids in Mineral Aggregate, VMA	13.0	13.5	13.8			13.4	0.40	3.0
Voids Filled with Asphalt, VFA	60.7	62.1	62.9			61.9	1.11	1.8
Percent Absorbed Asphalt, Pba	0.9	0.9	0.9					
Percent Effective Asphalt, Pbe	4.2	4.2	4.2					
Percent Passing 0.075 mm sieve	5.3	5.3	5.3					
Dust Proportion	1.26	1.26	1.26					

**Test Data**

	A1	A2	A3			Ave.	Std. Dev.	COV, %
Mix Bulk Specific Gravity, Gmb	2.334	2.321	2.313			2.323	0.011	0.5
Ave. Max. Theo. Density, Gmm	2.417	2.417	2.417			2.417	N/A	N/A
Percent Density	96.6	96.0	95.7			96.1	0.44	0.5
Percent Air	3.4	4.0	4.3			3.9	0.44	11.2
Ave. Specimen Ht., mm	52.365	50.826	49.775			50.989	1.303	2.6
Ave. Specimen Ht., in.	2.062	2.001	1.960			2.007	0.051	2.6
Total Strain, (mm/mm)x1000	0.592	0.559	0.598			0.583	0.021	3.6
*Creep Stiffness, kPa	115356	122415	114706			117492	4276	3.6
*Permanent Strain, (mm/mm)x1000	0.24	0.21	0.18			0.21	0.03	14.3
Recovery Efficiency, %	60.2	62.4	70.7			64.4	5.54	8.6
*Slope of SS curve, (mm/mm/sec)xE8	2.4	2.3	1.6			2.1	0.44	20.8
Ave. Strain Difference, (mm/mm)xE6	192	45	176			138	81	58.6
Max. Strain Difference, (mm/mm)xE6	249	84	229			187	90	48.1

Sensitivity Analysis  
6 inch samples of SA (J) Blend

Sensitivity Analysis  
6 inch samples of SA (J) Blend  
SST (saw-cut)

<b>Common Volumetric Properties</b>	A4	A5	A6			Ave.	Std. Dev.	COV, %
Percent Asphalt, Pa	5.1	5.1	5.1					
Specific Gravity of Binder, Gb	1.031	1.031	1.031					
Stone Bulk Specific Gravity, Gsb	2.545	2.545	2.545					
Stone Effective Specific Gravity, Gse	2.605	2.605	2.605					
Voids in Mineral Aggregate, VMA	13.3	13.7	13.3			13.4	0.23	1.7
Voids Filled with Asphalt, VFA	61.8	62.8	61.7			62.1	0.61	1.0
Percent Absorbed Asphalt, Pba	0.9	0.9	0.9					
Percent Effective Asphalt, Pbe	4.2	4.2	4.2					
Percent Passing 0.075 mm sieve	5.3	5.3	5.3					
Dust Proportion	1.26	1.26	1.26					

**Test Data**

	A4	A5	A6			Ave.	Std. Dev.	COV, %
Mix Bulk Specific Gravity, Gmb	2.324	2.314	2.325			2.321	0.006	0.3
Ave. Max. Theo. Density, Gmm	2.417	2.417	2.417			2.417	N/A	N/A
Percent Density	96.2	95.7	96.2			96.0	0.25	0.3
Percent Air	3.8	4.3	3.8			4.0	0.25	6.3
Ave. Specimen Ht., mm	51.682	49.217	50.570			50.490	1.234	2.4
Ave. Specimen Ht., in.	2.035	1.938	1.991			1.988	0.049	2.4
Total Strain, (mm/mm)x1000	0.413	0.670	0.543			0.542	0.129	23.7
*Creep Stiffness, kPa	167980	103283	126056			114670	16103	14.0
*Permanent Strain, (mm/mm)x1000	0.10	0.29	0.29			0.29	0.00	0.0
Recovery Efficiency, %	75.8	56.7	46.6			51.7	7.2	13.9
*Slope of SS curve, (mm/mm/sec)xE8	1.8	3.2	3.8			3.5	0.4	12.1
Ave. Strain Difference, (mm/mm)xE6	59	47	20			34	19	57.0
Max. Strain Difference, (mm/mm)xE6	118	99	72			86	19	22.3

\*Specimen A4 was not included in the statistical analysis because the results are for a re-test. The original test yielded no data in the datafile.

Sensitivity Analysis  
6 inch samples of SA (J) Blend

ETS (molded to 50.8 mm)

<b>Common Volumetric Properties</b>	S3	S4	S6			Ave.	Std. Dev.	COV, %
Percent Asphalt, Pa	5.1	5.1	5.1					
Specific Gravity of Binder, Gb	1.031	1.031	1.031					
Stone Bulk Specific Gravity, Gsb	2.545	2.545	2.545					
Stone Effective Specific Gravity, Gse	2.605	2.605	2.605					
Voids in Mineral Aggregate, VMA	13.2	13.0	13.1			13.1	0.10	0.8
Voids Filled with Asphalt, VFA	61.4	60.9	61.0			61.1	0.26	0.4
Percent Absorbed Asphalt, Pba	0.9	0.9	0.9					
Percent Effective Asphalt, Pbe	4.2	4.2	4.2					
Percent Passing 0.075 mm sieve	5.3	5.3	5.3					
Dust Proportion	1.26	1.26	1.26					

### **Test Data**

	S3	S4	S6			Ave.	Std. Dev.	COV, %
Mix Bulk Specific Gravity, Gmb	2.327	2.332	2.331			2.330	0.003	0.1
Ave. Max. Theo. Density, Gmm	2.417	2.417	2.417			2.417	N/A	N/A
Percent Density	96.3	96.5	96.4			96.4	0.11	0.1
Percent Air	3.7	3.5	3.6			3.6	0.11	3.0
Ave. Specimen Ht., mm	50.414	50.547	50.703			50.555	0.145	0.3
Ave. Specimen Ht., in.	1.985	1.990	1.996			1.990	0.006	0.3
Total Strain, (mm/mm)x1000	0.748	0.756	0.729			0.744	0.014	1.9
*Creep Stiffness, kPa	91716	90642	93963			92107	1695	1.8
*Permanent Strain, (mm/mm)x1000	0.20	0.23	0.21			0.21	0.02	7.2
Recovery Efficiency, %	72.9	69.8	71.7			71.5	1.56	2.2
*Slope of SS curve, (mm/mm/sec)xE8	1.2	3.0	1.2			1.8	1.04	57.7
Ave. Strain Difference, (mm/mm)xE6	280	251	117			216	87	40.3
Max. Strain Difference, (mm/mm)xE6	401	364	191			319	112	35.2

SST (molded to 50.8 mm)

<b>Common Volumetric Properties</b>	S1	S2	S5			Ave.	Std. Dev.	COV, %
Percent Asphalt, Pa	5.1	5.1	5.1					
Specific Gravity of Binder, Gb	1.031	1.031	1.031					
Stone Bulk Specific Gravity, Gsb	2.545	2.545	2.545					
Stone Effective Specific Gravity, Gse	2.605	2.605	2.605					
Voids in Mineral Aggregate, VMA	13.3	13.0	13.0			13.1	0.17	1.3
Voids Filled with Asphalt, VFA	61.7	60.8	60.7			61.1	0.55	0.9
Percent Absorbed Asphalt, Pba	0.9	0.9	0.9					
Percent Effective Asphalt, Pbe	4.2	4.2	4.2					
Percent Passing 0.075 mm sieve	5.3	5.3	5.3					
Dust Proportion	1.26	1.26	1.26					

### **Test Data**

	S1	S2	S5			Ave.	Std. Dev.	COV, %
Mix Bulk Specific Gravity, Gmb	2.325	2.333	2.334			2.331	0.005	0.2
Ave. Max. Theo. Density, Gmm	2.417	2.417	2.417			2.417	N/A	N/A
Percent Density	96.2	96.5	96.6			96.4	0.20	0.2
Percent Air	3.8	3.5	3.4			3.6	0.20	5.7
Ave. Specimen Ht., mm	51.058	50.578	51.142			50.926	0.304	0.6
Ave. Specimen Ht., in.	2.010	1.991	2.013			2.005	0.012	0.6
Total Strain, (mm/mm)x1000	0.753	0.785	0.716			0.751	0.035	4.6
*Creep Stiffness, kPa	91657	88837	96387			92294	3815	4.1
*Permanent Strain, (mm/mm)x1000	0.30	0.36	0.25			0.30	0.06	18.2
Recovery Efficiency, %	60.2	54.1	65.1			59.8	5.48	9.2
*Slope of SS curve, (mm/mm/sec)xE8	3.2	4.1	3.6			3.6	0.45	12.4
Ave. Strain Difference, (mm/mm)xE6	63	10	47			40	27	68.0
Max. Strain Difference, (mm/mm)xE6	120	48	96			88	37	41.7

## Sensitivity Analysis

6 inch samples of SM Blend (SA + 0.5% AC)

ETS (molded to 50.8 mm)

<b>Common Volumetric Properties</b>	M5	M7	M10			Ave.	Std. Dev	COV, %
Percent Asphalt, Pa	5.6	5.6	5.6					
Specific Gravity of Binder, Gb	1.031	1.031	1.031					
Stone Bulk Specific Gravity, Gsb	2.545	2.545	2.545					
Stone Effective Specific Gravity, Gse	2.596	2.596	2.596					
Voids in Mineral Aggregate, VMA	14.5	14.7	14.7			14.6	0.12	0.8
Voids Filled with Asphalt, VFA	61.5	61.4	61.9			61.6	0.26	0.4
Percent Absorbed Asphalt, Pba	0.8	0.8	0.8					
Percent Effective Asphalt, Pbe	4.8	4.8	4.8					
Percent Passing 0.075 mm sieve	5.3	5.3	5.3					
Dust Proportion	1.09	1.09	1.09					

## Test Data

	M5	M7	M10			Ave.	Std. Dev	COV, %
Mix Bulk Specific Gravity, Gmb	2.304	2.305	2.300			2.303	0.003	0.1
Ave. Max. Theo. Density, Gmm	2.393	2.393	2.393			2.393	N/A	N/A
Percent Density	96.3	96.3	96.1			96.2	0.11	0.1
Percent Air	3.7	3.7	3.9			3.8	0.11	2.9
Ave. Specimen Ht., mm	51.065	50.762	50.589			50.805	0.241	0.5
Ave. Specimen Ht., in.	2.010	1.999	1.992			2.000	0.009	0.5
Total Strain, (mm/mm)x1000	0.659	0.785	0.893			0.779	0.117	15.0
*Creep Stiffness, kPa	103983	87226	76619			89276	13797	15.5
*Permanent Strain, (mm/mm)x1000	0.14	0.26	0.27			0.22	0.07	32.4
Recovery Efficiency, %	78.7	67.4	70.2			72.1	5.88	8.2
*Slope of SS curve, (mm/mm/sec)xE8	1.9	2.0	1.8			1.9	0.10	5.3

## Sensitivity Analysis

6 inch samples of SN Blend (SA - 0.5% AC)

ETS (molded to 50.8 mm)

<b>Common Volumetric Properties</b>	N5	N6	N9			Ave.	Std. Dev	COV, %
Percent Asphalt, Pa	4.6	4.6	4.6					
Specific Gravity of Binder, Gb	1.031	1.031	1.031					
Stone Bulk Specific Gravity, Gsb	2.545	2.545	2.545					
Stone Effective Specific Gravity, Gse	2.601	2.601	2.601					
Voids in Mineral Aggregate, VMA	12.5	12.7	12.9			12.7	0.20	1.6
Voids Filled with Asphalt, VFA	63.1	63.7	64.3			63.7	0.60	0.9
Percent Absorbed Asphalt, Pba	0.9	0.9	0.9					
Percent Effective Asphalt, Pbe	3.8	3.8	3.8					
Percent Passing 0.075 mm sieve	5.3	5.3	5.3					
Dust Proportion	1.41	1.41	1.41					

## Test Data

	N5	N6	N9			Ave.	Std. Dev	COV, %
Mix Bulk Specific Gravity, Gmb	2.335	2.330	2.324			2.330	0.006	0.2
Ave. Max. Theo. Density, Gmm	2.431	2.431	2.431			2.431	N/A	N/A
Percent Density	96.1	95.8	95.6			95.8	0.23	0.2
Percent Air	3.9	4.2	4.4			4.2	0.23	5.4
Ave. Specimen Ht., mm	50.831	51.215	50.844			50.963	0.218	0.4
Ave. Specimen Ht., in.	2.001	2.016	2.002			2.006	0.009	0.4
Total Strain, (mm/mm)x1000	0.577	0.690	0.772			0.680	0.098	14.4
*Creep Stiffness, kPa	118181	99099	88638			101973	14980	14.7
*Permanent Strain, (mm/mm)x1000	0.15	0.17	0.19			0.17	0.02	11.8
Recovery Efficiency, %	74.8	75.0	75.9			75.2	0.59	0.8
*Slope of SS curve, (mm/mm/sec)xE8	1.3	1.6	1.3			1.4	0.17	12.4

## Sensitivity Analysis

6 inch samples of SP Blend (SB + 0.5% AC)

ETS (molded to 50.8 mm)

<b>Common Volumetric Properties</b>	P5	P6	P10			Ave.	Std. Dev	COV, %
Percent Asphalt, Pa	5.3	5.3	5.3					
Specific Gravity of Binder, Gb	1.031	1.031	1.031					
Stone Bulk Specific Gravity, Gsb	2.542	2.542	2.542					
Stone Effective Specific Gravity, Gse	2.609	2.609	2.609					
Voids in Mineral Aggregate, VMA	13.2	13.3	13.3			13.3	0.06	0.4
Voids Filled with Asphalt, VFA	59.7	60.1	60.3			60.0	0.31	0.5
Percent Absorbed Asphalt, Pba	1.0	1.0	1.0					
Percent Effective Asphalt, Pbe	4.3	4.3	4.3					
Percent Passing 0.075 mm sieve	5.7	5.7	5.7					
Dust Proportion	1.32	1.32	1.32					

## Test Data

	P5	P6	P10			Ave.	Std. Dev	COV, %
Mix Bulk Specific Gravity, Gmb	2.331	2.328	2.326			2.328	0.003	0.1
Ave. Max. Theo. Density, Gmm	2.413	2.413	2.413			2.413	N/A	N/A
Percent Density	96.6	96.5	96.4			96.5	0.10	0.1
Percent Air	3.4	3.5	3.6			3.5	0.10	3.0
Ave. Specimen Ht., mm	50.841	50.621	50.723			50.728	0.110	0.2
Ave. Specimen Ht., in.	2.002	1.993	1.997			1.997	0.004	0.2
Total Strain, (mm/mm)x1000	0.773	0.738	0.820			0.777	0.041	5.3
*Creep Stiffness, kPa	88387	92455	83504			88115	4482	5.1
*Permanent Strain, (mm/mm)x1000	0.28	0.22	0.28			0.26	0.03	13.3
Recovery Efficiency, %	64.4	70.0	66.3			66.9	2.85	4.3
*Slope of SS curve, (mm/mm/sec)xE8	2.3	1.9	2.2			2.1	0.21	9.8

## Sensitivity Analysis

6 inch samples of SR Blend (SC + 0.5% AC)

ETS (molded to 50.8 mm)

<b>Common Volumetric Properties</b>	R5	R7	R10			Ave.	Std. Dev	COV, %
Percent Asphalt, Pa	5.3	5.3	5.3					
Specific Gravity of Binder, Gb	1.031	1.031	1.031					
Stone Bulk Specific Gravity, Gsb	2.534	2.534	2.534					
Stone Effective Specific Gravity, Gse	2.631	2.631	2.631					
Voids in Mineral Aggregate, VMA	12.7	12.3	12.7			12.6	0.23	1.8
Voids Filled with Asphalt, VFA	58.3	56.7	58.3			57.8	0.92	1.6
Percent Absorbed Asphalt, Pba	1.5	1.5	1.5					
Percent Effective Asphalt, Pbe	3.9	3.9	3.9					
Percent Passing 0.075 mm sieve	6.9	6.9	6.9					
Dust Proportion	1.78	1.78	1.78					

## Test Data

	R5	R7	R10			Ave.	Std. Dev	COV, %
Mix Bulk Specific Gravity, Gmb	2.336	2.348	2.336			2.340	0.007	0.3
Ave. Max. Theo. Density, Gmm	2.431	2.431	2.431			2.431	N/A	N/A
Percent Density	96.1	96.6	96.1			96.3	0.28	0.3
Percent Air	3.9	3.4	3.9			3.7	0.28	7.6
Ave. Specimen Ht., mm	50.529	50.718	50.626			50.624	0.095	0.2
Ave. Specimen Ht., in.	1.989	1.997	1.993			1.993	0.004	0.2
Total Strain, (mm/mm)x1000	0.746	0.805	0.693			0.748	0.056	7.5
*Creep Stiffness, kPa	91726	85130	98873			91910	6873	7.5
*Permanent Strain, (mm/mm)x1000	0.20	0.24	0.19			0.21	0.03	12.6
Recovery Efficiency, %	72.7	70.7	72.6			72.0	1.13	1.6
*Slope of SS curve, (mm/mm/sec)xE8	2.1	2.1	1.8			2.0	0.17	8.7