Rational Test Methods for Predicting Permanent Deformation in Asphalt Concrete Pavement
### Abstract

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In this report the existing test methods are reviewed to understand the mechanisms of permanent deformation in asphalt concrete. Based upon this information, the uniaxial compression creep/recovery tests are designed to characterize the mechanical behavior of the asphalt concrete. The characteristics of permanent strains are analyzed as inferred from test results. It is shown that the separation of the strains into the time independent elastic and plastic strains and the time dependent viscoelastic strains requires tests with various unloading times. The analysis is illustrated with results of a series of tests on one asphalt concrete mixture.

The tested material exhibits temperature dependent elastic and plastic strains that are proportional to the level of stress. The time and temperature dependent viscoelastic strains are nonlinearly related to the stress level and stress history. An approximate constitutive equation which disregards this nonlinearity is presented.
RATIONAL TEST METHODS
FOR
PREDICTING PERMANENT DEFORMATION
IN
ASPHALT CONCRETE PAVEMENT

Final Report

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In this report the existing test methods are reviewed to understand the mechanisms of permanent deformation in asphalt concrete. Based upon this information, the uniaxial compression creep/recovery tests are designed to characterize the mechanical behavior of the asphalt concrete. The characteristics of permanent strains are analyzed as inferred from test results. It is shown that the separation of the strains into the time independent elastic and plastic strains and the time dependent viscoelastic strains requires tests with various unloading times. The analysis is illustrated with results of a series of tests on one asphalt concrete mixture.

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CHAPTER 1
INTRODUCTION

There are vast challenges facing those administrators, engineers, and technicians who are responsible for maintaining the pavements portion of the nation's infrastructure. The materials used in pavements are changing with respect to quality and type, and the loading conditions for pavements are changing as legislation and transport technology change. However, the evolution of tools and knowledge which engineers use for evaluating materials and making design decisions have lagged behind these changes. There has been a recognition of these deficiencies in the past five to ten years and measures are being taken to correct some of these.

Most of the test methods currently used for pavement materials are the products of research conducted twenty to fifty years ago. They were designed to be simple tests which could be performed with the technology available at that time. The results of these were correlated to extensive observations of pavement performance in order to determine what constituted "acceptable results." However, the measurements from these products were reported in the rather nebulous terms of stability and flow, which relate only indirectly to fundamental engineering quantities such as stress and strain. Thus, while the test methods were able to adequately predict the performance of the materials under the conditions which existed at that time, they did not have the flexibility to properly characterize new materials or predict performance under new conditions.

Material types used in pavements are in a paradoxical state in that technological improvements are being made at the same time that the
quality of the raw materials may be diminishing. The use of polymer and mineral additives in asphalt cement seems to hold the promise of beneficiating some of the characteristics of asphalt concrete pavements, while arguments have been made that the quality of asphalt cements produced since the oil embargo of mid-1970's seems to be declining. To further complicate the issue, quality aggregate sources in the U.S. are being depleted, especially in metropolitan areas. A partial response to these problems has been an increased use of recycled pavement materials. Thus, it can be concluded that the standard materials of the past will not be representative of the current or future materials.

Loading conditions for pavements are likewise in a state of flux. Truck tire pressures and loads are increasing in order to improve the efficiency of transporting goods. Furthermore, as the use of rail transportation declined over a number of years, more trucks have been needed to fill the gap. Dual tire configurations are being replaces by "super-single" tires to reduce truck maintenance costs, and this has the same effect as increasing tire pressures since it tends to concentrate loads applied to the pavement surface over a smaller area. Trucking firms are also switching from bias-ply to radial tires because of the increased reliability. Radial tires tend to "track" more consistently than bias-ply tires which means that loads more frequently pass over the same point on the pavement surface. Changes in pavement loading conditions require that the performance criteria for pavements be based upon models which can accommodate these changes while maintaining their accuracy.

One method of accomplishing this is to use mechanistic methods of modelling pavement systems and identifying the fundamental material
properties required to apply them. An approach to this has already been initiated through the use of the resilient modulus test for characterizing pavement materials and using the results as input into layered elastic models. This method has worked effectively in predicting one type of load related pavement distress known as fatigue cracking. However, the purely elastic approach is inappropriate for the other major type of traffic related distress which is permanent deformation.

Permanent deformation typically manifests itself as depressions in the wheelpaths of a pavement called ruts. These result when the loads passing over a point in the pavement are sufficiently large to cause the materials in one or more layers to move without recovering to their original positions. In recent years, the problem has been more acute with respect to asphalt concrete than other materials. Ironically, this is partially due to the use of the layered elastic approach to pavement design in which the strategy is to increase the thickness of the asphalt concrete layer in order to minimize the bending strain in the surface layer. By increasing the thickness of the bituminous layer, designers have been increasing the probability of rutting because the vertical displacement, i.e., the depth of the depression, increases for a given load with the thickness of the layer.

This report describes an approach for laboratory evaluation of the permanent deformation characteristics of asphalt concrete. Fundamental material properties are identified which may be used in more advanced mechanistic models to predict the potential for pavement rutting. These models could then be applied for designing a field test that would allow for a direct rutting risk evaluation in asphalt concrete pavements.
The objective of this research is not only to review the existing test methods but also to develop one or more suitable test methods for evaluating the mechanical characteristics of asphalt concrete that are critical in understanding the potential for rutting. The uniaxial static creep/recovery tests were performed and the results from these tests are in the form of fundamental material properties which can be used in mechanistic pavement models.

This report is divided into five chapters. This chapter discusses the basis for the research and the research objective. Chapter 2 is the literature review which summarizes several types of permanent deformation in asphalt concrete pavements. Five existing test methods and some formulae for calculating the magnitude of permanent strains or the depth of rutting are presented here. Chapter 3 deals with the laboratory tests; information on the materials and test method are presented. Chapter 4 analyzes the test results. The material behavior of the asphalt concrete due to the uniaxial static creep/recovery test is discussed. Chapter 5 presents conclusions and recommendations which are drawn from this research. References and appendices are included at the end of the report.
CHAPTER 2
LITERATURE REVIEW

The literature review is presented and summarized in this chapter. It contains five sections: Section 1 deals with the mechanism of permanent deformation of asphalt concrete. It shows how the permanent deformation takes place, what are the possible causes, the types of permanent deformation, and which traffic loads are related to these deformations. Section 2 discusses permanent deformation from a plasticity viewpoint. The bearing capacity relationship of pavements is discussed here. Section 3 describes permanent deformation due to asphalt concrete viscosity. The uncoupled elastic-viscous and viscoelastic approaches are discussed in this section. Section 4 reviews five main test methods which can be used to evaluate the asphalt concrete properties responsible for rutting. Section 5 gives examples of empirical formulae that can be used for predicting the depth of rutting.

2.1 Mechanism of Permanent Deformation

Traffic loads cause permanent deformation in asphalt concrete pavements. Monismith and McLean [1] have classified traffic associated permanent deformations into 3 types:

A. shear deformation
B. creep deformation
C. rutting
Shear deformation is associated with plastic flow due to single or comparatively few excessive loads. Creep deformation is time dependent and due to long-term or static load. Rutting deformation is the accumulation of small permanent deformation due to a large number of repetitions of the traffic load.

Permanent deformation in the form of rutting usually develops gradually with repeated traffic load applications and appears as a channeling of the pavement surface in the longitudinal direction in the wheelpath (Fig. 2.1). It may be associated with:

1. lack of asphalt concrete resistance to permanent deformation under high temperature, high tire pressures and large numbers of load repetitions,

2. lack of sufficient strength in underlying layers

The possible causes of these are:

1. structural design
2. asphalt (binder) properties
3. aggregate properties
4. asphalt concrete (mixture) properties
5. compaction of all layers

Permanent deformation visible as rutting may be due to the accumulation of permanent strains in all layers of the asphalt pavement structure. However, the following discussion concentrates on permanent deformations in the asphalt concrete layer.

A physical deformation model for an asphalt concrete is based on the assumption that any deformations in the asphalt concrete are a result of
Figure 2.1. Rutting in Asphalt Concrete Pavement
sliding displacement between adjacent mineral aggregates, separated by a thin film of asphalt [2]. This model is schematically illustrated in Fig.2.2. In this model, the accumulation of deformation with time is dependent on the asphalt properties as well as the magnitude of the applied stress and the thickness of the asphalt film which changes as a function of time.

Brown and Cooper [3] showed that the permanent deformation in asphalt concrete pavements is significantly influenced by the voids in mineral aggregate (VMA) (Fig.2.3). Low VMA of the asphalt concrete yields better resistance to permanent deformation. However, the asphalt viscosity, the asphalt content, the aggregate type, the aggregate shape, the aggregate grading, and the interlocking of aggregate in the asphalt mixture influence the permanent deformation.

In mechanistic terms, permanent deformations of asphalt concrete, or more precisely strains, are due to its two properties: a) plasticity, and b) viscosity. The asphalt concrete plasticity related permanent strains are time independent, being only a function of the magnitude of stresses induced by external load. If the stresses reach a certain threshold called the yield criterion, plastic deformation may occur. The asphalt concrete viscosity related permanent strains are a function of time and level of the applied load. They increase with the time of constant load applications (static creep), and with the number of repetitions of short-duration cyclic loads (dynamic creep).
Figure 2.2. Sliding Displacement between Two Aggregates
$V_{ma} =$ Volume of voids in mineral aggregate
$V_{mb} =$ Bulk volume of compacted mix
$V_{mm} =$ Voidless volume of paving mix
$V_a =$ Volume of air voids
$V_b =$ Volume of asphalt
$V_{ba} =$ Volume of absorbed asphalt
$V_{sb} =$ Volume of mineral aggregate (by bulk specific gravity)
$V_{se} =$ Volume of mineral aggregate
(by effective specific gravity)

Figure 2.3. Representation of Volumes in A Compacted Asphalt
Concrete Specimen (after [4])
2.2 Permanent Deformation Due to Plasticity of Asphalt Concrete

Plasticity related permanent strains in asphalt concrete are associated with a single or comparatively few excessive loads. They occur when the stresses induced by the wheel load surpass the yield criterion of the asphalt concrete. The concept of plasticity related permanent strains can be illustrated with example of a sliding block element (Fig.2.4). The relative displacement of the two parts of the block can only occur if the applied load surpasses the interface resistance; the yield criterion. The yield criterion of the asphalt concrete is usually identified with the Mohr-Coulomb strength criterion (Fig.2.5):

\[ \tau = \sigma \tan \phi + c \]  \hspace{1cm} (2.1)

where

\[ \tau = \text{shear stress} \]
\[ \sigma = \text{normal stress} \]
\[ \phi = \text{internal friction angle of asphalt concrete} \]
\[ c = \text{cohesion of asphalt concrete} \]

Expression (2.1) relates shear and normal stresses on a plane of failure. With the help of Mohr’s diagram (Fig.2.5), eq.(2.1) can be expressed in terms of principal stresses that act in a material element as:

\[ \sigma_1 - \sigma_3 = (\sigma_1 + \sigma_3) \sin \phi + 2c \cos \phi \]  \hspace{1cm} (2.2)
Figure 2.4. Sliding Block Element
Figure 2.5. Mohr-Coulomb Diagram
where
\[
\sigma_1 = \text{major principal stress} \\
\sigma_3 = \text{minor principal stress}
\]

Inspection of eq.(2.2) indicates that the Mohr-Coulomb yield criterion is independent of the magnitude of the intermediate principal stress \(\sigma_2\). To account for the influence of the intermediate principal stress \(\sigma_2\) on plastic yielding, Carpenter and Freeman [5], and Ameri-Gaznon and Little [6] suggested another form of the yield criterion, namely, the \textit{octahedral strength criterion}:

\[
\tau_{\text{oct}} = \sigma_{\text{oct}} \tan \phi' + c' \tag{2.3}
\]

where
\[
\tau_{\text{oct}} = \text{octahedral shear stress} \\
\sigma_{\text{oct}} = \text{octahedral normal stress} \\
\phi' = \text{transformed internal friction angle of asphalt concrete} \\
c' = \text{transformed cohesion of asphalt concrete}
\]

The octahedral shear and normal stresses act on a plane that is equally inclined to all three principal stresses (Fig.2.6) and, therefore, are functions of all three principal stresses:

\[
\tau_{\text{oct}} = \frac{1}{3} \left[ (\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 \right]^{1/2} \tag{2.4}
\]
Figure 2.6. Octahedral Plan
\[
\sigma_{oct} = \frac{1}{3} (\sigma_1 + \sigma_2 + \sigma_3)
\] (2.5)

The angle of internal friction \( \phi \), and the cohesion \( c \), appearing in eq.(2.2) can be determined from triaxial compression tests. Also the transformed angle of internal friction \( \phi' \), and the transformed cohesion \( c' \), appearing in eq.(2.3) can be determined from triaxial compression tests. Since in this test \( \sigma_2 = \sigma_3 \), the expressions for the octahedral shear and normal stresses given above reduce to:

\[
\tau_{oct} = \frac{\sqrt{2}}{3} (\sigma_1 - \sigma_3) = 0.47 (\sigma_1 - \sigma_3)
\] (2.6)

\[
\sigma_{oct} = \frac{1}{3} (\sigma_1 + 2\sigma_3) = 0.333 (\sigma_1 + 2\sigma_3)
\] (2.7)

and eq.(2.3) can be written in a form similar to eq.(2.2) as:

\[
\sigma_1 - \sigma_3 = 0.708 (\sigma_1 + 2\sigma_3) \sin \phi' + c'
\] (2.8)

The yield criterion (2.2) or (2.6) can be used to determine point(s) in the asphalt concrete layer where plastic yielding may occur [6]. It should be stressed here, however, that the yield criterion alone gives no information on the magnitude of plastic strains. A rule for plastic strains growth is required; this issue has not been addressed in analyzing plastic strains in asphalt concrete.
Without knowing this rule, however, it is possible to determine the pressure applied to the surface of an asphalt pavement causing its plastic failure. This can be done using the *bearing capacity* formula (Fig.2.7), whose derivation is based on eq.(2.2), and which is widely used in foundation engineering [7, 8, 9]:

\[
q_{ult} = c N_c
\]  
(2.9)

where

\[
q_{ult} = \text{ultimate bearing capacity of asphalt concrete}
\]
(ultimate pressure that can be applied)

\[
N_c = \text{bearing capacity factor}
\]

The bearing capacity factor \( N_c \) depends on the friction angle \( \phi \) and on the shape of the loading area. The bearing capacity factors for a strip loading area were given, for example, by Nijboer [7], and for a circular loading area by Smith [8] and Saal [9]. The use of the bearing capacity formula in permanent deformation prediction is limited, however, since it gives no indication of the actual amount of surface permanent displacement associated with pavement failure.

2.3 Permanent Deformation Due to Viscosity of Asphalt Concrete

*Viscosity* related permanent strains depend not only on the magnitude of stresses induced by external load but also on load duration. The concept of viscosity can be explained by means of a *dashpot element* (Fig.2.8). As opposed to *elasticity* which can be represented by a *spring*
$$q_{ult} = c N_o$$

Figure 2.7. Bearing Capacity
Figure 2.8. Dashpot Element
element (Fig.2.9), the stresses in the dashpot element are not proportional to strains but to strain-rates. Mathematically, this can be expressed as:

\[
\sigma = \eta \frac{d\varepsilon}{dt} \tag{2.10}
\]

where

\[
\begin{align*}
\sigma & = \text{axial stress} \\
\eta & = \text{viscosity coefficient} \\
d\varepsilon & = \text{strain increment} \\
dt & = \text{time increment}
\end{align*}
\]

The viscosity coefficient can be a constant or a function of the applied stress and time. In the first case, the differential equation (2.10) is linear, in the second it is nonlinear. If \(\eta=\text{const.}\), and constant stress \(\sigma\) is applied, integration of eq.(2.10) gives the following relationship between the strain, stress, and time:

\[
\varepsilon = \sigma \frac{t}{\eta} \tag{2.11}
\]

Thus, strains increase as a function of time, and this is referred to as creep.
Figure 2.9. Spring Element
2.4 Elasto-Visco-Plastic Properties

In many materials the plastic and viscous properties are usually coupled with elastic properties. The term elasto-visco-plasticity is then used to emphasize the coupling. Since the separation of these effects in laboratory tests is difficult, often either the plastic properties or the viscous properties are neglected. This leads to simplified models: elastoplastic or viscoelastic.

In analyzing rutting, it has been long recognized that plastic properties play insignificant role, and efforts have concentrated on viscoelastic properties. It should be noted, however, that the mutual interaction between the elastic and viscous properties may not be as simple as in a Maxwell model, where the dashpot is connected in series with a spring (Fig.2.10). In fact, it may be much more complicated, and it cannot be represented by a set of spring and dashpot elements. The mathematical form of the resulting relationship can be in a differential or integral form. In all cases, the strains increase with time under constant stress. On the other hand, upon removal of stress, the strains can be fully recoverable given a sufficiently long time or not fully recoverable (permanent). In the former case, the viscoelastic material is referred to as solid whereas in the latter case as liquid. For example, in the Maxwell model strains will never fully recover and, therefore, it belongs to the class of liquid models.

To determine asphalt concrete viscosity related permanent strains, two approaches have been suggested in the literature: a) the uncoupled elastic-viscous approach, and b) the viscoelastic approach.
Figure 2.10. Maxwell Model
2.4.1 Uncoupled Elastic-Viscous Approach

The basic idea of this approach is similar to that used in geotechnical engineering for evaluating consolidation settlements of shallow foundations. Instead of solving a complex differential or integral relationship between the stresses, strains, and time, that describes the mechanical response of asphalt concrete, together with the static equilibrium equations and boundary stresses (load), the evaluation of stresses in the pavement structure is separated from the evaluation of permanent strains.

The vertical stresses in the pavement structure are determined from an elastic layered system. This means that the actual viscosity related properties of the asphalt concrete layer are disregarded, and the asphalt concrete is assumed to be purely elastic. Next, based on experiments where the asphalt concrete specimens are subjected to constant or cyclic loading, the vertical permanent strains are determined as a function of load magnitude and load duration. This relationship can be expressed as:

$$\epsilon_p = f(\sigma, t)$$  \hspace{1cm} (2.12)

where

$\epsilon_p =$ permanent strain

$\sigma =$ applied stress

$t =$ time

In cyclic tests, the time appearing in eq. (2.12) is replaced by the number of traffic load repetitions. In the constant load tests, the time in
eq.(2.12) equals the cumulative period of traffic load repetitions.

In the application of this approach to predicting rutting, the pavement structure is divided into a number of sublayers and the average vertical stresses are determined for each sublayer. The permanent vertical displacement (depth of rutting) of the asphalt concrete pavement structure is obtained by summing up the products of the average permanent strains given by eq.(2.10) and the corresponding sublayers thicknesses. This can be expressed as:

\[ h = \sum_{i=1}^{n} (\epsilon_{p,i} H_i) \quad (2.13) \]

where

- \( h \) = depth of rutting in asphalt concrete pavement
- \( \epsilon_{p,i} \) = permanent strain in sublayer 'i'
- \( H_i \) = thickness of sublayer 'i'
- \( n \) = number of sublayers

It should be noted here that the permanent strains in eq.(2.13) should correspond to the calculated stresses.

The idea of separating stress determination from material viscosity related properties determination has been suggested by many investigators like Heukelom and Klomp [10], Barksdale [11], and Romain [12]. Both the linear and nonlinear elastic layered systems can be considered. Even though the effects of moving traffic loads are not directly accounted for in this approach, they can be indirectly considered by testing the
materials using loading times that result from the speed of the traffic. Barksdale [11], Morris et al [13], McLean and Monismith [14], Brown and Snaith [15], Chomton and Valayer [16], Brown and Bell [17], Kirwan et al [18], Meyer and Haas [19], and Monismith et al [20] used repeated triaxial compression tests. Shell investigators [2, 21, 22, 23, 24], Finn et al [25], Monismith et al [26], Monismith and Tayebali [27], Kim et al [28], and Mahboub and Little [29] used static uniaxial compression tests.

A modified version of the uncoupled elastic-viscous approach has been suggested by Barksdale and Leonards [30], Elliot and Moavenzadeh [31], and Huschek [32]. For determining the stresses in the pavement structure, these authors modeled the asphalt concrete by a simple, linear viscoelastic model, e.g., Maxwell model. With this assumption, the stresses in the pavement structure can be obtained from elastic multilayer solution by appropriate change of material parameters. Next, as in the original uncoupled elastic-viscous approach, the depth of rutting in the asphalt concrete layer is calculated from eq.(2.13) using the actual material response determined from laboratory tests and calculated stresses.

The major advantage of the uncoupled elastic-viscous approaches is the simplicity of evaluating stresses in the asphalt pavement structure by means of well-known numerical codes, e.g., BISAR. Also, any experimentally determined relationship between the permanent strains, stresses, and time, can be incorporated in the analysis. This can also include the dependence of the permanent strains on temperature. On the other hand, the separation of stress and permanent displacements determination introduces error in predicting the actual magnitude of
permanent displacements. It should also be mentioned that this approach disregards the effect of lateral strains, for usually only the axial strains measured in experiments are taken into account in the depth of rutting prediction.

2.4.2 Viscoelastic Approach

In contrast to the uncoupled elastic-viscous approach, in the viscoelastic approach the stresses and permanent strains are determined simultaneously. This is done by solving in one step the differential or integral equation describing the actual viscoelastic properties of asphalt concrete, and the static equilibrium equations for given boundary stresses (load). This requires, however, use of sophisticated mathematical techniques if the actual, nonlinear material behavior is taken into account. Accordingly, the existing elastic multilayer solution numerical codes cannot be used, and the number of solutions available in literature is limited (Battistato et al [33], Thrower [34]).

The main advantage of the viscoelastic approach is that no error is introduced in permanent displacement calculation due to the separation of stress and strain calculation that typifies the uncoupled approach. Furthermore, the moving traffic loads can be directly incorporated in the analysis. This allows for accurate modeling of loading time of each material element in the asphalt concrete layer, and leads to realistic prediction of lateral flow beneath the traffic load. As shown by Battistato et al [33], and Thrower [34], the uncoupled approach greatly overestimates the permanent displacements as compared with the viscoelastic approach.
2.5 Test Methods for Permanent Deformation Prediction

Five laboratory test methods for predicting permanent deformations (strains) in asphalt concrete are summarized in this section. They have been used for calculating the depth of rutting in conjunction with the uncoupled elastic-viscous approach. They also can be used as good guides to develop new tests methods.

2.5.1 VESYS (M.I.T) [35]

In this method, the permanent strains of an asphalt concrete are determined from incremental static or dynamic creep tests. The cylindrical specimen of 4-in. diameter and 8-in. height is subjected to uniaxial compressive stress and temperature expected in the pavement layer. In the incremental static creep test, with the load and rest periods shown in Fig.2.11, the incremental permanent strain per load/rest pulse, $\Delta \epsilon_p$, is defined as the difference between the strain at the beginning and the end of the pulse (Fig.2.12). The variation of the accumulated permanent strain, $\epsilon_p$, with the duration of the load period, $\Delta t$, is taken as power law:

$$\epsilon_p(\Delta t) = I \Delta t^S$$ (2.14)

where

$I, S =$ intercept and slope of a log $\epsilon_p$ vs log $\Delta t$ plot.

In the dynamic creep test, the compressive stress varying sinusoidally over a 0.1-sec load period, and followed by a 0.9-sec rest
Figure 2.11. Loading Program of VESYS (after [35])
Figure 2.12. Incremental Permanent Strain
period (Fig.2.11), is applied. The incremental permanent strain, defined as in the static test, is measured at 1, 10, 100, 1,000, 10,000, and 100,000 pulses. The accumulated permanent strain as a function of the number of cycles, N, is expressed by an equation similar to eq.(2.14):

$$\epsilon_p(N) = I N^s$$  \hspace{1cm} (2.15)

In the VESYS method, the permanent strain, $\epsilon_p$, is expressed as a function of the total strain, $\epsilon_t$, that can be determined from the multilayer solution:

$$\epsilon_p(N) = \epsilon_t f(N)$$  \hspace{1cm} (2.16)

To determine function $f(N)$ the increment of the permanent strain $d\epsilon_p$ and of the total strain $d\epsilon_t$ for one cycle are considered:

$$d\epsilon_p(N) = d\epsilon_t f(N)$$  \hspace{1cm} (2.17)

Differentiation of eq.(2.15) gives:

$$d\epsilon_p(N) = I S N^{s-1}$$  \hspace{1cm} (2.18)

and eq. (2.17) gives:

$$f(N) = \frac{d\epsilon_p(N)}{d\epsilon_t}$$  \hspace{1cm} (2.19)
The incremental total strain, \( \Delta \varepsilon \), is taken as that corresponding to the 200th load pulse in dynamic test, or as the total strain measured at 0.03-sec for any load pulse in the interrupted static creep test. Then, the permanent rut depth, \( h \), can be calculated as:

\[
I S N^{5-1} \quad h = h_t \frac{I S N^{5-1}}{e}
\]  

(2.20)

where

\( h_t \) = total vertical displacement obtained from the multilayer solution

2.5.2 Shell Method [24]

The depth of rutting is predicted from the viscous component of the total asphalt concrete (mixture) stiffness, \( S_{m,v}(t) \). This is evaluated from the total asphalt concrete (mixture) stiffness, \( S_m(t) \), the total asphalt (binder) stiffness, \( S_a(t) \), and the viscous component of the total asphalt (binder) stiffness, \( S_{a,v}(t) \).

In the creep test, a cylindrical specimen (4-in. diameter and 2.5-in. height when the aggregate size is less than 0.65 in.; 6-in. diameter and 4-in. height when the aggregate is between 0.65 in. and 1.25 in.) is subjected to uniaxial compressive stress \( \sigma_0 = 14.5 \text{ psi} \) at 104 °F for 1 hr.

The total asphalt concrete stiffness, \( S_m(t) \), is obtained from a single static creep test, and defined as:
\[ S_m(t) = \frac{\sigma_0}{\varepsilon_c(t)} \quad (2.21) \]

where

\[ \varepsilon_c(t) = \text{creep strain} \]

Figure 2.13 shows \( S_m(t) \) as a function of time, \( t \), at given \( \varepsilon_c(t) \). In evaluating \( \varepsilon_c(t) \), a power law approximation of experimental results is used.

The total asphalt stiffness, \( S_a(t) \), can be obtained from the Van der Poel's nomograph (Fig.2.14 [36]) knowing the temperature, \( T \), the Ring and Ball Temperature, \( T_{800} \), the asphalt penetration index, \( PI \), and the time, \( t \). Figure 2.15 shows \( S_a(t) \) as a function of time at given \( T, T_{800} \), and \( PI \).

The viscous component of the total asphalt stiffness, \( S_{a,v}(t) \), can be expressed as a function of the viscosity of the asphalt, \( \eta \), at a given temperature, as:

\[ S_{a,v} = \frac{3 \eta}{t} \quad (2.22) \]

where

\[ t = N_{eq} \cdot t_0 \]

\( N_{eq} \) = the total number of the equivalent wheel passes

\( t_0 \) = the average time of one equivalent wheel pass

From Figs.2.13 and 2.15, a diagram is constructed that relates \( S_a(t) \)
Figure 2.13, Asphalt Concrete Stiffness ($S_m$) vs. Time ($t$) (after [24])
The stiffness modulus, defined as the ratio \( \sigma / \varepsilon \) = stress/strain, is a function of time of loading (frequency), temperature difference with R&B point, and PI. At low temperatures and/or high frequencies the stiffness modulus of all bitumins asymptotes to a limit of approx. \( 3 \times 10^9 \) N/m².

Example for a bitumen with \( \text{PI} = +2.0 \) and \( T_{\text{R&B}} = 75^\circ \text{C} \). To obtain the stiffness modulus at \( T = -11^\circ \text{C} \) and a frequency of 10 Hz: connect 10 Hz on time scale with 75 -- (-11) = 86 on temporary scale. Read \( S = 5 \times 10^8 \) N/m² on network at \( \text{PI} = +2.0 \).

Example for a bitumen with \( \text{PI} = -1.5 \) and \( T_{\text{R&B}} = 47^\circ \text{C} \). To obtain the temperature for a viscosity of 5 poises connect 5 P at \( \text{PI} = -1.5 \) in the network with viscosity point. Read \( T_{\text{Dif}} = 70^\circ; T = 70 + 47 = 117^\circ \text{C} \).

Units:
1 N/m² = 10 dyn/cm² = 1.02 x 10⁻⁵ kgf/cm² = 1.45 x 10⁻⁴ lb/sq.in.
1 N/s/m² = 10 P

Figure 2.14. Van der Poel's Nomograph (after [36])
Figure 2.15. Asphalt Stiffness ($S_a$) vs. Time ($t$) (after [24])
to $S_a(t)$ at equal times, $t$ (Fig.2.16). In the Shell method, it is further assumed that the relationship between the total asphalt concrete stiffness, $S_a(t)$, and the total asphalt stiffness, $S_a(t)$ is the same as the relationship between the viscous component of the total asphalt concrete stiffness, $S_{m,v}(t)$, and the viscous component of the total asphalt stiffness, $S_{a,v}(t)$. Therefore, the viscous component of the total asphalt stiffness, $S_{m,v}(t)$ can be determined from Fig.2.16. The depth of rutting, $h$, is calculated from:

$$h = H C_m Z \frac{\sigma_a}{S_{m,v}} \tag{2.23}$$

where

- $H$ - thickness of the asphalt concrete layer
- $C_m$ - correction factor depending on the type of mix and must be determined empirically
- $Z$ - vertical stress distribution factor obtained from layered elastic solution
- $\sigma_a$ - average contact stress

It should be noted that many investigators have studied and modified the Shell method. For example, Oregon State University [37] used 3 hr. test loading time and computer code ELSYM5 to calculate $Z$. Monismith and Tayebali [27] modified the specimen size and test conditions. They recommended that 4-in. diameter and 8-in. height specimens, three levels of test temperature (77, 100 and 140 °F), and higher stress level (15 to
Figure 2.16. Asphalt Concrete Stiffness ($S_m$) vs. Asphalt Stiffness ($S_a$) (after [24])
30 psi) be used.

2.5.3 Federal Institute of Technology, Switzerland, Method [32]

In this method it is proposed to determine the permanent strains in asphalt concrete pavement from static creep tests (Fig.2.17) or interrupted static creep tests where a period of rectangular loading pulses is inserted (Fig.2.18). In the static creep test the load is applied for 120-min followed by a 30-min recovery period. In the interrupted creep test, 10-sec load pluses separated by a 30-sec rest periods are applied at 10, 60, and 120 min of the static creep test. In both tests, a cylindrical specimen of 4-in. diameter and 4.7-in. height is subjected to uniaxial compressive stress $\sigma_0=14.5$ psi at 68, 104, 122 °F.

In the static creep test, the accumulated permanent strain, $\epsilon_p$, is defined as the difference between the creep strain, $\epsilon_c(t)$, and the recoverable strain, $\epsilon_r(t)$, that is measured from $\epsilon_c(t)$ at loading time, $t$, when the stress is removed (Fig.2.17). Its dependence on the loading time is taken as:

$$\epsilon_p(t) = c \sigma_0 t^A$$

(2.24)

where

$c, A =$ material constants

A similar equation is taken for the accumulated permanent strains determined from the interrupted static creep test with the time $t$ appearing in eq.(2.24) replaced by the sum of the incremental loading.
Figure 2.17. Static Creep Test
Figure 2.18. Interrupted Static Creep Test
times, $\Delta t$. The form of eq. (2.24) indicates that permanent strains are assumed to be a linear function of the applied stress at a given temperature.

In applying this method to rut depth prediction, the vertical stresses in a pavement system are determined not from an elastic multilayer solution but from a viscoelastic multilayer solution. This solution is obtained assuming that the pavement system may be modeled by a linear Maxwell model, i.e., a series connection of spring and dashpot elements. This leads to the following equation relating stresses and strains:

$$\frac{d\varepsilon}{dt} = \frac{1}{E} \frac{d\sigma}{dt} + \frac{\sigma}{\eta}$$  (2.25)

where

$E =$ Young's modulus

$\eta =$ viscosity coefficient

The viscosity coefficient can be determined from the equation describing the response of the dashpot element:

$$\sigma = \eta \frac{d\varepsilon}{dt}$$  (2.10)

Using eqs. (2.10), and (2.24), the viscosity coefficient can be expressed as:
\[ \eta = \frac{t^{1-A}}{cA} \]  

(2.26)

Equation (2.26) indicates that viscosity coefficient is not constant. However, the Maxwell model assumed requires the viscosity coefficient to be constant. Therefore, a constant representative viscosity coefficient is used in evaluating stresses. The accumulated permanent strain for constant stress \( \sigma_a \) in the asphalt concrete layer can be then expressed as:

\[ \varepsilon_{pa}(t) = \frac{\sigma_a}{\eta} t \]  

(2.27)

Finally, the permanent reduction in thickness, \( h \), of a given asphalt concrete layer, \( H \), is calculated from the following equation:

\[ h = H \varepsilon_{pa}(t) \]  

(2.28)

2.5.4 Ohio State University Method [38]

The Ohio State University researchers suggested determining permanent strains in asphalt concrete from dynamic tests carried out at a temperature expected in the pavement layer. A cylindrical specimen of 4-in. diameter and 8-in. height is subjected to 2000 pulses of uniaxial compressive stress \( \sigma_o \) varying sinusoidally. The duration of the loading periods is 0.125-sec separated by rest periods of 0.375-sec, respectively.

The incremental permanent strain per pulse, \( d\varepsilon_p \), is defined as in VESYS method (Fig.2.12). It is postulated that a power law describes the
variation of the incremental permanent strain with the number of pulses \( N \):

\[
\frac{\epsilon_p(N)}{N} = A N^{-m}
\]  

(2.29)

which is equivalent to the following power relationship between the accumulated permanent strain and the number of pulses \( N \):

\[
\epsilon_p(N) = A N^{1-m}
\]  

(2.30)

In view of the power law, eq.(2.30), the coefficient \( m \) represents the slope of a linear \( \log \epsilon_p/N \) versus \( \log N \) plot, and \( A \) is the intercept (Fig.2.19). It is further assumed that \( m \) is a material constant, whereas the coefficient \( A \) is a function of the applied stress and temperature. For constant temperature, this function is taken as:

\[
A = J \left\{ \frac{M_r}{\sigma_0} \right\}^{-S}
\]  

(2.31)

where

\( M_r = \) resilient modulus of the asphalt concrete

\( J, S = \) material constants determined from dynamic tests.

Since eq.(2.31) has a power law form, the constant \( S \) is the slope and \( J \) is the intercept of a \( \log A \) versus \( \log M_r/\sigma_0 \) plot. To determine these constants, at least two dynamic tests at stress levels \( \sigma_0 \) close to those
Figure 2.19. Log $\epsilon_p/N$ vs. Log $N$ (after [38])
expected in the pavement must be performed, and a log A versus log $M_r/\sigma_o$ plot constructed (Fig.2.20). The resilient modulus $M_r$ is determined at the end of 2000 loading pulses.

The accumulated permanent strain in the asphalt concrete layer due to stress $\sigma_a$ can be thus expressed as:

$$\epsilon_{pa}(N) = J \left\{ \frac{M_r}{\sigma_a} \right\}^{-s} N^{1-m} \quad (2.32)$$

where

$N = \text{equivalent number of standard wheel passes.}$

The permanent thickness reduction, $h$, of a layer of initial thickness $H$ is calculated from eq.(2.28).

2.5.5 Texas A&M Method [29]

The permanent strains in asphalt concrete are determined from a single static creep test at temperature 70 °F. In this test, a cylindrical specimen of 4-in. diameter and 8-in. height, is subjected to uniaxial compressive stress $\sigma_o=14.5\text{ psi}$ for 1 hour, followed by a 1-hour recovery period. The axial strains are calculated from axial displacements measured over the whole 2-hour period.

The permanent strain at a given loading time $t$, $\epsilon_p(t)$, is defined as in the Federal Institute of Technology, Switzerland, method (Fig.2.17). In evaluating the creep strain, $\epsilon_c(t)$, and the recoverable strain, $\epsilon_r(t)$, a power law approximation of experimental results is used.
Figure 2.20. Log A vs. Log $M_z/\sigma_o$ (after [38])
The permanent strain at stress level \( \sigma_a \) different than \( \sigma_o \), \( \epsilon_{pa}(t) \), is determined from the following equation:

\[
\epsilon_{pa}(t) = \left( \frac{\sigma_a}{\sigma_o} \right)^{1.61} \epsilon_p(t) \tag{2.33}
\]

The exponent 1.61 in eq.(2.33), is the stress nonlinearity coefficient. The magnitude of this coefficient was taken from literature [29].

The stress \( \sigma_a \) at a given point in the asphalt concrete pavement is related to the tire contact pressure, \( \sigma_t \), by an equation that is identical to that of other methods, i.e.:

\[
\sigma_a = Z \sigma_t \tag{2.34}
\]

where

\( Z \) = the vertical stress distribution factor obtained from a layered elastic solution

The permanent reduction in thickness of a layer, \( h \), is then calculated from eq.(2.28).

Equation (2.33) is valid if the temperature of the asphalt concrete layer equals that of the creep test. Otherwise, the permanent strain, \( \epsilon_p(t) \) appearing in eq.(2.33) is replaced by a modified permanent strain, \( \epsilon_{pm}(t) \). To determine \( \epsilon_{pm}(t) \), first the time dependent asphalt concrete stiffness \( S_m(t) \) is calculated as:
\[ S_m(t) = \frac{\sigma_0}{\epsilon_p(t)} \]  \hspace{1cm} (2.35)

Next, knowing the Ring and Ball temperature and the temperature dependent shift factor (Fig.2.21), the modified asphalt concrete stiffness, \( S_m(t) \), is determined with Fig.2.22. The modified permanent strain is then calculated as:

\[ \epsilon_{pm}(t) = \frac{\sigma_0}{S_m(t)} \]  \hspace{1cm} (2.36)

2.6 Formulae for Permanent Strains and Depth of Rutting

In this section several formulae for calculating the magnitude of permanent strains or the depth of rutting that have been suggested in the literature are presented. These formulae have been obtained from laboratory tests on specific types of asphalt concrete. Their application to other types of asphalt concrete may not be correct. However, they may serve as guidelines for developing new formulae.

Brown and Snaith [15], and Kirwan et al [18], carried out dynamic triaxial compression tests on cylindrical specimens of 6-in. diameter and 9-in. height, and creep tests on specimens of 4-in. diameter and 6-in. height. These specimens were made from a dense bitumen macadam with 4\% of asphalt content by weight. The variables of this investigation were the vertical stress, horizontal stress, temperature, frequency, and loading and rest time. Brown and Snaith arrived at following equation:
Figure 2.21. Shift Factor for Texas A&M Method (after [29])
Figure 2.22. Calculation of Asphalt Concrete Stiffness with Shift Factor (after [29])
\[ \varepsilon_p(t) = 100 \left[ 0.00015 (0.68 + 0.0008 T^2 \log N)^{1.9} \sigma_o \right]^{1.75} \]  \hspace{1cm} (2.37)

where

\varepsilon_p(t) = \text{permanent vertical strain}

T = \text{temperature (°C)}

N = \text{number of load applications}

\sigma_o = \text{vertical stress (kN/m²)}

McLean and Monismith [14] estimated the permanent deformation in asphalt concrete pavements from dynamic triaxial compression tests and creep tests on cylindrical specimens of 4-in diameter and 8-in height. The variables were the vertical stress, horizontal stress, temperature, frequency, and loading and rest time. The results of these tests yielded the following equation:

\[ \log \varepsilon_p(N) = C_0 + 0.85 \log N - 0.013 (\log N)^2 - 0.14 (\log N)^3 \]  \hspace{1cm} (2.38)

where

\varepsilon_p(N) = \text{permanent vertical strain}

N = \text{number of load applications}

C_0 = k + n \log \left[ (\sigma_v - \sigma_h) \varepsilon_e \right]

\varepsilon_e = \text{elastic vertical strain}

k, n = \text{experimentally determined coefficients}

\sigma_v = \text{vertical stress}

\sigma_h = \text{horizontal stress}
Allen and Deen [39] carried out dynamic triaxial compression tests on cylindrical specimens of 2-in. diameter and 3-in. height. The mixture contained crushed limestone aggregate and 5.2% asphalt by weight with viscosity grade AC-20. The specimens were subjected to three deviator stresses (20, 50, 80 psi). Three test temperatures (45, 77, 100 °F) were used at each deviator stress level. In addition, three loading times (0.5, 1.0, 2.0 sec) were used at each temperature. These authors also carried out uniaxial dynamic compression tests [40]. The latter study developed the following equation for the permanent strains:

\[
\log \varepsilon_p(N) = C_0 + 0.63974 \log N - 0.10392 (\log N)^2 \\
+ 0.00938 (\log N)^3
\]

(2.39)

where

- \( \varepsilon_p \) = permanent vertical strain
- \( N \) = number of compressive stress repetitions
- \( C_0 = [-0.000663 T^2 + 0.1521 T - 13.304] + [(1.46 - 0.00527 T) \log \sigma_o] \)
- \( T \) = test temperature (°F)
- \( \sigma_o \) = compressive stress (psi)

Investigators at the University of Waterloo [19] tested cylindrical specimens of 4-in. diameter and 8-in. height in dynamic triaxial compression tests. They used a 300-400 penetration asphalt and the variables of their tests were the vertical stress, horizontal stress, temperature, air void, and number of load applications. They found that the depth of rutting can be calculated from the following equation:
\[ h = - 1.0318 + 1.2067 \text{EAT} + 0.0803 N - 2.3684 \ln \text{EAT} \\
+ 0.1896 \ln (N \text{EAT}) + 1.1639 E_a \ln \text{EAT} - 0.0216 E_a N \\
- 0.4114 E_a N \ln \text{EAT} + 0.0456 E_a N \ln \text{EAT} \]  

(2.40)

where

- \( h \) = depth of rutting (in.)
- \( \text{EAT} \) = equivalent asphalt thickness/10 (in.); 1 in. hot mix asphalt concrete = 2 in. granular base = 3 in. subbase
- \( E_a \) = resilient modulus of asphalt concrete/10^8 (psi)
- \( E_s \) = resilient modulus of subgrade/10^4 (psi)
- \( N \) = number of equivalent 18-kip load applications/10^5

This equation is valid up to 500,000 repetitions of 18-kip load (ESAL). When \( N \) is greater than 500,000, the following equation should be used to calculate the depth of rutting:

\[ h' = h + (1 - e^{-bc}) \]  

(2.41)

where

- \( h' \) = depth of rutting when \( N > 500,000 \) (in.)
- \( h \) = depth of rutting when \( N \leq 500,000 \) (in.)
- \( b \) = 0.2/\text{EAT}
- \( c \) = \((N - 500,000)/500,000\)

Al-Juraiban and Jimenez [41] conducted static uniaxial compression tests on cylindrical specimens with variable size (18-in. diameter and 2
to 6-in. height). The specimens were prepared from a dune-sand and an asphalt of 60-70 penetration grade. Three stress levels (38.5, 61.6, 98.0 psi) were used. These authors found that the depth of rutting can be predicted from the following equation:

\[ \log h = -10.7019 + 0.4820 \log N + 5.7627 \log T + 0.9756 \log \sigma_o \\
+ 0.9664 \log E_s \]  

(2.42)

where

\( h \) = depth of rutting (10^{-3} \text{ in.})
\( N \) = number of load repetitions
\( T \) = test temperature (°F)
\( \sigma_o \) = vertical stress (psi)
\( E_s \) = resilient modulus of subgrade (psi)

Monismith et al [19] suggested to determine the vertical permanent strain in an asphalt concrete pavement from a generalized elasticity law:

\[ \epsilon_p = R \left[ \sigma_z - 0.5 (\sigma_x + \sigma_y) \right] \]  

(2.43)

where

\( \epsilon_p \) = permanent vertical strain
\( \sigma_z \) = stress in z-direction (vertical)
\( \sigma_y \) = stress in y-direction
\( \sigma_x \) = stress in x-direction
\( R \) = compliance to be experimentally determined
These authors conducted dynamic triaxial tests and found the following expression for $R$:

$$R = T \, e^{\left(-A/T\right)} \, N^\alpha \, (\sigma_1 - \sigma_2)^{n-1} \quad (2.44)$$

where

$T = \text{temperature (°F)}$

$\sigma_1 = \text{major principal stress}$

$\sigma_2 = \text{minor principal stress}$

$\alpha, n, A = \text{experimentally determined coefficients}$

Francken [42], and Verstraeten et al [43] conducted dynamic triaxial tests at different temperatures (59, 86, 113 °F), frequencies (1, 3, 10, 30, 40 Hz), and amplitudes of vertical stress. From these tests the following relationship was developed:

$$\epsilon_p(t) = 115 \, \frac{(\sigma_v - \sigma_h)}{E_a} \, t^{0.25} \quad (2.44)$$

where

$\epsilon_p(t) = \text{permanent vertical strain}$

$t = \text{time (10^3 sec)}$

$\sigma_v = \text{the amplitude of the vertical stress}$

$\sigma_h = \text{constant horizontal stress}$

$E_a = \text{stiffness modulus of asphalt concrete}$
CHAPTER 3
LABORATORY TESTS

The experimentation is described in this chapter. Section 1 of this chapter describes the objective and scope of the laboratory tests proposed. In Section 2 the test program is described, and the test and response variables and their levels are specified. Section 3 discusses the characteristics and properties of the materials used in the laboratory tests. These are typical materials for asphalt pavements in Minnesota. Section 4 explains the preparation of the test specimens. Mixture preparation and compaction are also described in this section.

3.1 Test Objective and Scope

The objective of the proposed laboratory tests was to investigate the development of permanent strains in asphalt concrete as a function of applied stress, time, and temperature. The scope of the laboratory tests was exploratory for better understanding the dependence of permanent strains in asphalt concrete on the variables specified above. The results of the laboratory tests were used to identify a suitable test method for characterizing asphalt concrete susceptibility to rutting.

To accomplish the objective described above, an experimental program was designed. This program did not follow any of the laboratory test methods described in Chapter 2. The program focused, instead, on investigating the contribution of the test variables separately, rather than selecting one variable as dominating the development of permanent strains.
3.2 Test Program and Experimentation

Due to the exploratory nature of the proposed laboratory tests, a simple experimental method was selected, namely, uniaxial compression under constant stress (uniaxial static creep/recovery test) on cylindrical specimens of 4-in. diameter and 8-in. height.

The variables selected for these tests were the temperature, axial stress, and loading/unloading time with three or four different levels of each variable. The matrix form of these variables is shown in Table 3.1. Figure 3.1a presents schematically the variation of the axial stress with time.

The response variables measured in the test were the axial displacement and the varying circumference of the specimen. These can be used to calculate the axial and lateral strains. It was decided to measure the axial displacements over a 3-in. length at three different locations 120° apart, and the circumference of the specimen at its mid-height, at 10 sec time intervals for both the loading and the unloading period (Fig. 3.1b). The experimental set-up, instrumentation, and the role of the instrument components are described in Appendix A.1. The details of the test procedure are explained in Appendix A.2.

3.3 Materials

The materials selected for this research are available for use in asphalt concrete pavements in Minnesota. This particular mixture was classified as Type 2331.B.
Table 3.1. Test Variables and Replicates

<table>
<thead>
<tr>
<th>Temp (°F)</th>
<th>70</th>
<th>77</th>
<th>86</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress (psi)</td>
<td>14.5</td>
<td>21.75</td>
<td>29.0</td>
</tr>
<tr>
<td>Creep/recovery time (min)</td>
<td>5/25</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>30/30</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>60/60</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>120/120</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>
Figure 3.1. Loading Program and Displacement Measurement
3.3.1 Aggregate

Three different aggregates, BA 003, BA 004, and BA 005, were combined. Each aggregate consisted of coarse and fine particles. BA 003 is 95% crushed gravel and its bulk specific gravity is 2.595. BA 004 is a screened gravel and its bulk specific gravity is 2.689. BA 005 is 100% crushed limestone and its bulk specific gravity is 2.661. These aggregates were combined in proper proportions to give the required gradation (Table 3.2 and Fig.3.2). The bulk specific gravity of the combined aggregate is 2.658. The aggregates were obtained from Commercial Asphalt Inc., Osseo, Minnesota. The characteristics and properties of these aggregates are described in detail in Appendix B.1.

3.3.2 Asphalt Cement

The asphalt cement selected for the mixtures is a typical 120/150 penetration grade asphalt with the specific gravity of 1.017. This asphalt cement, which meets specifications set by the Minnesota Department of Transportation, was obtained from Ashland Oil Co., St. Paul Park, Minnesota. The characteristics and properties of the asphalt cement are described in Appendix B.2.

3.4 Specimens

3.4.1 Mixture Preparation

One batch of mixture was prepared for each six specimens. Since the selected asphalt content by weight in the mixture was 5.5%, 22,000 grams of combined aggregate, and 1,280 grams of asphalt cement, were used for one batch. The combined aggregate, asphalt cement, and mixing equipment
<table>
<thead>
<tr>
<th>sieve number</th>
<th>sieve size (mm)</th>
<th>passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2&quot;</td>
<td>12.7</td>
<td>100</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>9.51</td>
<td>90</td>
</tr>
<tr>
<td>4</td>
<td>4.75</td>
<td>61</td>
</tr>
<tr>
<td>10</td>
<td>2.0</td>
<td>47</td>
</tr>
<tr>
<td>40</td>
<td>0.425</td>
<td>19</td>
</tr>
<tr>
<td>120</td>
<td>0.213</td>
<td>8</td>
</tr>
<tr>
<td>200</td>
<td>0.075</td>
<td>4.2</td>
</tr>
</tbody>
</table>
Figure 3.2. Combined Gradation
were first heated at 290 ± 10 °F (asphalt cement and mixing equipment for 4 hours; aggregates for 12 hours). The asphalt cement was then poured into the mixing bowl while the aggregates were being mixed with an automatic mixer for 2 minutes. The details of the mixture preparation are given in Appendix C.1.

3.4.2 Specimen Preparation

The required sixty and additional nine cylindrical specimens of 4 x 8 in. were prepared at 275 ± 10 °F by placing the mixture in a 4 x 10 in. split mold and performing compaction. Since the Marshall compaction method could not be used for 4 x 8 in. specimens, a somewhat equivalent compaction method using a vibratory hammer was developed. This compaction method is described in Appendix C.2 and the compaction procedure is in Appendix C.3. Upon the completion of the specimen compaction the specimen was removed from the mold, and its bulk specific gravity, air void content, and VMA were determined. Finally, the specimen was stored in a freezer at -12 ± 2 °F.

To check the uniformity of the compaction, two specimens (No.3 and No.4) were cut into three parts coinciding with the three compacted layers and their bulk specific gravity, air void content, and VMA were determined. The results are listed in Table C.1. It can be seen that the difference in the bulk specific gravity of any two specimens was smaller than 0.03 which, according to the Marshall method requirement, implies that the specimens are statistically uniform. Table C.2 in Appendix C.4 shows the characteristics and properties of the 69 specimens prepared. The summary of the characteristics and properties of the specimens is
given in Appendix C.4.
CHAPTER 4
TEST RESULTS AND DATA ANALYSIS

This chapter is organized in five sections. Section 4.1 presents the results obtained from the uniaxial static creep/recovery tests described in Chapter 3. The general concepts and methodology of the data analysis are presented in Section 4.2. Section 4.3 discusses the axial strain components as recorded in the tests. In particular, the elastic strains, plastic strains, and viscoelastic strains are discussed here. The experimental results are further used to analyze the linearity of the asphalt concrete with the help of the concept of the creep and recovery compliance, and the Boltzmann Superposition Principle discussed in Section 4.2. In Section 4.4, some observations regarding lateral strains are presented. Finally, the conclusions drawn from the experimental program are presented in Section 4.5.

4.1 Test Results

Fifty-nine tests were completed according to the test program described in Chapter 3. Figures 4.1 and 4.2 are examples of results obtained from tests at a uniaxial stress ($\sigma_0$) of 14.5 psi and 29.0 psi, and temperature (T) of 70 °F. These diagrams show the variation of the axial stress and axial strain. The axial stress was calculated as the ratio between the axial load and the area of the initial cross-section of the specimen:
Figure 4.1. Example of Test Results
Figure 4.2. Example of Test Results

Temp = 70 F
Stress = 29.0 psi
\[ \sigma_o = \frac{P}{A} \]  

(4.1)

where

\( \sigma_o \) = axial stress

\( P \) = axial force

\( A \) = area of the initial cross-section of the specimen

It can be seen that the assumed stress history (constant stress for the first 0.5 hr and zero stress for the next 0.5 hr) was well-reproduced. The average axial strains were calculated from displacements measured by the three vertical LVDTs. The following formula was used:

\[ \varepsilon_a = \frac{\Delta L_1 + \Delta L_2 + \Delta L_3}{L_1 + L_2 + L_3} \]  

(4.2)

where

\( \varepsilon_a \) = axial strain

\( L_1, L_2, L_3 \) = initial base length for LVDT 1, 2, and 3

\( \Delta L_1, \Delta L_2, \Delta L_3 \) = change in base length \( L_1, L_2, \) and \( L_3 \)

The diagrams representing the results for all the fifty-nine tests are presented in Appendix D.

The results from individual tests were used to construct the average axial strain versus time curves for given stress level and temperature. These curves are presented in Figs. 4.3 to 4.7. The scatter in the axial
Figure 4.3. Average Axial Strain vs. Time at $\sigma_0 = 14.5$ psi, $T = 70 ^\circ F$
Figure 4.4. Average Axial Strain vs. Time at $\sigma_0=21.75$ psi, $T=70 \degree F$
Figure 4.6. Average Axial Strain vs. Time at $\sigma_0=14.5$ psi, $T=77$ °F
Temp = 86°F
Stress = 14.5 psi

Figure 4.7: Average Axial Strain vs. Time at $\sigma_0=14.5$ psi, $T=86\,^\circ F$
strains, due primarily to the variation in the air void contents of the specimens, ranged from ±5% at \( t=0 \) to ±15% at \( t=120 \) min. It should also be mentioned that at a temperature of 86 °F some specimens failed at about 20 min, which is illustrated in Fig.4.8. In this case, the average curves were constructed using only the initial part of the strain versus time curve.

4.2 Data Analysis Methodology

There are two main issues related to the experimental evaluation of the permanent deformation in asphalt concrete. One issue is the overall mechanical characteristics of the asphalt concrete. As discussed in Chapter 2, the permanent deformation can be due to either plasticity or viscosity of the asphalt concrete, or due to a combination of both. Even though the existence of both properties in asphalt concrete has been long recognized, usually one of them is ignored in predicting the permanent deformation. The other issue is the mathematical form of the assumed mechanical model. In particular, the answer to the question of whether or not the mechanical model is linear is of great importance, for a nonlinear model leads to much greater difficulties in applications than a linear one. Some investigators have assumed that asphalt concrete behavior is linear [32] whereas others have shown that its behavior is essentially nonlinear [44]. In the following, these two issues are discussed in detail with regard to axial strains measured in the uniaxial creep/recovery tests described in Chapter 3.

Figure 4.9 is a schematic creep and recovery curve drawn from Figs.4.1 to 4.7. It is observed that when the load is applied (\( t=0 \)), an
Figure 4.8. Axial Strain vs. Time at $\sigma_0 = 21.75$ psi, $T = 86 \, ^\circ F$
Figure 4.9. Schematic Representation of Creep/Recovery Test
instantaneous strain that will be termed *instantaneous creep strain*, and denoted as \( \varepsilon_c(0) \), appears. With elapsing time \((0 \leq t \leq t_1)\), the strain increases and the total strain will be termed *creep strain* and denoted by \( \varepsilon_c(t) \). Once the load is removed \((t = t_1)\), there is an *instantaneous recoverable strain* \( \varepsilon_r(0) \) and, during the unloading period \((t_1 \leq t \leq t_2)\), part of the creep strain is further recovered. The strain measured in this period is termed *recovery strain* and denoted by \( \varepsilon'_r(s) \) where \( s = t - t_1 \). Finally, the difference between the extension of the creep curve and the recovery curve will be termed *recoverable strain* and denoted by \( \varepsilon_r(s) \). It should be mentioned here that the strain notation shown in Fig. 4.9 differs from that often used in the analysis of creep tests on asphalt concrete \([29, 44]\); the recoverable strain \( \varepsilon_r(s) \) is measured from the extension of the creep curve \( \varepsilon_c(t) \) rather than from the strain at \( t = t_1 \), \( \varepsilon_c(t_1) \). Also, there is a difference between the *recoverable strain* and *recovery strain* denoted by \( \varepsilon_r(s) \) and \( \varepsilon'_r(s) \), respectively. The latter is measured during the recovery period, and the former is calculated. The notation introduced is necessary if one attempts to analyze the results within the framework of the linear or nonlinear viscoelasticity theory. The above schematic creep/recovery curve (Fig. 4.9) is typical for many materials and further analysis is required to identify the meaning of these strains.

By definition, in a material without *plastic* properties, the strain upon instantaneous loading/unloading must be fully recoverable, \( \varepsilon_c(0) = \varepsilon_r(0) \), and this behavior is called *elastic* (Fig. 4.10). On the other hand if, for this particular loading/unloading cycle, the strain is not fully recoverable, \( \varepsilon_r(0) < \varepsilon_c(0) \), the irrecoverable strain, \( \varepsilon_c(0) - \varepsilon_r(0) \), is plastic (Fig. 4.11). Another way of investigating as whether or not the
Figure 4.10. Elastic Strain at Instantaneous Loading/Unloading Cycle
Figure 4.11. Plastic Strain at Instantaneous Loading/Unloading Cycle
material possesses plastic properties, is to use the instantaneous recoverable strains $\epsilon_r(0)$ at various unloading times $t_1$ and extrapolate them to $t_1=0$. If this extrapolation gives $\epsilon_c(0)-\epsilon_r(0)=0$ the material has no plastic properties (Fig.4.12b), otherwise plastic properties are present (Fig.4.12c).

Since, by definition, the plastic strain is independent of time, the time dependent part of the creep strain $\epsilon_c(t)$ and the time dependent part of the recoverable strain $\epsilon_r(s)$ can only be viscous or viscoelastic. Furthermore, if the instantaneous recoverable strain $\epsilon_r(0)$ at time $t_1 > 0$ is smaller than the instantaneous creep strain $\epsilon_c(0)$, and the material has no plastic properties, the instantaneous recoverable strain $\epsilon_r(0)$ cannot be identified as purely elastic because, by definition, elastic strains are time independent; the instantaneous recoverable strain $\epsilon_r(0)$ can then be regarded as a result of viscoelastic properties.

The term linearity is used in mechanics to describe a particular response of materials to external loads. The response is called linear if the relationship between the stresses and strains is directly proportional regardless as whether the stresses increase or decrease (loading and unloading). This implies that a material possessing plastic properties cannot be linear for plastic strains do not recover upon unloading. On the other hand, the term linearity equally applies to materials whose deformations are elastic or viscoelastic for only direct proportionality between stress and strains.

In the case of materials exhibiting time dependent strains, such as asphalt concrete, there are various ways of investigating their linearity from the results of a creep/recovery test. One way is to construct graphs
Figure 4.12. Extrapolation of Unloading Time to 0
showing the relationship between the stresses and creep strains $\epsilon_c(t)$ at a given time of loading. Hypothetical curves so obtained are shown in Fig.4.13. Figure 4.13a shows an example when the relationship between stresses and creep strains $\epsilon_c(t)$ taken at various times of loading is directly proportional (linear material), whereas Fig.4.13b shows a relationship where stresses are nonlinear functions of creep strains $\epsilon_c(t)$ (nonlinear material). A similar graph can be constructed for the recoverable strains $\epsilon_r(s)$.

Another way is to introduce the creep and recovery compliances; ratios of creep and recovery strains to the constant applied stress. Mathematically, the creep compliance is defined as (Fig.4.14a):

$$J_c(\sigma_o, t) = \frac{\epsilon_c(t)}{\sigma_o} \quad (4.3)$$

where

$$J_c(\sigma_o, t) \quad \text{creep compliance}$$

$$\epsilon_c(t) \quad \text{creep strain}$$

$$\sigma_o \quad \text{applied stress}$$

Similarly, the recovery compliance is defined as (Fig.4.14b):

$$J_r(\sigma_o, s) = \frac{\epsilon_r(s)}{\sigma_o} \quad (4.4)$$

where
Figure 4.13. Stress vs. Creep Strain at Various Loading Times
Figure 4.14. Creep and Recovery Compliances
\( J_r(\sigma_0, s) \) = recovery compliance
\( \varepsilon_r(s) \) = recoverable strain
\( \sigma_0 \) = applied stress

For a material that is linear the **Boltzmann Superposition Principle** holds true. This principle states that the strains due to constant stress \( n\sigma_0 \) is the \( n \)-sum of strains due to stress \( \sigma_0 \). For example, in Fig.4.15, the strains \( \varepsilon_2(t) \) due to stress \( 2\sigma_0 \) are equal \( \varepsilon_1(t) + \varepsilon_1(t) \) where \( \varepsilon_1(t) \) is the strain due to stress \( \sigma_0 \). This principle also applies to a creep/recovery test, where the difference between the creep strain \( \varepsilon_c(t) \) and the recovery strain \( \varepsilon_r'(s) \), i.e., the recoverable strain \( \varepsilon_r(s) \), taken at time \( t > t_1 \) must be equal to the creep strain \( \varepsilon_c(t) \) for time \( t = \) s (Fig.4.16). This implies that for a linear material both the creep compliance and the recovery compliance must be equal and independent of the magnitude of the applied stress, i.e.:

\[
\begin{align*}
J_c(\sigma_0, t) &= J_c(t) \\
J_r(\sigma_0, s) &= J_r(s) \\
J_c(t) &= J_r(s) & \text{for } t = s
\end{align*}
\] (4.6)

The description above of the various strains related to elastic, plastic, and viscous properties of a material in a creep/recovery test and the ways of investigating material linearity are used below for analyzing the test results of asphalt concrete.
Figure 4.15. Schematic Representation of Boltzmann Superposition Principle in Creep/Recovery Test

\[ \varepsilon_2(t) = \varepsilon_1(t) + \varepsilon_1(t) \]
Figure 4.16. Linearity in Creep/Recovery Test
4.3 Analysis of Strains in Creep/Recovery Tests

4.3.1 Plastic Strains

To investigate the presence of plastic strains in the asphalt concrete the extrapolation procedure to time \( t_1 = 0 \) of the instantaneous recoverable strains \( \varepsilon_r(0) \) was used. The extrapolation procedure was done using the average axial creep/recovery curves. The instantaneous recoverable strains \( \varepsilon_r(0) \) at \( t_1 = 5, 30, 60, \) and \( 120 \) min, were used. Figure 4.17 shows the dependence of the instantaneous recoverable strains \( \varepsilon_r(0) \) on the unloading time \( t_1 \) at the three applied stress levels and temperature \( T = 70 \) °F. It is evident from this figure that the magnitude of the instantaneous recoverable strains \( \varepsilon_r(0) \) is approximately independent of \( t_1 \) for a given axial stress. In other words, the instantaneous recoverable strain \( \varepsilon_r(0) \) is almost constant and can be identified as a purely elastic strain. Figure 4.18 shows the relationship between the instantaneous creep and recoverable strains and the magnitude of the applied stress. It is seen that these relationships are linear. Also, the instantaneous recoverable strain \( \varepsilon_r(0) \) is significantly smaller than the instantaneous creep strain \( \varepsilon_c(0) \) which implies that plastic strains developed upon instantaneous loading. The magnitude of the plastic strains \( \varepsilon_p \) was calculated from:

\[
\varepsilon_p = \varepsilon_c(0) - \varepsilon_r(0)
\]  

(4.7)

and the results plotted in Fig.4.19 as a relationship between the stress and the plastic strain indicate that plastic strains are proportional to stress. Similar observation was reported by Perl et al. [44]. This means
Temp = 70 F

29.0 psi

21.75 psi

14.5 psi

Time t, (min)

0.0015

0.001

0.0005

Instantaneous Recoverable Strain

Figure 4.17. Instantaneous Recoverable Strain vs. Unloading Time at Three Stress Levels
Figure 4.18. Instantaneous Strains vs. Stress

Temp = 70 F

creep

plastic

recoverable

elastic

Instantaneous Strain

0.005 0.0025

29 14.5 0

Stress (psi)

43.5 0
that plastic strains occur at arbitrarily small stress levels, and no stress threshold (yield condition) can be defined below which no plastic strains develop.

It should be noted here, that qualitatively similar results were obtained for other temperatures, i.e., the instantaneous recoverable strains $\epsilon_r(0)$ were independent of the unloading time and smaller than the instantaneous creep strains $\epsilon_c(0)$ (Figs. 4.20 and 4.21).

The slope of the straight line shown in Fig. 4.19 can be identified with the plastic modulus, and the relationship between the plastic strain and stress can be mathematically written as:

$$M_p = \frac{\sigma_0}{\epsilon_p} \tag{4.8}$$

where

- $M_p$ = plastic modulus
- $\epsilon_p$ = plastic strain
- $\sigma_0$ = applied stress

The plastic modulus $M_p=11,288.9$ psi at a temperature $T=70 \degree F$.

To investigate the influence of the temperature on the plastic strains determined above, Fig. 4.22 was constructed. It is seen that plastic strains at a given stress are not proportional to temperature. With increasing temperature, the plastic strains increase more than proportionally.
Figure 4.20. Instantaneous Recoverable Strain vs. Unloading Time at Three Temperatures
Figure 4.21. Instantaneous Strains vs. Temperature
Figure 4.22. Plastic Strain vs. Temperature

Stress = 14.5 psi

Plastic Strain
4.3.2 Elastic Strains

As mentioned above, the elastic strains at temperature T=70 °F are a linear function of stress (Fig.4.23). The slope of the straight line in Fig.4.23 can be identified with the elastic modulus and the dependence of the elastic strains on stress can be expressed as:

\[
M_e = \frac{\sigma_0}{\epsilon_e}
\]  

(4.9)

where

\[M_e = \text{elastic modulus}\]
\[\epsilon_e = \text{elastic strain}\]
\[\sigma_0 = \text{applied stress}\]

The elastic modulus \(M_e = 36,657.3\) psi at temperature \(T=70\) °F. It should be mentioned here that the elastic modulus \(M_e\) in this analysis is different from the resilient modulus \(M_r\) which is commonly used in pavement engineering since the test methods are totally different from each other.

Figure 4.24 depicts the dependence of the elastic strains at stress \(\sigma_0 = 14.5\) psi on temperature. Like the plastic strains, the elastic strains are temperature dependent.

4.3.3 Viscoelastic Strains

In the creep/recovery test, one deals with the viscoelastic strains \(\epsilon_{ve}(t)\) during the creep period and during the recovery period. The viscoelastic creep strains \(\epsilon_{vec}(t)\) during the creep period are represented
Figure 4.24. Elastic Strain vs. Temperature
by the difference between the total creep strains and the sum of the elastic and plastic strains:

$$\varepsilon_{\text{ve}}(t) = \varepsilon_c(t) - \varepsilon_e - \varepsilon_p$$  \hspace{1cm} (4.10)

The viscoelastic recoverable strains $\varepsilon_{\text{ver}}(s)$ during the recovery period, however, are represented by the difference between the total recoverable strains $\varepsilon_z(s)$ and only the elastic strain $\varepsilon_e$:

$$\varepsilon_{\text{ver}}(s) = \varepsilon_z(s) - \varepsilon_e$$  \hspace{1cm} (4.11)

Figure 4.25 shows the dependence of the viscoelastic creep and recoverable strains on time at $\sigma_0 = 14.5$ psi and $T = 70 \degree F$. It is seen that the viscoelastic creep strains $\varepsilon_{\text{ve}}(t)$ are larger than the viscoelastic recoverable strains $\varepsilon_{\text{ver}}(s)$; the latter ones also depend on the unloading time $t_1$. A similar result was obtained for other stress levels and temperatures. However, the difference between the viscoelastic creep and recoverable strains is not substantial and, for practical applications, could be disregarded. With this simplifying assumption, the dependence of the viscoelastic strains $\varepsilon_{\text{ve}}(t)$ on time is shown in Fig.4.26. This averaged curve can then be used to arrive at a mathematical expression relating the viscoelastic strains $\varepsilon_{\text{ve}}(t)$ to time. This was not attempted, however, and an alternative approach was selected in which the creep and recovery compliances are first constructed. The advantage of this approach is that the creep and recovery compliances are useful in analyzing the linearity of the viscoelastic strains. This is discussed in
Figure 4.25. Viscoelastic Creep/Recoverable Strains vs. Time

Temp = 70 F
Stress = 14.5 psi

Time, t (min)
Temp = 70 F
Stress = 14.5 psi

Figure 4.26. Viscoelastic Strain vs. Time
the following section.

4.3.4 Linearity of Viscoelastic Strains

To investigate the linearity of the viscoelastic strains, the creep and recovery compliances were constructed. These compliances were constructed only for the time dependent viscoelastic strains $\epsilon_{ve}(t)$, and were defined as:

$$J_{vec}(t) = \frac{\epsilon_{vec}(t)}{\sigma_0} \quad (4.12)$$

$$J_{ver}(s) = \frac{\epsilon_{ver}(s)}{\sigma_0} \quad (4.13)$$

where

$J_{vec}(t) = \text{viscoelastic creep compliance}$

$J_{ver}(s) = \text{viscoelastic recovery compliance}$

Figure 4.27 shows an example of the creep and recovery compliances for $\sigma_0=14.5$ psi and $T=70$ °F. Since the creep and recovery compliances differ, the material response is not linearly viscoelastic. To assess the influence of the stress level on the creep and recovery compliances, Fig.4.28 was constructed where the values of the creep and recovery compliance at various time instances are plotted as a function of stress. It is seen that even though for stresses above about 14.5 psi both compliances are dependent upon stress they differ from each other. This again means that the response is not linearly viscoelastic. It should be
Figure 4.27. Viscoelastic Creep/Recovery Compliances vs. Time

Temp = 70 F
Stress = 14.5 psi

0.02
0.01
0.00

Jvec & Jver

Time, t (min)

30
30
0
0
Figure 4.28. Viscoelastic Creep/Recovery Compliances vs. Stress
kept in mind, however, that these conclusions are based on a limited number of tests with specimens having slightly different void contents. Furthermore, as the difference between the creep and recovery compliances is small, it may be warranted to disregard this difference and assume that the viscoelastic response is linear.

To arrive at a mathematical expression for the average linear viscoelastic compliances $J_{ve}(t)$, a log $J_{ve}(t)$ versus log $t$ plot was constructed at $T=70^\circ F$ (Fig. 4.29). For $t$ greater than about 5 min, the following power law can be accepted:

$$J_{ve}(t) = at^b$$  \hspace{1cm} (4.14)

where

$$J_{ve}(t) = J_{vec}(t) = J_{ver}(s)$$

$$a = 2.94 \times 10^{-5}$$

$$b = 0.33$$

$t = \text{time in minute}$

This implies that the viscoelastic strains can be expressed as:

$$\epsilon_{ve}(t) = \sigma_0 J_{ve}(t)$$  \hspace{1cm} (4.15)

Finally, combining the elastic, plastic, and viscoelastic strains, the total creep strains can be expressed as:
Figure 4.29. Log (Viscoelastic Compliance) vs. Log (Time)
\[ \varepsilon_c(t) = \varepsilon_p + \varepsilon_e + \varepsilon_{ve}(t) \]
\[ = \sigma_o [1/M_p + 1/M_e + J_{ve}(t)] \]
\[ = \sigma_o (8.86 \times 10^{-5} + 2.73 \times 10^{-5} + 2.94 \times 10^{-5} t^{0.33}) \]  \hspace{1cm} (4.16)

and, since total recoverable strains are given by:

\[ \varepsilon_I(s) = \varepsilon_e + \varepsilon_{ve}(s) \]
\[ = \sigma_o [1/M_e + J_{ve}(s)] \]
\[ = \sigma_o (2.73 \times 10^{-5} + 2.94 \times 10^{-5} s^{0.33}) \]  \hspace{1cm} (4.17)

the total recovery strains as:

\[ \varepsilon'_I(s) = \varepsilon_c(t) - \varepsilon_I(s) \]
\[ = \sigma_o [1/M_p + J_{ve}(t) - J_{ve}(s)] \]
\[ = \sigma_o (8.86 \times 10^{-5} + 2.94 \times 10^{-5} (t^{0.33} - s^{0.33})) \]  \hspace{1cm} (4.18)

Due to a limited number of tests conducted at temperature other than 70 °F, a detailed analysis of the temperature influence on the viscoelastic and total strains was not attempted.

4.4 Lateral Strains

The lateral strains at mid-height of the specimen can be calculated from the measured by the extensometer change in the diameter of the specimen:
\[ \epsilon_1 = \frac{\Delta r}{r} \quad (4.19) \]

where

\[ \epsilon_1 = \text{lateral strain} \ (< 0) \]
\[ \Delta r = \text{initial radius} \]
\[ r = \text{change in radius} \]

With the additional assumption that the specimen deforms uniformly, the \textit{volumetric strains} can be calculated as the sum of the axial strain, lateral strain, and circumferential (hoop) strain:

\[ \epsilon_{vol} = \epsilon_a + \epsilon_1 + \epsilon_h \quad (4.20) \]

where

\[ \epsilon_{vol} = \text{volumetric strain} \]
\[ \epsilon_a = \text{axial strain} \ (> 0) \]
\[ \epsilon_1 = \text{lateral strain} \ (< 0) \]
\[ \epsilon_h = \text{hoop strain} \ (< 0) \]

Since the hoop strain in uniform cylindrical deformation equals the lateral strain, eq. (4.20) can be written as:

\[ \epsilon_{vol} = \epsilon_a + 2\epsilon_1 \quad (4.21) \]

The positive sign of \( \epsilon_{vol} \) means compaction, and the negative dilation.
When inspecting the recorded changes in the diameter of the specimen, it was found that the accuracy of the extensometer was insufficient to obtain reliable results, particularly during the recovery period. Therefore, only qualitative results could be obtained and they are shown in the following figures. Figure 4.30 shows an example of the variation of the volumetric strain with time during the creep period at $\sigma_o=29$ psi, $t_1=60$ min, and $T=70^\circ$F. It is seen that upon loading the volume of the specimen initially decreases (compaction) and with elapsing time increases (dilation). Figure 4.31 depicts the variation of the lateral strain/axial strain ratio with time at $\sigma_o=14.5$ psi, $t_1=30$ min, and $T=70^\circ$F.
Figure 4.30. Volumetric Strain vs. Time

Temp = 70 F
Stress = 29.0 psi
Figure 4.31. Strain Ratio (Lateral/Axial) vs. Time

Temp = 70 F

Stress = 14.5 psi
CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations which were drawn from this research are presented here.

5.1 Conclusions

Based upon the literature review the following conclusions are made:

1. Permanent deformation in the form of rutting is due to either lack of asphalt concrete resistance under high temperature, high tire pressures, large numbers of load repetitions or lack of sufficient strength in underlying layers.

2. The main physical influence factors of permanent deformation in asphalt concrete are the air void content, the asphalt viscosity, the asphalt content, the aggregate type and shape, the aggregate grading, and the compaction resulting in the interlocking of aggregate in the asphalt concrete (Table 5.1).

3. In mechanistic terms, two material properties can be the cause of permanent deformation: a) asphalt concrete plasticity, and b) asphalt concrete viscosity.

4. The permanent deformation related to the asphalt concrete plasticity is time independent, and its onset requires the stresses to surpass the yield criterion. The development of unrestricted plastic deformation resulting in asphalt concrete pavement failure due to a single load can be predicted by means of the bearing capacity formula. However, this formula gives no information on the actual
Table 5.1. Influence Factors of Rutting in Asphalt Concrete [45]

<table>
<thead>
<tr>
<th>Factor</th>
<th>Change in Factor</th>
<th>Rutting Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate</td>
<td>Smooth to rough</td>
<td>Increase</td>
</tr>
<tr>
<td></td>
<td>Open to dense</td>
<td>Increase</td>
</tr>
<tr>
<td></td>
<td>Rounded to angular</td>
<td>Increase</td>
</tr>
<tr>
<td></td>
<td>Increase max. size</td>
<td>Increase</td>
</tr>
<tr>
<td>Asphalt</td>
<td>Increase</td>
<td>Increase</td>
</tr>
<tr>
<td>Mixture</td>
<td>Increase</td>
<td>Decrease</td>
</tr>
<tr>
<td></td>
<td>Increase</td>
<td>Decrease</td>
</tr>
<tr>
<td></td>
<td>Increase</td>
<td>Decrease</td>
</tr>
<tr>
<td>Test(field) Condition</td>
<td>Increase</td>
<td>Decrease</td>
</tr>
<tr>
<td></td>
<td>Increase</td>
<td>Decrease</td>
</tr>
<tr>
<td></td>
<td>Increase</td>
<td>Decrease if mix is water sensitive</td>
</tr>
</tbody>
</table>
amount of deformation, and its applicability in predicting rutting is thus limited.

5. The permanent deformation related to the asphalt concrete viscosity is time dependent and seems to be the primary cause of rutting.

6. Two approaches have been suggested in the literature for predicting the viscosity related permanent deformation in asphalt concrete pavements: a) an uncoupled elastic-viscous approach, and b) a viscoelastic approach.

7. In the uncoupled elastic-viscous approach, the stresses in the asphalt layer are determined independently from the actual material properties; usually, this is done by means of an elastic multilayer solution. The depth of rutting then is predicted using an experimentally determined relationship between the permanent strains and the stress, temperature, and time.

8. The main advantage of the uncoupled elastic-viscous approach is the possibility of using existing numerical codes for stress determination. On the other hand, the uncoupled elastic-viscous approach introduces errors that are difficult to evaluate. For example, usually only vertical stresses are incorporated in the analysis and moving loads cannot be modeled adequately.

9. In the viscoelastic approach, the depth of rutting is determined simultaneously with the evaluation of stresses, incorporating the experimentally determined dependence of permanent strains on stress, temperature, and time.

10. The main advantage of the viscoelastic approach is the elimination of errors inherent in the uncoupled elastic-viscous approach.
However, this approach requires sophisticated mathematical methods, and the number of existing solutions is very limited.

11. There is no universally accepted laboratory method for testing the permanent deformation behavior of asphalt concrete samples.

12. The various methods suggested in the literature use uniaxial or triaxial compressions tests, with constant (static) or cyclic (dynamic) load application. The load level, time of loading, temperature, and the definition of permanent strains differ from method to method. Most of the existing test methods use cylindrical specimens of 4-in. diameter and 8-in. height (Table 5.2).

Based upon the laboratory tests and data analysis the following conclusions are made:

1. The uniaxial compression creep/recovery laboratory test is simple to perform with the help of a universal compression machine.

2. The three LVDTs mounted to clamps provided accurate measurements of the axial strains. However, the lateral displacements extensometer was not sensitive enough to obtain reliable quantitative results.

3. The compaction method employed (vibratory hammer) could not guarantee constancy of the air void content in the specimens.

4. Even though the tests were limited in scope and the test replicates were limited in number (3), the results clearly indicate the appropriateness of the test program and methodology selected.

5. The tested asphalt concrete exhibits three types of strains: elastic, plastic, and viscoelastic. All strains strongly depend on temperature and this dependence is nonlinear; with an increase in
Table 5.2. Summary of Existing Test Methods

<table>
<thead>
<tr>
<th></th>
<th>VESYS</th>
<th>Shell</th>
<th>Swiss</th>
<th>Ohio</th>
<th>Texas A&amp;M</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample shape</td>
<td>Cylinder</td>
<td>Cylinder</td>
<td>Cylinder</td>
<td>Cylinder</td>
<td>Cylinder</td>
</tr>
<tr>
<td>Sample size</td>
<td>4x8 in</td>
<td>4x2.5 in</td>
<td>4x4.7 in</td>
<td>4x8 in</td>
<td>4x8 in</td>
</tr>
<tr>
<td></td>
<td>6x4 in</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loading type</td>
<td>Uniaxial</td>
<td>Uniaxial</td>
<td>Uniaxial</td>
<td>Uniaxial</td>
<td>Uniaxial</td>
</tr>
<tr>
<td>Loading program</td>
<td>Static/dynamic</td>
<td>Static</td>
<td>Static/dynamic</td>
<td>Dynamic</td>
<td>Static</td>
</tr>
<tr>
<td>Stress</td>
<td>Variable</td>
<td>14.5 psi</td>
<td>14.5 psi</td>
<td>Variable</td>
<td>14.5 psi</td>
</tr>
<tr>
<td>Time</td>
<td>Variable</td>
<td>1 hr</td>
<td>Variable</td>
<td>0.125 s/ 0.375 s</td>
<td>1 hr/1 hr</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Temp</td>
<td>Variable</td>
<td>104 °F</td>
<td>68, 104 °F</td>
<td>Variable</td>
<td>70 °F</td>
</tr>
<tr>
<td></td>
<td></td>
<td>122 °F</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
temperature all strains increase more than proportionally.

6. The elastic strains at constant temperature are proportional to the stress level and are smaller than the plastic strains.

7. The plastic strains at constant temperature are proportional to the stress level. This means that no threshold exists below which no plastic strains develop.

8. The viscoelastic strains at constant temperature increase with time but the rate of increase decreases with time. The viscoelastic strains increase with the stress more than proportionally. This deviation from proportionality is small at temperatures 70 and 77 °F, whereas at 86 °F a sudden increase in viscoelastic strains may occur and the specimen fails.

9. The volume of the specimen upon loading initially decreases (compaction) and with elapsing time increases (dilation).

10. Due to the presence of plastic strains, the total response of the tested asphalt concrete cannot be described by a linear model. Also the viscoelastic strains show some deviation from stress linearity. However, at lower temperatures the viscoelastic response can be approximated by a relation that is linear with respect to stress.

5.2 Recommendations

Based upon the results of this study the following recommendations are made:

1. Institute uniaxial static creep/recovery tests in further research projects to examine its viability in distinguishing different materials.
2. Investigate the use of dynamic creep tests as will be proposed by the Strategic Highway Research Program in a comparison with the methodology proposed herein.

3. Investigate a variety of stress histories in a creep/recovery test to find a scheme for extracting the most useable information in the shortest possible test time.

4. Define a test procedure, starting from specimen preparation and going through data analysis, which may be implemented within Mn/DOT.

5. Provide documentation and training for Mn/DOT personnel to implement the test procedure.
CHAPTER 6
SUGGESTED IMPLEMENTATION APPROACH

One test method for determining the mechanical properties of permanent deformation in asphalt concrete is evaluated based upon this study. The uniaxial static creep test was found to be a relatively simple technique of ascertaining the permanent deformation properties of the material requiring little in the way of sophisticated equipment. However, the study was limited in that only one dense-graded asphalt mixture was tested. Further work is needed to investigate whether the test method can distinguish between mixtures of differing characteristics.

Questions have also been raised about the validity of uniaxial creep results on asphalt mixtures which do not contain densely graded aggregates. Recent research at the University of Nevada-Reno [46] indicate that a triaxial creep test is preferable, particularly in mixtures that primarily rely upon stone-to-stone contact rather than cohesion for strength. Further research is needed to investigate the use of triaxial creep tests.

Guidelines are given below relative to the collection of data, testing, and analysis required to establish criteria for the permanent deformation potential of asphalt mixtures in pavements.

6.1 Database

In order to successfully establish rutting criteria for asphalt mixtures, correlations must be found to exist between creep test results and the performance of pavements. Additionally, methods should be
established for the permanent deformation behavior of the mixtures by examining the parameters of the mixtures as well as the properties of the mixture components.

A database containing the materials information, creep test results, and construction and performance information needs to be designed and used in order to achieve the above objectives. The information required is given in sections 6.2, 6.7, and 6.8.

6.2 Materials Information

6.2.1 Aggregates

A. Coarse Aggregate

Source
Type
Percent Crushing
Gradation

B. Fine Aggregate

Source
Type
Percent Crushing
Gradation

C. Combined Aggregate Gradation

6.2.2 Asphalt Cement

Source
Grade
Properties
Penetration at 77 °F
Viscosity at 140 °F
Viscosity at 275 °F

6.2.3 Recycled Materials
Mix Type
Binder Content
Gradation

6.2.4 Final Mixture
Percent Recycled Materials
Optimum Asphalt Content
Percent Air Voids
Voids in Mineral Aggregate

6.3 Mixture and Specimen Preparation
The general procedures of the asphalt concrete mixture and the specimen preparation are summarized.

6.3.1 Calculation of Specimen Weight
The general procedure to determine the weight of specimen with the bulk specific gravity required for target air void content is as follows:
1. Combine the aggregate and the asphalt at optimum asphalt content and appropriate mixing temperature.
2. Compact asphalt mixture by Marshall method at appropriate compaction
temperature - three specimens each at 50, 75, 100 blows.
3. Calculate the bulk specific gravity of each specimen.
4. Calculate the maximum specific gravity of the asphalt mixture.
5. Calculate the air void content of each specimen.
6. Plot the air void content versus the bulk specific gravity.
7. Find the bulk specific gravity corresponding to target air void.
8. Compute the weight of specimen.

6.3.2 Mixture Preparation

The following information is for one batch.
1. Weigh the required amount of aggregates.
2. Heat the aggregates, asphalt cement, mixing bowl and molds to the mixing temperature.
3. Place the heated bowl on the automatic mixer, add the heated aggregates, and mix thoroughly.
4. Pour a small amount of the heated asphalt cement into the bowl while mixing. Check the weight after each pour until the target amount is achieved.
5. Record the actual amount and percentage of asphalt cement added.
6. Continue mixing until particles are thoroughly coated (about 2 minutes).
6.3.3 Compaction Procedure

The asphalt concrete mixture is compacted in three layers - 2.0, 4.0, and 2.0 in. of height, respectively. The compaction procedure for a single 4 x 8 in. specimen is:

1. Coat the inside surface of the 10 in. high steel mold (Fig.6.1) with a silicon spray.
2. Place a 4 in. diameter paper disc on the bottom of the mold.
3. Put one quarter of the asphalt concrete mixture for a single specimen (for bottom layer) in the mold and tamp it 25 times with a steel bar (10 times in the middle, 15 times around the edge).
4. Place another paper disc on the top of the mixture.
5. Compact the mixture with the vibratory hammer monitoring the thickness of the layer at three locations until the thickness of the layer is 2 in.
6. Remove the top paper disc.
7. Put one half of the asphalt concrete mixture for a single specimen (for middle layer) on the top of the compacted layer. Tamp the mixture with a steel bar as described in step 3.
8. Place a paper disc on the top of the mixture.
9. Compact the mixture with the vibratory hammer as described in step 5 until the total thickness of the mixture is 6 in.
10. Remove the top paper disc.
11. Put one quarter of the asphalt concrete mixture for a single specimen (for top layer) on the top of the compacted layers. Tamp the mixture with a steel bar as described in step 3.
12. Place a paper disc on the top of the mixture.
Figure 6.1 Outline of Asphalt Concrete Mixture Compaction
13. Insert a 2 in. high steel cap (Fig. 6.1) in the mold.
14. Compact the mixture with the vibratory hammer until the edge of the cap meets the top of the mold.
15. Remove the cap and the paper disc.

6.3.4 Specimen Preparation

Following steps are used to prepare the specimens:

1. Prepare one batch of asphalt mixture as described in Mixture Preparation.
2. Place the mixture in a large pan.
3. Scrape the sides and bottom of the bowl as well as the paddles to remove any material clinging to them.
4. Compact the mixture as described in Compaction Procedure.
5. Remove the bottom of the mold and write the batch number and the date on the bottom surface.
6. Let the specimen rest for 1 hour after compaction.
7. Remove the specimen from the mold using an extruder and pushing the specimen from the bottom of the mold.
8. Write the specimen number on the surface of the specimen.
9. Measure and calculate the bulk specific gravity of the specimen.
10. Measure with a caliper the height of the specimen at three different locations at 120° apart and mark these locations.
11. Lubricate each end of the specimen with silicone grease.
12. Store the specimen at room temperature of 70 ± 2 °F.
13. Drill a hole of 0.25 in. diameter and 2 in. depth at mid-height of the dummy specimen.
6.4 Experimental Setup

The outline of the uniaxial static creep test device is shown in Fig.6.2. The main equipment and its role is as follows:

A. **Loading platens**: For applying the load to the specimen at each end; diameter of 6 in., height of 1 in., and well polished.

B. **Loading lever**: For holding a weight and a piston; length of 4 ft., width of 2 in., and depth of 3 in. (Fig.6.2).

C. **Piston**: For applying the load to the specimen; diameter of 2 in. and 2 ft. of height (Fig.6.2).

D. **Weight**: For applying the load to the specimen; 35.6 lbs. (Fig.6.2).

E. **Hydraulic Jack**: For adjusting the loading lever (Fig.6.2).

F. **LVDTs**: For measuring the vertical displacements of the specimen; length of 3 in. and range of ± 0.25 in.; distance between the LVDT and specimen surface is approximately 0.5 in. (Fig.6.3).

G. **Clamps**: For mounting the LVDTs to the specimen (Fig.6.3).

H. **Environmental chamber**: For maintaining the test temperature constantly for during the test.

I. **Data acquisition/processing system**: For recording/storing the data.

At least 5 channels are needed for:

1. LVDT # 1 (vertical displacement at $\alpha = 0^\circ$)
2. LVDT # 2 (vertical displacement at $\alpha = 120^\circ$)
3. LVDT # 3 (vertical displacement at $\alpha = 240^\circ$)
4. Thermocouple # 1 (dummy specimen temperature)
5. Thermocouple # 2 (environmental chamber temperature)
Figure 6.2 Uniaxial Static Creep Test Device
Figure 6.3 Outline of LVDTs Setting
6.5. Testing

The general procedure for the creep test is summarized. The testing conditions should be a temperature of 77 or 86 °F (or both) and an applied stress of 14.5 psi with a loading time of one hour and an unloading time of one hour. The test procedure is as follows:

1. Remove the test specimen from the storage freezer and let it rest at the room temperature of T = 70 °F (21 °C) for 24 hours.

2. Measure the height of the test specimen with a caliper at three different locations 120° apart and mark these points.

3. Set the clamps on the surface of the test specimen at 2.5 in. from each end of the specimen.

4. Attach the LVDTs to the clamps at equal angle of 120° around the circumference.

5. Measure with caliper the distance between the clamps at location of LVDTs.

6. Insert the thermocouple #1 into the hole of the dummy specimen.

7. Place the dummy specimen in the environmental chamber 1 in. away from the lower platen and connect the wiring of the thermocouple #1 to the data acquisition/processing system.

8. Place the test specimen on the lower platen in the environmental chamber and connect the wiring of the LVDTs to the data acquisition/processing system.

9. Heat the environmental chamber to the required test temperature.

10. Move down the loading lever with the hydraulic jack until the loading platen contacts the test specimen gently.

11. Check each connection to the data acquisition/processing system.
12. When the thermocouple #1 indicates the required test temperature, move down the loading lever to touch the test specimen for two minutes, then move up the loading lever and let the specimen rest for 10 minutes.

13. Set the digital indicators of the LVDTs to 0.

14. Put the weight at the end of the loading lever and move down them to touch the specimen for one hour.

15. Move up the loading lever, remove the weight, and let the test specimen rest for one hour.

16. Turn off the heating switch and disconnect the wiring from the LVDTs, and thermocouples from the data acquisition/processing system.

17. Remove the specimens from the environmental chamber.

18. Remove the LVDTs, clamps, and thermocouple from the specimen.

19. Measure with a caliper the height of the specimen at three locations as specified in step 2.

20. Store the dummy specimen at room temperature.

21. After one test is completed, use another specimen and repeat the steps above.

6.6. Results

For each test, obtain a plot of axial displacement versus time as shown in Figure 6.4. This may be done either by direct means of a strip chart recorder or by acquiring the data automatically in a computer. In the latter case, the following sampling rates are recommended:
Figure 6.4 Loading Program and Displacement Measurement
<table>
<thead>
<tr>
<th>Test Time Interval, min.</th>
<th>Sampling Rate, samples/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 1</td>
<td>10.0</td>
</tr>
<tr>
<td>1 to 10</td>
<td>1.0</td>
</tr>
<tr>
<td>10 to 60</td>
<td>0.1</td>
</tr>
<tr>
<td>60 to 61</td>
<td>10.0</td>
</tr>
<tr>
<td>61 to 70</td>
<td>1.0</td>
</tr>
<tr>
<td>70 to 120</td>
<td>0.1</td>
</tr>
</tbody>
</table>

Convert the axial displacement readings obtained in the test to axial strains by dividing the displacement readings by the gage length of the measurement (3 inches). Plot the axial strain versus time as shown in Figure 6.5.

6.7 Analysis

Determine the instantaneous creep strain ($\epsilon_c(0)$) upon loading and the instantaneous recoverable strain ($\epsilon_r(0)$) upon unloading as illustrated in Figure 6.5. These values should be obtained from the vertical portions of the curve. The elastic strain ($\epsilon_e$) is same as the instantaneous recoverable strain ($\epsilon_r(0)$). The plastic strain ($\epsilon_p$) is calculated as the difference between the instantaneous creep strain ($\epsilon_c(0)$) and the elastic strain ($\epsilon_e$). The permanent strain ($\epsilon_{pr}(t)$) can be determined from the test result directly since the permanent strain is same as the recovery strain ($\epsilon'_r(t)$). The details of the analysis methodology is given in chapter 4 of this report.
Figure 6.5 Typical Result of Creep/Recovery Test
Record the following values into the database:

1. The creep strains ($\epsilon_c(t)$) for every 10 min.
2. The recovery strains ($\epsilon'_r(t)$) for every 10 min.
3. The elastic strain ($\epsilon_e$).
4. The plastic strain ($\epsilon_p$).

6.8 Project Construction and Performance Information

For each project under evaluation, the following construction information should be obtained:

1. Location by beginning and ending milepost.
2. Type of project (new construction, reconstruction, overlay).
3. Date of construction.
4. Type of asphalt mixture.
5. Thickness of asphalt concrete.
6. Type of material in layer 2.
7. Thickness of material in layer 2.
8. Type of material in layer 3.
9. Thickness of material in layer 3.
10. Type of material in layer 4.
11. Thickness of material in layer 4.
12. Type of subgrade.

The following performance data should be collected one time per month in the summer.

1. Cumulative ESALs since opening to traffic or HCADT.
2. Average rut depth measured at a rate of five times per mile
with not less than five measurements for any project. Rut depth measurements are to be made at the same locations for each survey.

3. High and low air temperature for each month after construction.

6.9 Correlations

When a sufficient number of projects have been evaluated, the data should be sorted and analyzed to investigate potential correlations between creep test results and construction and performance data. For instance, a strong relationship may exist between rut depth and total permanent strain, cumulative traffic, and average air temperature.

At the same time, it would be useful to examine possible relationships between the creep behavior of mixtures and the mixture parameters and component properties.
REFERENCES


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APPENDIX A

EXPERIMENTATION

A.1 Experimental Setup

The outline of the experimental setup is shown in Fig.A.1. The main equipment and its role is as follows:

A. **Loading platens**: For holding the specimen at each end; diameter of 6 in. (150 mm), height of 0.5 in. (12.7 mm), and well polished.

B. **Loading frame**: For applying the test stress on the specimen; load capacity of 550 lbs (2.4 kN) and stress control over 2 hours.

C. **LVDTs**: For measuring the vertical displacements of the specimen; length of 3 in. and range of ± 0.25 in. The LVDTs are located at mid-height of the specimen at equal angles of 120° around the perimeter of the specimen. The distance between the LVDT and specimen surface is approximately 0.5 in. (12.7 mm) (Fig.A.2).

D. **Clamps**: For mounting the LVDTs at location as specified in point C of this section (Fig.A.2).

E. **Environmental chamber**: For maintaining the constant temperature of up to 113 °F (45 °C) during the test.

F. **Extensometer**: For measuring the varying circumference of the specimen at mid-height (Fig.A.2).

G. **Data acquisition/processing system**: For recording/storing the data (two temperatures, stress, time, three vertical displacements, and circumference). At least 7 channels are needed for:
Figure A.1. Outline of Experimental Setup
Figure A.2. Setting of LVDTs and Extensometer
1. LVDT #1 (vertical displacement at $\alpha = 0^\circ$)
2. LVDT #2 (vertical displacement at $\alpha = 120^\circ$)
3. LVDT #3 (vertical displacement at $\alpha = 240^\circ$)
4. Thermocouple #1 (dummy specimen temperature)
5. Thermocouple #2 (environmental chamber temperature)
6. Load cell (vertical load)
7. Extensometer (change in circumference)

A.2 Test Procedure

The test procedure is as follows:

1. Remove the test specimen from the storage freezer and let it rest at the room temperature of $T = 70 \, ^\circ\text{F} (21 \, ^\circ\text{C})$ for 24 hours.

2. Measure the height of the test specimen with a caliper at three different locations 120° apart and mark these points.

3. Set the clamps on the surface of the test specimen at 2.5 in. from each end of the specimen.

4. Attach the LVDTs to the clamps at equal angle of 120° around the circumference. Attach the extensometer at mid-height of the specimen.

5. Measure with caliper the distance between the clamps at location of LVDTs.

6. Insert the thermocouple #1 into the hole of the dummy specimen.

7. Place the dummy specimen in the environmental chamber 1 in. away from the lower platen and connect the wiring of the thermocouple #1 to the data acquisition/processing system.

8. Place the test specimen on the lower platen in the environmental
chamber and connect the wiring of the LVDTs and extensometer to the
data acquisition/processing system.

9. Heat the environmental chamber to the required test temperature.

10. Lower the loading piston with the loading platen attached until it
gently contacts the test specimen.

11. Check each connection to the data acquisition/processing system.

12. When the thermocouple # 1 indicates the required test temperature,
apply the preloading stress of 1.45 psi (10 kN/m²) for 2 minutes,
remove the stress and let the specimen rest for 10 minutes.

13. Set the digital indicators of the LVDTs and extensometer to 0.

14. Apply the test stress for required test time.

15. Remove the test stress and let the test specimen rest for the
required time.

16. Turn off the heating switch and disconnect the wiring from the
LVDTs, extensometer, and thermocouples from the data acquisition/
processing system.

17. Remove the specimens from the environmental chamber.

18. Remove the LVDTs, clamps, extensometer, and thermocouple from the
specimen.

19. Measure with a caliper the height of the specimen at three locations
as specified in step 2.

20. Store the dummy specimen at room temperature.

21. After one test is completed, use another specimen and repeat the
steps above.
APPENDIX B

MATERIALS

The characteristics and properties of the selected materials are listed as follows:

B.1 Aggregate

To make the asphalt concrete mixture three aggregates, BA 003, BA 004, and BA 005, were selected. Each aggregate comprises two types of particles: + No.4 (coarse particles) and - No.4 (fine particles). The characteristics of these aggregates are described below:

Source: Commercial Asphalt Inc., Osseo, Minnesota

Type

BA 003: 95% crushed gravel
BA 004: screened gravel
BA 005: 100% crushed limestone

Ratio of coarse/fine particles

BA 003: 77/23
BA 004: 11/89
BA 005: 57/43

Particle shape

BA 003: flat, triangular to rectangular/disc
BA 004: ovoid, semirounded
BA 005: cubical, rectangular
Particle texture

BA 003: uneven, crushed, smooth to rough
BA 004: smooth to semirough
BA 005: powdery, porous

Bulk specific gravity

BA 003
coarse: 2.593
fine: 2.603
combined: 2.595

BA 004
coarse: 2.662
fine: 2.692
combined: 2.689

BA 005
coarse: 2.654
fine: 2.670
combined: 2.661

% of water absorption

BA 003
coarse: 1.48
fine: 2.89
combined: 1.80

BA 004
coarse: 1.36
fine: 1.40
combined: 1.40

BA 005
coarse: 2.19
fine: 2.23
combined: 2.21
Proportion in combined aggregate

BA 003 : 25 %
BA 004 : 50 %
BA 005 : 25 %

B.2 Asphalt Cement (Binder)

Source: Ashland Co., St. Paul Park, Minnesota

Penetration grade: 120-150

Penetration (at 77 °F): 127 dmm

PVN (penetration viscosity number)

at 140 °F: - 1.26
at 275 °F: - 1.13

Viscosity

at 140 °F: 454 poise
at 275 °F: 1.89 poise

Specific gravity: 1.017
APPENDIX C

SPECIMENS

C.1 Mixture Preparation

Six (6) specimens are made in one batch. The following information is for one batch.

1. Weigh the aggregates as follows:

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<th>Size</th>
<th>Weight (gram)</th>
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<tr>
<td></td>
<td></td>
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<td>-4</td>
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<tr>
<td></td>
<td></td>
<td>subtotal (pan 4) 5,500 gram</td>
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</table>

Total 22,000 gram

2. Heat the aggregates to 290 ± 10 °F for 12 hours.

3. Heat the asphalt cement to 290 ± 10 °F for 4 hours.

4. Heat the mixing bowl (Fig.C.1) and molds (Fig.C.2) to 290 ± 10 °F for 4 hours.

5. Place the heated bowl on the automatic mixer (Fig.C.3), add the hot aggregates, and mix thoroughly.
Figure C.1. Mixing Bowl
Figure C.2. Specimen Mold
Figure C.3. Automatic Mixer
6. Pour small amount of the asphalt cement into the bowl while mixing. Check the weight after each pour until the target amount \(1,280.4 \pm 1.1\) gram is achieved.

7. Record the actual amount and percentage of asphalt cement added.

8. Continue mixing until particles are thoroughly coated (about 2 minutes).

C.2 Compaction Method

The Marshall method is the typical compaction technique used in Minnesota. However, since the Marshall method cannot be used for 4 x 8 in. specimens a somewhat equivalent compaction method had been developed. This compaction method uses 10 in. high compaction mold and the Black and Decker Type I vibratory hammer (3000 BPM) (Fig.C.4) compacting three layers of asphalt concrete mixture until required thickness of each layer is obtained. The following describes the methodology used to develop this method.

Ten thousand (10,000) grams of asphalt mixture were prepared according to the procedure described in Mixture Preparation. Six (6) 4 x 2.5 in. specimens were compacted with Marshall method using three different number of blows: 2 specimens were compacted with 75 blows, 2 specimens with 100 blows, and 2 specimens with 125 blows. The bulk specific gravity and air void content of each specimen were determined. The maximum specific gravity was determined using the rest of the mixture. The results were as follows:
Figure C.4. Vibratory Hammer
Group # 1 (75 blows)

Bulk specific gravity = 2.377
Air void content = 4.00 %

Group # 2 (100 blows)

Bulk specific gravity = 2.388
Air void content = 3.55 %

Group # 3 (125 blows)

Bulk specific gravity = 2.391
Air void content = 3.43 %

Maximum specific gravity = 2.476

Using the data above, plot was constructed relating the air void content to the bulk specific gravity (Fig.C.5). For the target air void content of 6.0%, the corresponding bulk specific gravity is 2.33. Knowing the maximum specific gravity and the target bulk specific gravity, the weight of a 4 x 2.5 in. specimen was determined as 1203 grams. Thus, the required weight of a 4 x 8 in. specimen with the target air void content 6.0% is 3850 grams. It follows then that using this amount of mixture in a 10 in. mold, compaction with the vibratory hammer to height of 8 in. should guarantee the required air void content.

The above the compaction method does not take into account the nonuniformity of compaction in a 10 in. mold due to the friction between the mixture and the mold. To reduce friction effect, the mixture can be compacted in several layers; three (3) layers of compacted thickness 2 in., 4 in., and 2 in. were selected. The corresponding weights of these layers are: 2 in. layer - weight 963 grams, 4 in. layer - weight 1925
Figure C.5. Air Void Content vs. Bulk Specific Gravity
grams. The adopted compaction procedure is described in C.3.

It should be mentioned here that the compaction procedure adopted assumes that each compacted layer does not compact anymore if the next layer is compacted. This is not true, and results in slightly less compaction of the middle and upper layers (Table C.1).

C.3 Compaction Procedure

The compaction procedure for a 4 x 8 in. specimen is:

1. Coat the inside surface of the 10 in. high steel mold with a silicon spray.
2. Place a 4 in. diameter paper disc on the bottom of the mold.
3. Put 963 grams of the asphalt concrete mixture in the mold and tamp it 25 times with a steel bar (10 times in the middle, 15 times around the edge).
4. Place another paper disc on the top of the mixture.
5. Compact the mixture with the vibratory hammer monitoring the thickness of the layer at three locations until the thickness of the layer is 2 in.
6. Remove the top paper disc.
7. Put 1,925 grams of the asphalt concrete mixture on the top of the compacted layer. Tamp the mixture with a steel bar as described in step 3.
8. Place a paper disc on the top of the mixture.
9. Compact the mixture with the vibratory hammer as described in step 5 until the total thickness of the mixture is 6 in.
10. Remove the top paper disc.
Table C.1. Characteristics and Properties of Layers

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<tr>
<th>Sample # &amp; Layer</th>
<th>Dry wt (gram)</th>
<th>SSD wt (gram)</th>
<th>IMM wt (gram)</th>
<th>Volume (cm$^3$)</th>
<th>B.S.G</th>
<th>M.S.G</th>
<th>% Air Voids</th>
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T : Top  
M : Middle  
B : Bottom
11. Put 963 grams of the asphalt concrete mixture on the top of the compacted layers. Tamp the mixture with a steel bar as described in step 3.

12. Place a paper disc on the top of the mixture.

13. Insert a 2 in. high steel cap (Fig.C.6) in the mold.

14. Compact the mixture with the vibratory hammer until the edge of the cap meets the top of the mold.

15. Remove the cap and the paper disc.

C.4 Specimen Preparation

Required number of specimen = 69
Creep test = 60
Spare = 7
Dummy specimen = 2

Required amount of aggregate : 900 lbs (408 kg)
Required amount of asphalt : 50 lbs (22.7 kg)

Following steps were used to prepare the specimens:

1. Prepare one batch of asphalt mixture as described in Mixture Preparation.

2. Place the mixture in a large pan.

3. Scrape the sides and bottom of the bowl as well as the paddles to remove any material clinging to them. This material had a high asphalt content and had to be carefully and thoroughly blended by hand into the rest of the mixture.
Figure C.6. Steel Cap
4. Compact the mixture as described in Compaction Procedure.
6. Remove the bottom of the mold and write the batch number and the date on the bottom surface.
7. Let the specimens rest for 1 hour after compaction.
8. Remove the specimen from the mold using the electrical extruder and pushing the specimen from the bottom of the mold.
9. Write the specimen number on the surface of the specimen.
10. Measure and calculate the bulk specific gravity of the specimen.
11. Measure with a caliper the height of the specimen at three different locations at 120° apart and mark these locations.
12. Lubricate each end of the specimen with silicone grease.
13. Store the specimen at room temperature of 70 ± 2 °F.
14. Repeat steps 1 through 13 for 64 specimens.
15. Drill a hole of 0.25 in. diameter and 2 in. depth at mid-height of the dummy specimen.

Table C.2 list the specimens' characteristics and properties. The summary of these are presented as follows:

Asphalt content : 5.50 %
Aggregate content : 94.50 %
Mixing temperature : 290 ± 10 °F
Compaction type : vibratory hammer (one side)
Compaction temperature : 275 ± 10 °F
Table C.2. Specimens’ Characteristics and Properties

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<th>Imm. wt. (grams)</th>
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<th>A.V. (%)</th>
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Bulk S.G. Average 2.329
Bulk S.G. STD 0.014

A.V. Average 5.928
A.V. STD 0.582

VMA Average 16.593
VMA STD 0.516

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% Air void content

Range : 4.32 - 7.11
Average : 5.93
Standard deviation : 0.58

VMA (Voids in the mineral aggregate)

Range : 15.168 - 17.639
Average : 16.593
Standard deviation : 0.516

Bulk specific gravity

Range : 2.300 - 2.369
Average : 2.329
Standard deviation : 0.014

Maximum specific gravity : 2.476
APPENDIX D

TEST RESULTS
Figure D.1. Test Results at $\sigma_0=14.5$ psi, $t_1=5$ min, and $T=70$ °F
Figure D.2. Test Results at $\sigma_0=14.5$ psi, $t_1=30$ min, and $T=70^\circ$F
Figure D.3. Test Results at $\sigma_0=14.5$ psi, $t_1=60$ min, and $T=70^\circ F$
Figure D.4. Test Results at σ₀=14.5 psi, t₀=120 min, and T=70 °F

Temp = 70°F
Stress = 14.5 psi
Temp = 70°F
Stress = 21.75 psi

Figure D.5. Test Results at \( \sigma_0 = 21.75 \) psi, \( t_i = 5 \) min, and \( T = 70 \) °F
Temp = 70°F
Stress = 21.75 psi

Figure D.6. Test Results at \( \sigma_0 = 21.75 \text{ psi} \), \( t_1 = 30 \text{ min} \), and \( T = 70 \, ^\circ\text{F} \)
Figure D.7. Test Results at $\sigma_0=21.75$ psi, $t_1=60$ min, and $T=70$ °F
Figure D.8. Test Results at $\sigma_0=21.75$ psi, $t_j=120$ min, and T=70°F

Temp = 70°F
Stress = 21.75 psi
Figure D.9. Test Results at $\sigma_0=29.0$ psi, $t_1=5$ min, and $T=70^\circ$F
Figure D.11. Test Results at $\sigma_0 = 29.0$ psi, $t_1 = 60$ min, and $T = 70$ °F
Figure D.12. Test Results at $\sigma_0 = 29.0$ psi, $t_i = 120$ min, and $T = 70 \, ^\circ \text{F}$.
Figure D.13. Test Results at \( \sigma = 14.5 \) psi, \( T = 5 \) min, and \( T = 77 \) °F.
Figure D.15. Test Results at $\sigma_o=14.5$ psi, $t_i=60$ min, and $T=77$ °F
Figure D.16. Test Results at $\sigma_0=14.5$ psi, $t_1=5$ min, and $T=86 ^\circ F$
Figure D.17. Test Results at \( \sigma_0 = 14.5 \text{ psi}, T = 30 \text{ min}, \text{ and } T = 86^\circ \text{ F} \)
Figure D.19. Test Results at $\sigma_0 = 14.5$ psi, $t_1 = 120$ min, and $T = 86$ °F
Figure D.20. Test Results at $\sigma_o=21.75$ psi, and $T=86^\circ$F