Permanent Deformation Properties of Asphalt Concrete Mixtures

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SUMMARY

Rutting is recognized to be the major distress mechanism in flexible pavements as a result of increase in tire pressures and axle loads. Rutting is caused by the accumulation of permanent deformation in all or some of the layers in the pavement structure. The accumulation of permanent deformation in the asphalt surfacing layer is now recognized to be the major component of rutting in flexible pavements. This is a consequence of increased tire pressures and axle loads, which subjects the asphalt surfacing layer nearest to the tire-pavement contact area to increased stresses. Thus the study of permanent deformation properties of asphalt mixtures has become the focus of research, which aim to mitigate or reduce rutting in flexible pavements. The research work reported in this thesis aims to contribute towards understanding of the material properties and factors affecting permanent deformation in asphalt mixtures, mechanisms of the permanent deformation, and methods of its prediction.

The specific objectives of this research work include; review and evaluation of available models for permanent deformation of asphalt concrete mixtures, investigation of the effect of volumetric composition, loading and temperature conditions on the permanent deformation of asphalt concrete, and the identification and definition of simple measures of resistance to permanent deformation. To meet the objectives of the study a laboratory investigation is conducted on several asphalt concrete specimens with varying volumetric composition. Two testing procedures are adopted; the repeated load triaxial and triaxial creep and recovery tests. The tests were conducted at two temperature levels of 25 and 50^{o}C under varying stress conditions. A review of literature on factors affecting permanent deformation and available models for prediction of the permanent deformation is also conducted.

The literature review indicated that most of the research work done so far concentrated on evaluation of the effect on permanent deformation response of component material properties such as aggregate gradation, aggregate angularity and binder type (or grade). Most of the studies conducted on permanent deformation properties of asphalt mixtures were also found to be based on different testing procedures and methods of evaluation, which makes it difficult to compare them and draw firm conclusions. The literature also indicated that, as yet, there is no comprehensive model for deformation of asphalt concrete.

Results of tests conducted in this study are analysed to investigate the effect of volumetric composition, particularly binder content and void content, and loading conditions on the permanent

deformation response of the mixture. Both the binder content and void content are found to significantly influence the permanent deformation characteristics. The effect of loading conditions, i.e., the confining stress and the deviatoric stress, is also found to be significant.

Throughout this study emphasis is placed on methods and parameters that are used to evaluate mixtures for their resistance to permanent deformation.The traditionally used parameters such as the slope and intercept of the power model are evaluated for their sensitivities to changes in volumetric composition. This evaluation is based on the premises that any measure of resistance to permanent deformation should be sensitive to changes in volumetric composition to be good enough. It is found that most of these parameters are not sensitive to changes in volumetric composition and therefore are not suitable for comparison of mixtures made from the same materials but with varying proportion of the components.

Permanent deformation in asphalt concrete is caused by both densification and shear deformation.The mode of deformation in asphalt concrete pavements, for greater part of their service life, is considered to be the shear deformation. Therefore it is necessary to evaluate mixtures for their susceptibility to shear deformation. The shear deformation manifests itself in the form of large lateral deformation relative to axial deformation. It is found that one dimensional analysis, which does not take the lateral deformation into account may lead to misleading results regarding the resistance to permanent deformation of mixtures. Therefore parameters which include volumetric and lateral strain are proposed for use in evaluation of mixtures.

Substantial effort is put into modelling the accumulation of permanent deformation under repeated loading. For this purpose two approaches were selected: the cyclic hardening model based on bounding surface plasticity concept and an elasto-viscoplastic model based on strain decomposition approach. The bounding surface plasticity approach is found to be a convenient method to model the accumulation of permanent deformation. It is demonstrated that deformations calculated using cyclic hardening model based on bounding surface plasticity fits the measured deformation quite well. The elasto-viscoplastic model, which is based on strain decomposition approach, provides a suitable method for analysis of creep and recovery test results. Deformations calculated using this model also fit the measured deformation quite well.

Finally a new composite measure of resistance to permanent deformation is developed. The resistance index is based on strain decomposition approach and is simple to calculate. The index incorporates a parameter related to shear susceptibility of mixtures and is sensitive to changes in volumetric composition. It is believed that this index can be used to compare and select mixtures at mixture design stage. If its applicability to other materials is proved by further research, it can also be linked to performance related specifications, as a simple measure of performance with regard to rutting.

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LIST OF SYMBOLS AND ABBREVIATIONS

Symbol	**Meaning**
V_a	Air void content of an asphalt mixture
P_b	Binder content
VMA	Void in mineral aggregate
V_{ba}	Absorbed asphalt volume
V_{beff}	Effective asphalt volume
VFA	Voids filled with asphalt
S_m	Stiffness modulus of asphalt mixture
S_b	Stiffness modulus of the binder
v_b	Percent volume of binder
v_g	Percent volume of aggregate
$\lvert E^*\rvert$	Dynamic modulus
η	Viscosity
f	Frequency of loading
τ	Shear strength
c	Cohesion
φ	Angle of internal friction
C	Degree of complex flow
S	Shear rate
T	Shear stress
G*	Complex shear modulus
δ	phase angle
W_c	Work dissipated per cycle
σ	Stress
ε	Strain
G’’	Loss modulus
G’	Storage modulus
η_0	Zero-shear-viscosity
ω	Angular frequency
J(t)	Compliance
R	Spring constant
t_R	Relaxation time

t_c	Retardation time
ε_e	Elastic component of strain
ε_{ve}	Viscoelastic component of strain
ε_p	Plastic component of strain
ε_{vp}	Viscoplastic component of strain
ε_{ij}	Strain tensor
I_1	The first stress invariant
J_2	The second deviatoric stress invariant
ε_1	Major principal strain
ε_3	Minor principal strain
σ_1	Major principlal stress
σ_3	Minor principal stress
K	Ratio of incremental work in to incremental work out
ε_v	Volumetric strain
σ_{ij}	Stress tensor
ζ	Irrecoverable deviatoric strain trajectory
ξ	Plastic strain tarjectory
h_c	Cyclic hardening parameter
D_e	Elastic compliance parameter
D_p	Plastic compliance parameter
D_{ve}	Viscoelastic compliance parameter
D_{vp}	Viscoplastic compliance parameters
δ_{ij}	Kronecker delta
ε_{ij}^{p}	irrecovarable strain tensor
$\varepsilon^{e}ij$	Elastic strain tensor
s_{ij}	Deviatoric strain tensor
μ	Shear modulus
E	Elastic modulus
γ	Viscosity constant
Abbreviations	**Meaning**
SHRP	Strategic highway research program
AASHTO	American association of state highway and transportation officials
ASTM	American society for testing and materials

DEM	Discrete element method
VESYS	A computer program for analysing a multi-layer viscoelastic pavement system
SuperPave	Superior performing pavements
SST	SuperPave shear tester
SSD	Saturated surface dry

CHAPTER 1: INTRODUCTION

1.1 Background

Permanent deformation in the form of rutting is one of the most important distress (failure) mechanisms in asphalt pavements. With increase in truck tire pressure in recent years, rutting has become the dominant mode of flexible pavement failure.Pavement rutting, which results in a distorted pavement surface, is primarily caused by the accumulation of permanent deformation in all or a portion of the layers in the pavement structure. Rutting can also be caused by wear of pavements resulting from use of studded tires. Longitudinal variability in the magnitude of rutting causes roughness. Water may become trapped in ruts resulting in a reduced skid resistance, increased potential for hydroplaning and spray that reduces visibility. Progression of rutting can lead to cracking and eventually to complete disintegration or failure. Rutting accounts for a significant portion of maintenance and associated costs in both main highways and secondary roads.

The economics of truck transportation has caused the average gross weight of trucks to increase so that a majority of trucks are operating close to the legal axle loads limits. In countries where enforcement of the legal axle load limits is relaxed or non-existent (typical of developing countries), trucks operate at axle loads, which by far exceed the legal axle load limit. As axle loads have increased, the use of higher tire pressures has become more popular in the trucking industry. Higher tire pressures reduce the contact area between the tire and the pavement, resulting in high stress which contributes to greater deformation in flexible pavements, manifested as severe wheel track rutting.

As a consequence of the increased tire pressure and axle load, the surfacing asphalt layer is subjected to increased stresses, which result in permanent (irrecoverable) deformations. The permanent deformation accumulates with increasing number of load applications. The permanent deformation in the surfacing layer thus accounts for a major portion of rutting on flexible pavements subjected to heavy axle loads and high tire pressures.

1.2 Problem Statement

Although the rutting observed on flexible pavements can be the total sum of accumulated permanent deformations in one or more layers of the pavement structure, the accumulation of permanent deformation in the asphalt surfacing layer is now considered to be the major cause of rutting. To minimize this form of rutting, it is necessary to pay extra attention to material selection and mixture design. To be able to design a mixture that has adequate resistance to rutting, knowledge of the effect of mixture composition and properties of the component materials is of paramount importance. Furthermore, the questions of how to measure rutting resistance of asphalt mixtures, what parameters to use as a measure of resistance, and how to model and predict the development of permanent deformation need to be addressed. In particular, the issue of development of simple performance tests and measure of performance with regard to rutting have become the focus of current research.

Several research works have been conducted on permanent deformation of asphalt concrete materials. Most of these research works were conducted on different materials using various testing procedures and mainly based on uniaxial tests. Thus it is very difficult to make comparisons and draw conclusions. Furthermore, the methods of analysis and the parameters used to evaluate the permanent deformation behaviour of mixtures in most of these studies were found to be inadeqaute. Thus, there is a need to make more detailed studies of the permanent deformation response of asphalt concrete mixtures.

An attempt is made in this study to tackle the issues raised in the preceding paragraphs. Based on laboratory tests that are judged to be simulative of field loading conditions, the study attempts to provide more knowledge on the effect of volumetric composition, loading, and temperature conditions on permanent deformation response of asphalt concrete mixtures. In particular substantial effort is made to evaluate various measures of rutting resistance in terms of their sensitivity to change in volumetric composition and to define a simple measure of resistance that can be linked to mixture design. Modelling the permanent deformation behaviour of asphalt concrete mixtures forms the other major part of this study.

1.3 Objectives

The objectives of this study are:

1. to review and evaluate available models for permanent deformation response of asphalt mixtures with the aim of selecting an appropriate model or making some improvements,
2. to investigate the effect of volumetric composition, loading and temperature conditions on permanent deformation behaviour of asphalt mixtures,
3. to identify important material parameters that are related to the resistance to permanent deformation of asphalt mixtures, and
4. to define a measure of resistance to permanent deformation of mixtures and to investigate its sensitivity to changes in volumetric composition.

1.4 Methodology

The methodology adopted to meet the objectives of this study involves a review of literature and a laboratory investigation. The literature review is conducted to identify important component material properties, that influence the permanent deformation response of mixtures, and available permanent deformation models and their theoretical basis. Testing methods that are used to characterize permanent deformation property of asphalt mixtures are also reviewed.

The laboratory investigation is conducted using two testing procedures; the cyclic load triaxial test and the triaxial creep and recovery test. Specimens made with different levels of binder content and void content are tested in both procedures.The cyclic load triaxial test results are used both for modelling purposes and evaluation of the effect of various factors on the permanent deformation response. The creep and recovery test results are used to study the various components of permanent strain in connection with an elasto-viscoplastic modelling approach and to define a measure of resistance to rutting (permanent deformation).

1.5 Organization

This thesis is divided into 8 chapters. Following the introductory first chapter, chapter 2 discusses the problem of rutting in flexible pavements with emphasis on the rutting caused by accumulation of permanent deformation in asphalt layers. Chapter 2 also briefly reviews the methods that have been used to take the rutting resistance of asphalt mixtures into account both in the mixture design and in structural design of pavements.

Investigation of the effect of volumetric composition and properties of the component materials on resistance to permanent deformation forms a substantial part of this work. Accordingly chapter 3 deals with review of the effect of composition, aggregate properties, and binder properties on permanent deformation behaviour of asphalt mixtures.

As in all other aspects of pavement engineering, the prediction of permanent deformation has traditionally relied on empirical methods. However, some attempts have been made to use the more fundamental mechanics based material modelling approaches to describe the deformation response of asphalt concrete mixtures. Understanding the theoretical basis and limitations of these modelling approaches would assist in selection of appropriate model and/or modelling method. Chapter 4 presents review of these models. A summary of the more traditional permanent deformation models and equations is also provided in chapter 4.

Chapter 5 describes the testing and experimental procedure used in this study. Beginning with discussion on various test methods that are used to characterize the deformation behaviour of asphalt concrete mixtures, it provides the justification for selection of the particular test methods adopted in this study. Chapter 5 also presents the materials used in this study.

Chapter 6 presents and discusses the results of the laboratory tests. Using graphical presentation, the observed effects of various factors on the permanent deformation behaviour of asphalt mixture used in this study is discussed. Evaluation of several parameters, which are used to characterize the resistance to permanent deformation, for their sensitivity to changes in volumetric composition of the material is a central issue in this study. The evaluation of some of these traditionally used parameters is also discussed in chapter 6.

Chapter 7 deals with modelling of permanent deformation response of asphalt mixtures under repeated loading. First the mechanism of deformation of asphalt concrete materials under load is discussed. Then the development of permanent deformation is modelled using two modelling approaches: the bounding surface plasticity and the elasto-viscoplastic method based on strain decomposition. Chapter 7 also presents the definition of a new measure of resistance to rutting, which is defined based on the model parameters, and its relation to volumetric composition of mixtures. Chapter 8 presents the conclusions and recommendations from this thesis work.

CHAPTER 2: THE PROBLEM OF RUTTING IN FLEXIBLE PAVEMENTS

Rutting is one of the major distress mechanisms in flexible pavements. Because of the increase in tire pressure and axle loads in recent years, rutting has become the dominant mode of failure of flexible pavements in many countries. There are various causes of rutting depending on configuration and structural capacity of the various layers and environmental conditions. In this chapter, the problem and the mechanisms of rutting in flexible pavements in general and the rutting caused by permanent deformation in the asphalt layer in particular are discussed. The consideration of rutting at the pavement design and mixture design stages are also discussed.

2.1 Rutting in Flexible Pavements

Rutting is a longitudinal surface depression in the wheel path accompanied, in most cases, by pavement upheaval along the sides of the rut. Significant rutting can lead to major structural failure and hydroplaning, which is a safety hazard. Rutting can occur in all layers of the pavement structure and results from lateral distortion and densification. Moreover, rutting represents a continuous accumulation of incrementally small permanent deformations from each load application.

Eisemann and Hilmar[1] studied asphalt pavement deformation phenomenon using wheel tracking device and measuring the average rut depth as well as the volume of displaced materials below the tires and in the upheaval zones adjacent to them. They concluded that:

1. In the initial stages of trafficking the increase of irreversible deformation below the tires is distinctly greater than the increase in the upheaval zones. Therefore, in the initial phase, traffic compaction or densification is the primary mechanism of rut development.
2. After the initial stage, the volume decrease below the tires is approximately equal to the volume increase in the adjacent upheaval zones. This indicates that most of the compaction under traffic is completed and further rutting is caused essentially by shear deformation, i.e., distortion without volume change. Thus, shear deformation is considered to be the primary mechanism of rutting for the greater part of the lifetime of the pavement.

2.2 Causes of Rutting in Flexible Pavements

Generally there are three causes of rutting in asphalt pavements: accumulation of permanent deformation in the asphalt surfacing layer, permanent deformation of subgrade, and wear of pavements caused by studded tires. In the past subgrade deformation was considered to be the primary cause of rutting and many pavement design methods applied a limiting criteria on vertical strain at the subgrade level. However recent research indicates that most of the rutting occurs in the upper part of the asphalt surfacing layer. These three causes of rutting can act in combination, i.e., the rutting could be the sum of permanent deformation in all layers and wear from studded tires.

2.2.1 Rutting Caused by Weak Asphalt Mixture

Rutting resulting from accumulation of permanent deformation in the asphalt layer is now considered to be the principal component of flexible pavement rutting.This is because of the increase in truck tire pressures and axle loads, which puts asphalt mixtures nearest the pavement surface under increasingly high stresses.

Brown and Cross [2] reported on an extensive national study of rutting in hot mix asphalt pavements in United States. The study was initiated in 1987 to evaluate pavements from all areas of the United states encompassing various climatic regions, containing aggregates of differing origins and angularity, encompassing different specifying agencies and construction practices and a large sample size to make the study results national in scope.The study involved collection of pavement core samples for material characterization, measurement of rut depth and layer thicknesses, and investigation to determine the location of rutting. The conclusion from this study regarding the location of rutting was that the majority of rutting was occurring in the top 3 to 4 inches (75 to 100 mm) of the asphalt concrete layers. They found that the rutting in the subgrade was generally very small.

In Europe, a survey was conducted, under the COST 333 program, to determine the most common type of pavement deterioration [3]. Accordingly, countries were asked to rate the most common forms of deterioration observed on their roads using a rising scale of increasing importance from 0 to 5: where 0 indicates that it is not observed; and 5 it is a major determinant of pavement performance. Figure 2.1 shows the result of the survey. The figure clearly shows that rutting originating in bituminous layers is the most common form of pavement deterioration on European roads.

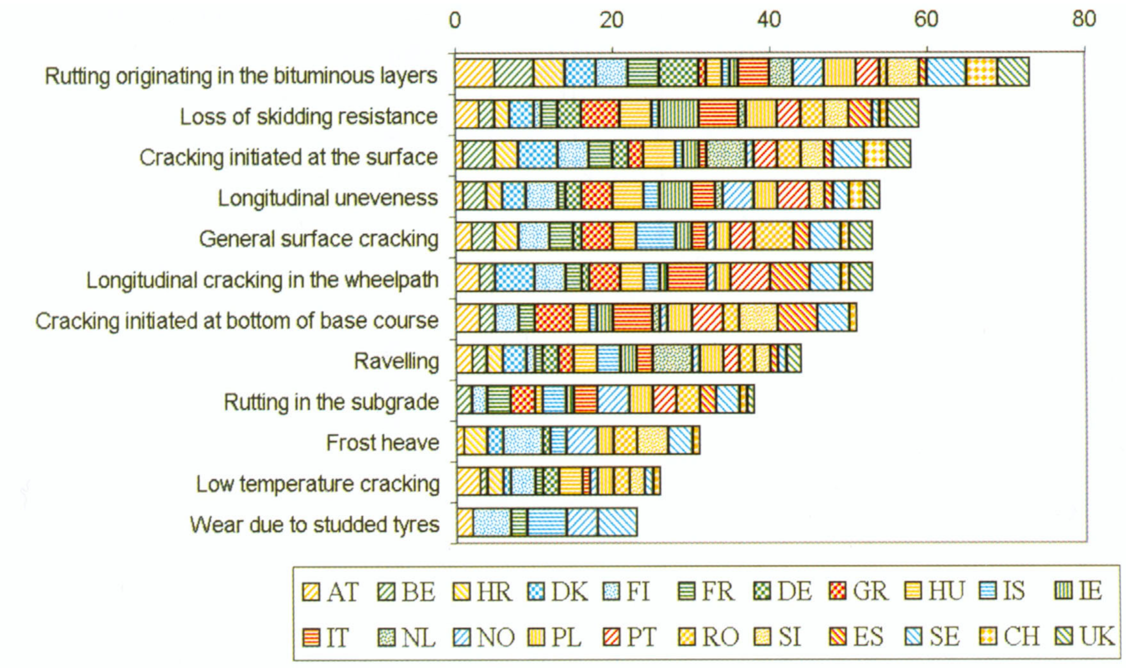


Figure 2.1 Rating of observed deterioration[3]

It is thus abundantly clear that rutting caused by accumulation of permanent deformation in asphalt layers is the primary cause of flexible pavement deterioration. To reduce this form of deterioration it is necessary to pay more attention to the selection of materials and mix design.To be able to design mixtures that have adequate resistance to rutting, the effect of mixtures' volumetric composition and properties of the component materials on their permanent deformation response must be clearly understood. Further, there should be a simple measure of resistance of mixtures to rutting that can be used at mixture design stage to enable evaluation and selection of rut resistant mixtures. This issues are the main areas focus of this thesis work.

Rutting in asphalt layers is caused by an asphalt mixture that is too low in shear strength to resist the repeated heavy loads to which it is subjected. Asphalt pavement rutting from weak asphalt mixtures is a high temperature phenomenon, i.e., it most often occurs during the summer when high pavement temperatures are evident. Figure 2.2 illustrates rutting caused by weak asphalt mixture.

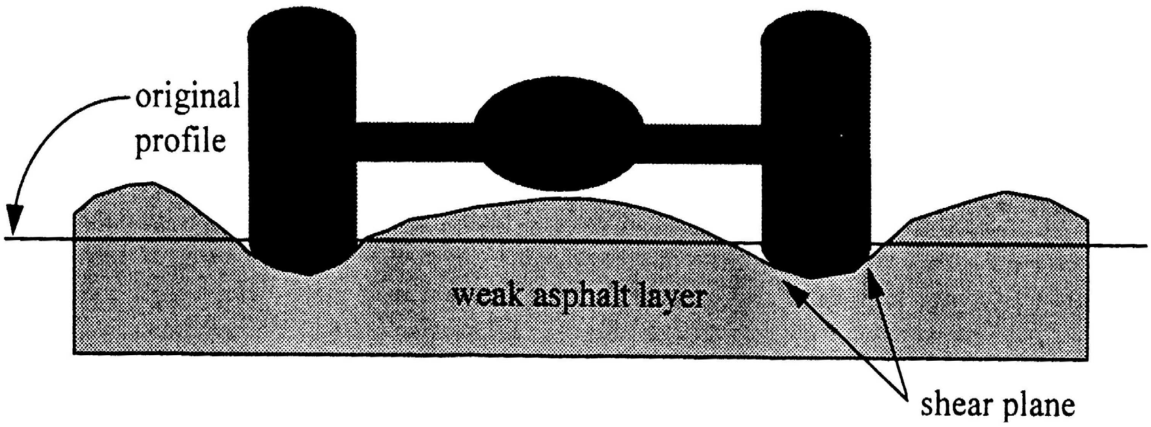


Figure 2.2 Rutting caused by weak asphalt layer[4]

As mentioned before, the permanent deformation in asphalt concrete consists of densification and shear deformation. Shear deformation occurs with no change in volume,i.e., it is distortional. Asphalt concrete may also dilate or increase in volume under load. Deformation involving dilatancy is also referred to as shear flow or plastic flow in some literatures. Such deformation can lead to debonding at the binder aggregate interface and deterioration of the pavement. Figure 2.3 illustrates the mechanisms of rutting in asphalt layers.

Thus in evaluating mixtures for their rutting resistance, it is necessary to pay more attention to their shearing and dilatant behaviour. Traditionally, the evaluation of rutting resistance of asphalt concrete mixtures is based on axial (one dimensional) permanent strain. This approach fails to capture the shearing response of the material, which may manifest itself in the form of relatively large lateral deformation. These issues will be discussed in more detail in chapter 7.

2.2.2 Rutting Caused by Weak Subgrade

Rutting can be caused by too much repeated load applied to subgrade, subbase or base below the asphalt layer. In many cases this is due to insufficient depth of cover on the subgrade resulting from too thin an asphalt section to reduce the stress from applied loads to tolerable level. Thus this type of rutting is considered to be more of a structural problem than a materials problem and is often referred to as structural rutting. Intrusion of moisture can also be the cause for weakening of the subgrade. In this type of rutting, the accumulated permanent deformation occurs in the subgrade.Figure 2.4 illustrates rutting from weak subgrade.

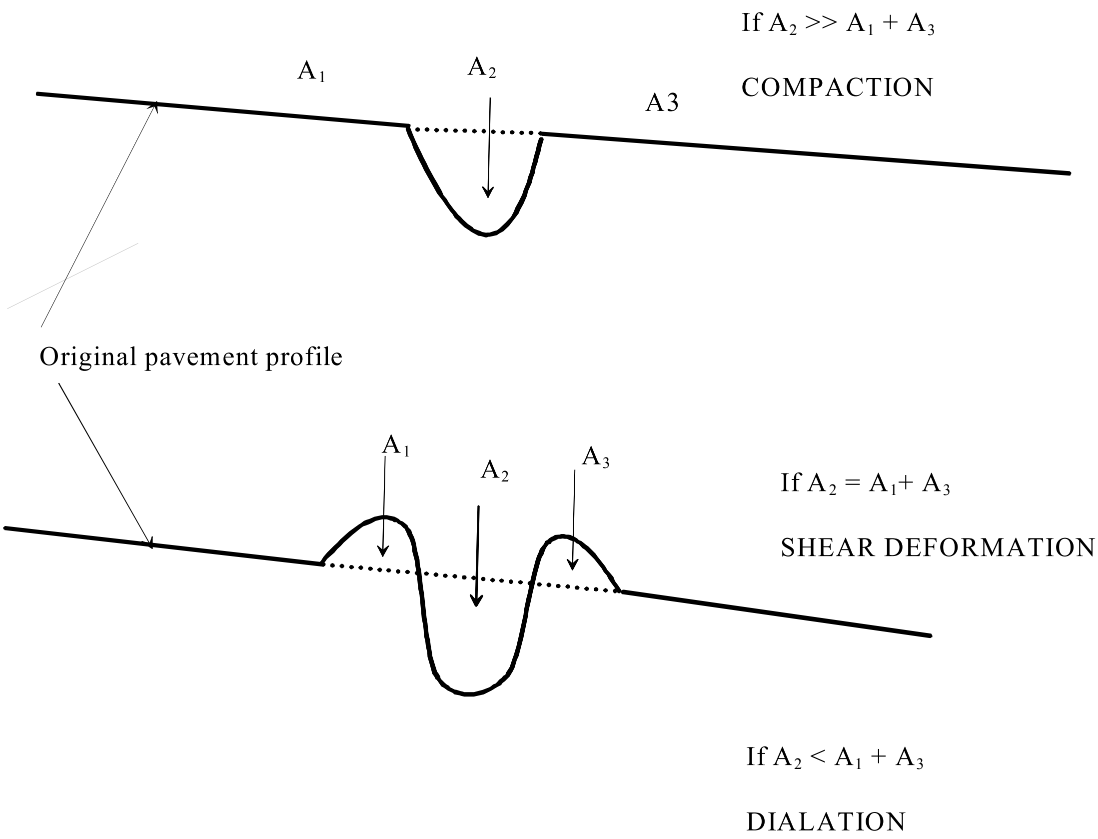


Figure 2.3 Illustration of the rutting mechanism

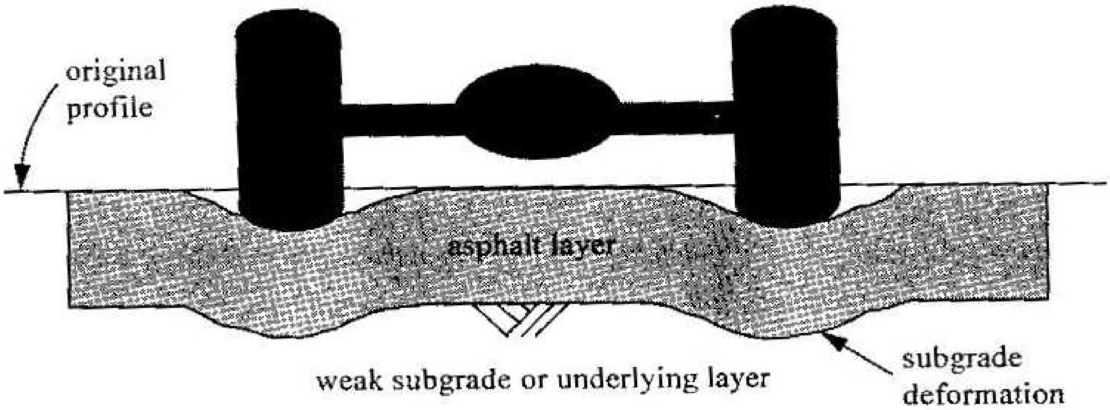


Figure 2.4 Rutting from weak subgrade[4]

2.2.3 Rutting Caused by Pavement Wear

The studded tires, used in Nordic countries, cause significant wear of the pavements, which results in longitudinal depression in the wheel path. The studded tire wear is estimated to cost the Norwegian Public Roads Administration about 500 million NOK every year, for instance.Because of this, wear resistance mixtures, which are usually of high binder content and low void content are specified for high volume roads. But this kind of mixtures are also susceptible to shear deformation as will be discussed in chapter 7. Therefore the observed rutting in the field would most probably be the combined effect of wear and permanent deformation. Figure 2.5 shows rutting caused by studded tire wear as measured on a Norwegian road.

Figure 2.5 Rutting caused mainly by studded tire wear

2.3 Rutting Consideration in Pavement Design

In the past, mainly empirical methods were used to design pavements.These methods do not consider pavement distress explicitly. In recent years, the more rational mechanistic - empirical methods have been developed and are being implemented. Generally two procedures have been used in the mechanistic - empirical methods to limit rutting: one to limit the vertical compressive strain on top of the subgrade and the other to limit the total accumulated permanent deformation on the pavement surface based on the permanent deformation properties of each individual layer. Given that with increased tire pressures most of rutting occurs in the asphalt surfacing layer rather than the subgrade, the first approach appears inappropriate for consideration of rutting in pavement structural design.

In the second approach, the permanent deformation properties of each individual layer is taken into account. This requires testing and characterization of the materials used in the pavement structure. It also requires the calculation of stresses at selected points in each layer. The permanent deformation of each layer is then calculated and summed up to find the total permanent deformation. This approach is rational and it allows the explicit consideration of permanent deformation properties of materials in each layer.

2.4 Rutting Consideration in Mixture Design

The purpose of mix design is to determine the proportions of aggregate and binder that would produce a mix, which is economical and has the following desirable properties:

- sufficient binder to ensure durability
- sufficient voids in mineral aggregate, so as to minimize post construction compaction without loss of stability and without causing bleeding, and to minimize harmful effects of air and water.
- sufficient workability to permit laying of the mix without risk of segregation, and
- sufficient performance characteristics over the service life of the pavement

Luminari and Fidato published a state of the art report on mix design [5], in which they classified the various asphalt concrete mix design methods into six categories: recipe, empirical, analytical, volumetric, performance - related, and performance - based methods. The recipe method is based on the experience of traditional mixes of known composition, which over long period of time and given site, traffic and environmental conditions, have performed successfully. The recipe defines the bituminous mixture in terms of the aggregate gradation, binder grade, mix composition, layer thickness and mix characteristics during manufacture, laying and compaction. No material characterization tests are involved and hence this method does not allow the consideration of rutting resistance or the resistance to any other form of pavement distress.

Empirical mix design methods involve the selection of the binder content based on optimization of several variables, taking into account the specification limits set based on prior experience, including those determined by void analysis. The most commonly used and best known example of the empirical mix design method is the Marshal method. The variables optimised in empirical mix design methods are not direct measure of performance. For instance the Marshal stability is a surrogate measure of mixture's shear strength. The Marshal flow is specified to limit permanent deformation. But the Marshal method has several shortcomings including:

1. the impact hammer used to prepare specimens in this method does not simulate the compaction that occurs in pavements, and
2. it is not suited to the present day traffic conditions as evidenced by the steady increase in rutting problems in recent years with mixes prepared using this method.

In the volumetric mix design method, design binder content and aggregate gradation are chosen by analysing the proportional volume of air voids, binder and aggregate for mixtures which have been compacted using a compaction procedure that is assumed to reproduce, in the laboratory, the in situ compaction process. No tests are conducted on the mechanical properties of the mixtures. Volumetric mix design method is expected to produce mixtures that would perform satisfactorily under low traffic conditions. This method has to be supplemented by some sort of mechanical tests if it is to be used for design of mixtures that would be subjected to medium or heavy traffic conditions. A prime example of the volumetric mix design method is the level 1 of the Superpave mix design system developed in the US under the Strategic Highway Research Program (SHRP). In the Superpave method, specimens are compacted using the SHRP Gyratory shear compactor. This method also utilizes performance - based bitumen specifications and empirical performance - related specifications for the aggregates.

In the performance - related mix design methods, mixes that meet established volumetric criteria are compacted and tested to measure or estimate mix properties, which are related to pavement performance. The most satisfactory mixture is then selected based on these additional performance related criteria. The French mix design method and the mix design method developed at University of Nottingham in Britain are examples of the performance - related mix design methods. In the French mix design method, the resistance to permanent deformation is specified as the maximum rut depth resulting from the wheel tracking test. However, the wheel tracking test itself is empirical in nature and has shortcomings as discussed in chapter 5. Stiffness modulus (measured by direct tensile test) and fatigue strength (measured in a constant deformation fatigue bending test) are also measured and specified.

In the University of Nottingham method, the Nottingham Asphalt Tester (NAT) is used to measure the stiffness modulus, fatigue resistance, and the resistance to permanent deformation for mixtures that meet certain criteria with regard to void content, voids in mineral aggregate and and voids filled with binder. The resistance to permanent deformation is measured using repeated load test and the axial creep test, while the resistance to fatigue and the stiffness modulus are measured using repeated load indirect tension test. The problem with NAT is that the magnitude of the confining stress that can be obtained in repeated load permanent deformation test is considered to be low as compared to the confining stress expected in the field. Permanent deformation response of asphalt concrete mixtures in repeated load testing is found to be significantly influenced by the confining stress as discussed in chapter 6 in this study. Creep rate (rutting rate) is used as a measure of resistance to permanent deformation. There are difficulties that may be encountered in using the creep rate as a measure of resistance and accelerated field tests have shown that it does not correlate well with rutting observed in the field. These issues are also discussed in chapter 6.

In performance - based mix design methods, a selected mix is subjected to performance - based tests and to an integrated system of assessment to determine how the mix will perform over a period of time. Specimens are compacted and tested to determine their fundamental properties that are proven to be related to performance and that can be used as an input into material models. Different models regarding material properties, environmental effects, pavement response and distress are applied to predict pavement performance, providing realistic estimates of the evolution of different kinds of distress over the working life of the pavement.

Under SHRP, proposals for performance - based mix design methodology were presented as levels 2 and 3 of the Superpave mix design system to suite intermediate and high traffic levels. However, these two levels are currently considered to be not feasible and therefore are not being implemented because of problems encountered with regard to prediction models [5].But the approach and the analysis framework established under this program appears to be valid and deserve to be pursued further. If implemented, the performance - based mix design methods would allow the permanent deformation as well as other distresses to be taken into account in more fundamental and scientific manner. It also provides a framework for connecting the mixture design to pavement structural design and performance prediction.

In summary, with the exception of the performance - related mix design methods, most of the mix design methods currently in use do not properly evaluate the rutting resistance of asphalt concrete mixtures. It appears that parameters that can be used for evaluation of the resistance to rutting and that can be correlated to actual performance of the mixture is yet to be developed. This issue forms one of the areas of focus in this study and is discussed in chapter 7.

CHAPTER 3
EFFECTS OF COMPOSITION AND PROPERTIES OF COMPONENT MATERIALS ON PERMANENT DEFORMATION OF ASPHALT CONCRETE MIXTURES

Asphalt concrete consists of asphalt binder, aggregates and air voids. The properties of asphalt concrete depends on the quality of its components, the construction process, and the mix design proportions. In service, asphalt concrete must provide a stable, safe, and durable road surface. Stability of the asphalt concrete depends on strength and flexibility of the mixture and the degree of compaction during placing. The strength must be sufficient to carry the load without shear deformation occurring between particles. Rutting, which is a dominant mode of failure in asphalt pavements, occurs as a result of the accumulation of permanent deformation in pavement layers.

Several factors related to the characteristics of the component materials of an asphalt mixture are known to affect the resistance to rutting to a varying degree. In order to be able to produce asphalt mixtures that have adequate resistance to rutting, it is necessary to know the properties of the component materials that influence the resistance to rutting of the mixture. By carefully choosing the types and proportions of component materials that have desirable properties with regard to rutting, it might be possible to minimize rutting in flexible pavements. The effect of the properties of each of the components of asphalt mixture, i.e., binder and aggregates, and their proportions (air void content and binder content) on permanent deformation properties will be reviewed in this chapter after brief discussion on volumetrics of asphalt concrete mixtures.

3.1 Asphalt Concrete Volumetrics

Asphalt concrete mixtures contain three components; air voids, mineral aggregates, and bituminous binder.The primary volumetric parameters are those relating directly to the relative volumetric proportions of these components. Volumetric properties of a compacted asphalt mixture are illustrated in Figure 3.1 and definitions of the volumetric parameters are as follows:

- **Void content (V_a)**- is the percent by volume of air between the coated aggregate particles in a compacted asphalt mixture.
- **Binder content (P_b)**- is the percent by weight of asphalt binder in the total mixture, including asphalt binder and aggregates.
- **Voids in mineral aggregates (VMA)**- is the volume of compacted paving mix not occupied by the aggregates when the volume of the aggregates is calculated based on their bulk specific gravity.
- **Absorbed asphalt volume (V_{ba})**- is the volume of asphalt binder absorbed in to the aggregates.
- **Effective asphalt volume (V_{beff})**- is the volume of asphalt binder not absorbed into the aggregates
- **Voids filled with asphalt (VFA)**- is the percentage of voids in mineral aggregate filled with asphalt binder.

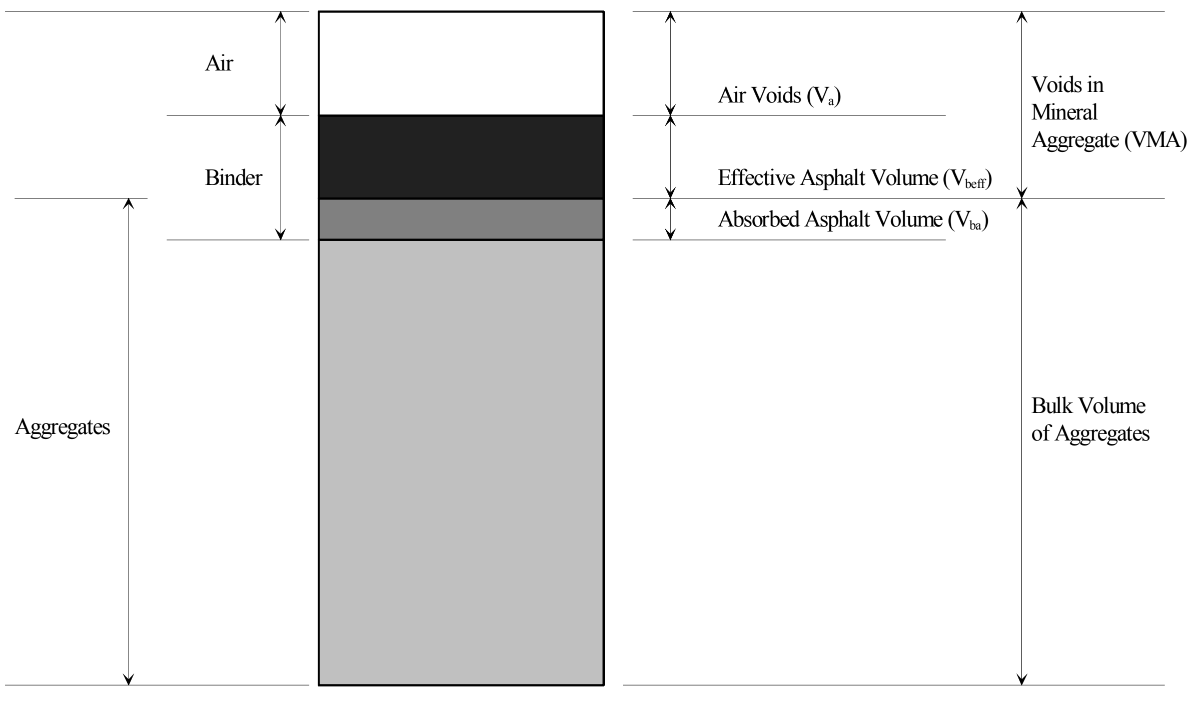


Figure 3.1 Volumetric properties of compacted asphalt mixture

The following relationships are used to compute some of the volumetric parameters. Air voids, (V_a), expressed as a percent of total volume is given by:

$$V_a = 100\left(\frac{G_{mm} - G_{mb}}{G_{mm}}\right) \qquad 3.1$$

Where:

G_{mm} = maximum specific gravity of the mixture, and

G_{mb} = bulk specific gravity of compacted mixture.

Voids in mineral aggregate (VMA) as a percent of bulk volume can be calculated using equation 3.2:

$$VMA = 100 - \left(\frac{G_{mb}P_s}{G_{sb}}\right) \qquad 3.2$$

Where:

P_s = aggregate as percent of total weight of mixture, and

G_{sb} = bulk specific gravity of aggregates.

Voids filled with asphalt can be expressed as:

$$VFA = \left(\frac{VMA - V_a}{VMA}\right)100 \qquad 3.3$$

3.1.1 Effect of Volumetric Composition on Performance of Asphalt Mixtures

It is generally recognized that the volumetric composition of mixtures greatly influence their performance, i.e., their resistance to distresses. A mixture with good performance is one, which is resistant to various load-related and thermally induced distresses such as rutting, fatigue cracking and low temperature cracking. A well performing mixture should also have resistance to other types of distresses such as roughness, ravelling, shoving, corrugation and formation of potholes.

The level of compaction as expressed by void content and the binder content are known to affect the resistance of mixtures to these distresses in various ways. The available knowledge on the effect of the volumetric composition on performance is, however, generally qualitative. But, some attempts have been made to develop predictive equations for some properties of asphalt mixture such as the stiffness modulus and dynamic modulus, which include some of the volumetric parameters as a variable. The stiffness modulus has been used in prediction of rutting in some pavement design methods, most notably, the Shell Oil method. Shell's researchers developed predictive equations and nomographs for calculation of mixture stiffness from binder stiffness and volumetric composition.

The main assumption in the Shell Oil predictive method for mixture stiffness is that the stiffness of the mix is primarily governed by the stiffness of the binder. Nomographs were developed for evaluation of the binder stiffness, S_b. The mixture stiffness modulus (S_m) is determined based on the stiffness of the binder, the percent volume of binder, and the percent volume of mineral aggregates using either nomograph or equations listed below.

For binder stiffness $5x10^6 < S_b$ (N/m^2) $< 10^9$:

$$\log S_m = \frac{\beta_4 + \beta_3}{2}(\log S_b - 8) + \frac{\beta_4 + \beta_3}{2}|\log S_b - 8| + \beta_2 \qquad 3.4$$

For binder stiffness of $10^9 < S_b$ (N/m^2) $< 3x10^9$:

$$\log S_m = \beta_2 + \beta_4 + 2.0959(\beta_1 - \beta_2 - \beta_4)(\log S_b - 9) \qquad 3.5$$

$$\beta_1 = 10.82 - \frac{1.342(100 - v_g)}{v_g + v_b} \qquad 3.6$$

$$\beta_2 = 8.0 + 0.00568 v_g + 0.0002135 v_g^2 \qquad 3.7$$

$$\beta_3 = 0.6\log\left(\frac{1.37 v_b^2 - 1}{1.33 v_b - 1}\right) \qquad 3.8$$

$$\beta_4 = 0.7582(\beta_1 - \beta_2) \qquad 3.9$$

Where:

S_m = stiffness modulus of the mix,

S_b = stiffness moduls of the binder,

v_b = percent volume of binder, and

v_g = percent volume of aggregate.

A comprehensive predictive equation for dynamic modulus of asphalt mixtures was developed by Witczak and co-workers at the University of Maryland in the US. The current form of the predictive equation (equation 3.10) is reported to be based on over 2800 dynamic modulus measurements on about 200 different asphalt mixtures tested in the laboratories of the Asphalt Institute, the University of Maryland, and the US Federal Highway Administration[6]. The predictive equation is expressed as follows:

$$\log|E^*| = -1.249937 + 0.029232(p_{200}) - 0.001767(p_{200})^2 - 0.00284(p_4) - 0.058097(V_a) \qquad 3.10$$

$$+\frac{-0.802208(V_{beff})}{V_{beff}+V_a}+\frac{3.871977-0.0021(p_4)+0.003958(p_{38})-0.000017(p_{38})^2+0.00547(p_{34})}{1+e^{(-0.603313-0.313351\log(f)-0.39353\log\eta)}}$$

Where:

$|E^*|$ = dynamic modulus, 10^5 psi

η = bitumen viscosity, 10^6 Poise,

f = loading frequency, Hz,

V_a = air void content,%,

V_{beff} = effective bitumen content,% by volume,

p_{34} = cumulative % retained on 19 mm sieve,

p_{38} = cumulative % retained on 9.5 mm sieve,

p_4 = cumulative % retained on 4.76 mm sieve, and

p_{200} = % passing 0.075 mm sieve.

The dynamic modulus is being considered as a measure of performance of asphalt concrete mixtures. In particular, it is reported to correlate well with measured rutting [6]. The above predictive equations for stiffness modulus and dynamic modulus represent some of the attempts made to quantify the effect of volumetric composition on properties and performance of asphalt mixtures and they illustrate the importance of the volumetric composition. Some attempts have also been made to establish a direct relationship between the volumetric parameters and permanent deformation response of asphalt mixtures.These relationships and predictive equations are reviewed in chapter4. In the following sections review of the effect of the composition and the properties of the component materials on permanent deformation response of asphalt mixtures is provided.

3.2 Effect of Aggregate Properties

Aggregate represents a major portion of asphalt concrete and it is responsible for the strength and toughness of the material. The physical properties of aggregates significantly affect the performance of asphalt concrete pavement in service. In order to design asphalt concrete mixes properly, one has to understand the basic properties of aggregates, interactions between aggregates and binders, and the effect of different aggregate characteristics on the performance of asphalt concrete mixtures. The Strategic Highway Research Program (SHRP) divided aggregate properties into two parts: consensus properties and source properties. Consensus properties include aggregate gradation, coarse aggregate angularity, fine aggregate angularity, flat and elongated particles and clay content. Source properties include toughness, soundness, and deleterious materials. All of these properties affect the performance of asphalt concrete in one way or another, but many authors have singled out aggregate gradation, aggregate angularity, and filler content as having significant influence on the rutting resistance of asphalt concrete. It has been mentioned that rutting occurs in mixtures with low shear resistance or strength compared to the repeated stress it is subjected to. The shear strength of frictional and cohesive materials, including asphalt mixtures, has been expressed using the following equation:

$$\tau = c + \sigma \tan \varphi \qquad 3.11$$

Where:

τ = Shear strength

c = Cohesion

σ = Normal stress

φ = Angle of internal friction

One can get an insight into the effect of aggregate properties on shear strength of mixtures by considering their effect on c and φ in the above equation. For a given level of stress, temperature and rate of loading, the shear strength depends on the cohesion c and angle of internal friction φ. The cohesion c is affected by the viscosity of asphalt binder and the proportion of fines.The angle of internal friction is obtained from aggregate interlocking. Higher values of φ are developed if the aggregate is rough textured, angular and well graded. The mechanical interlock of the aggregate particles thus plays a key role in shearing resistance.The binder content is also known to affect φ because it changes the degree of mechanical interlock between the particles, i.e., the higher the proportion of binder in the mix, the further apart the aggregate particles are spread.

3.2.1 Aggregate Gradation

Gradation refers to the particle size distribution of aggregates. Gradation is usually described in terms of particle size distribution curve (also known as gradation curve or gradation chart), which is determined from sieve analysis. However, it is difficult to find a single parameter that quantitatively characterize all aspects of the particle size distribution curve. Therefore, parameters such as the area enclosed between the particle size distribution curve and the Fuller's (maximum density) curve and the median size (d_{50})are used to characterize the gradation curve. Fuller's curve describes the ideal particle size distribution curve of a theoretical material consisting of spheres for which, for a given maximum grain size (d_{max}), the highest packing or the maximum density can be achieved. Equation 3.12 describes Fuller's curve.The median size indicates the relative coarseness or fineness of aggregate materials.

$$p = \left(\frac{d}{d_{max}}\right)^{0.5} \qquad 3.12$$

Where p is percent passing sieve size, d.

As mentioned before, the shear strength and hence the resistance to permanent deformation of asphalt mixtures depends on the mechanical interlock of the aggregate skeleton especially the stone structure of coarse aggregates. Loss of stability, which can lead to rutting, can in general occur when gradations containing excesses of certain size fractions are used. It is believed that, strength or resistance to shear failure, in pavements and other aggregate layers that carry loads is increased greatly if the mixture is dense graded. In dense graded aggregates, the larger particles are in contact with each other, developing frictional resistance to shearing failure, and tightly bound together due to interlocking effect of the smaller particles. Several authors have considered the effect of aggregate gradation on resistance to permanent deformation.

El-Basyouny and Mamlouk [7] evaluated the effect of aggregate gradation on the rutting potential of Superpave mixes. They prepared several mixtures with different aggregate gradations, performed creep test, and analysed the results using the VESYS-3AM (a viscoelastic multilayer program) to estimate the rut depth of the different mixtures. They concluded that both the aggregate gradation and aggregate nominal size affected the rut depth for specific pavement section as estimated by the VESYS-3AM software. Specifically they found out that mixtures prepared using aggregate gradation passing below the restricted zone (on the Superpave gradation chart) had better resistance to rutting as compared to those made from aggregates with gradation passing through or above the restricted zone. However, as the comparisons were made based on rut depth predicted by model rather than measured rut depth, the study is subject to limitations of the model.

Cross et al [8] evaluated two mixtures made from aggregates with different gradations. The aggregate for first mixture had an S-shaped gradation that stays below the maximum density line and passes below the restricted zone. The second mixture was made from aggregate with finer gradation that stayed above the maximum density line and above the restricted zone. Other mixtures with aggregate gradations 5 to 20% coarser, as measured on the 4.75 mm sieve, than the fine and coarse mixtures were also prepared and tested. The mixtures were evaluated for permanent deformation as well as other mechanical properties using Asphalt Pavement Analyser (Georgia rut tester). They concluded that the finer gradation had better resistance to permanent deformation, which apparently was in contradiction with conclusions of El-Basyouny and Mamlouk in the previous paragraph. Figure 3.2 shows measured rut depth at 8000 cycles for the mixtures evaluated in the study by Cross et al.

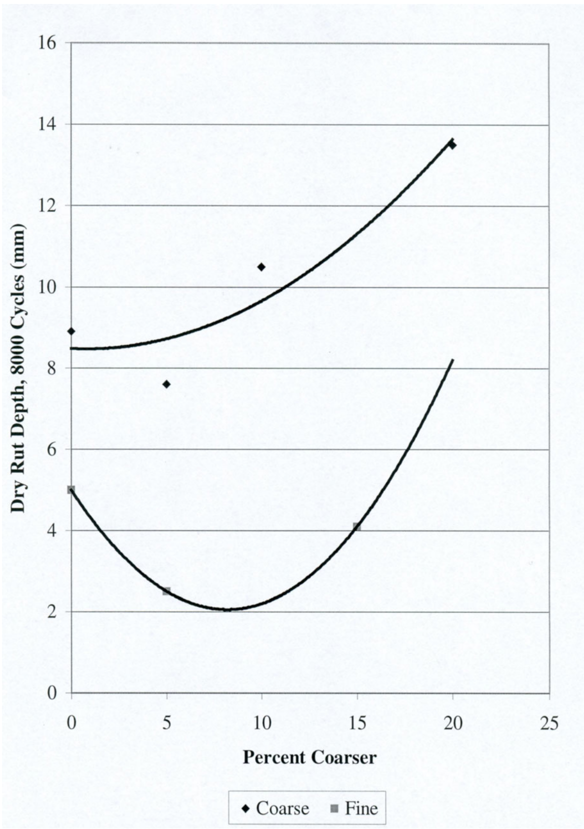


Figure 3.2 Maximum dry rut depth[8]

Carpenter and Enockson [9] studied 32 overlay projects in Illinois, USA, and concluded that majority of the rutting problem can be attributed to aggregate gradation. Oliver et al [10] carried out both field and laboratory study on several mixtures in Australia and concluded that, among other factors, aggregate gradation has a significant influence on rutting resistance. Dukatz [11] argued that gradation is a key factor in permanent deformation resistance and specifically he argued that bumps on the 0.45 power curve tend to give mixes that are tender, i.e., mixes that rut easily under traffic. However, Brown and Cross [12] argued that aggregate properties have little effect on rutting when voids are less than 2.5% based on their study, which involved laboratory tests on samples collected from field. Even when percentage of voids is greater than 2.5, Brown and Cross argued that, it is the fine aggregate angularity and not the gradation that has a significant influence on rutting resistance. Also Barksdale (cited in Ramsamooj et al, [13]) studied rutting of paving materials and found that permanent deformation in dense graded asphalt concrete was not sensitive to gradation of aggregates.

Thus researchers have come to different conclusions with regard to the effect of aggregate gradation on resistance to rutting of asphalt mixtures. However, most of the authors seem to agree that aggregate gradation has an influence on the rutting resistance of asphalt mixtures. But, since the studies were conducted with different methodology and experimental set up, conclusions and results can not be directly comparable.

3.2.2 Aggregate Angularity

Researchers at the Strategic Highway Research Program (SHRP) defined coarse aggregate angularity as the percent by weight of the aggregate particles larger than 4.75 mm with one or more fractured faces [14]. A fractured face was defined as angular, rough or broken surface of an aggregate particle created by crushing, by other artificial means or by nature. Fine aggregate angularity was defined as the percent of air voids present in loosely compacted aggregate that passes the 2.36 mm sieve. Aggregate particle shape and surface texture affect the strength of aggregate particles, the bond with asphalt binder, and the resistance to sliding of one particle over another. Particles with rough, fractured faces allow a better bond with the asphalt binder than do rounded smooth gravel particles. Rough faces on the aggregate particles also allow a higher friction strength to be developed if some load would tend to force one particle to slide over an adjacent particle.

One important issue regarding the shape and surface texture properties of aggregates is the question of how to quantify them. Cominsky et al [14] suggested the use of Pennsylvania Department of Transportation's Test Method No. 621, *Determining the Percentage of Crushed Fragments in Gravel* for measuring coarse aggregate angularity in Superpave mixture design method. The suggested method is based on fractured face count and therefore it is subjective. In Superpave mixture design method, fine aggregate angularity is measured on the fine aggregate portion of the blended aggregate by AASHTO Standard Method of Test TP 33 (ASTM C1252), *Uncompacted Void Content of Fine Aggregate (as influenced by Particle Shape, Surface Texture, and Grading).* In Europe the flakiness index is mostly used to quantify the aggregate shape. A version of the flakiness index called 'flisighetstallet' is used in Norway. Other authors including Li et al [15] and Perdomo et al [16] recommended the use of fractals to quantify aggregate angularity and surface texture.

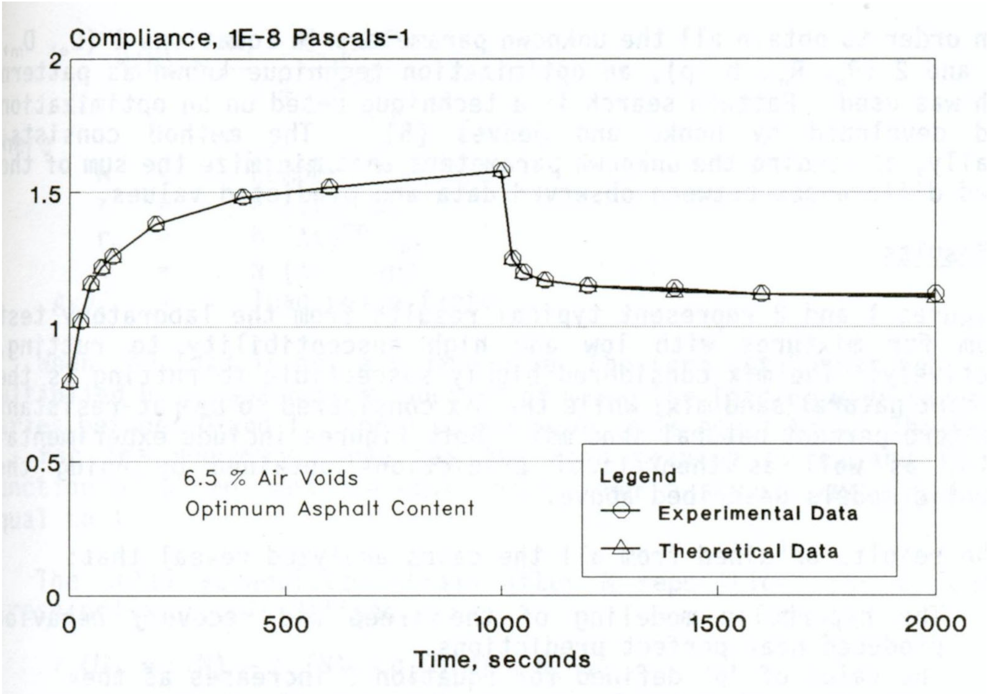
Fractals are relatively novel class of mathematical functions developed and popularized to describe the natural structures and shapes produced by a number of basic physical processes. Fractals contain new mathematical tools and image processing techniques that can be used to describe natural shape and structure that is irregular, rough, or fragmented. A fractal is a geometrical term used to describe an object whose shape is intermediate between topological ideals. The dimensions of Ecculidean geometry are given as the integers 0, 1, 2, and 3, corresponding to dots, lines, planes, and bodies respectively. However, this simple classification is not adequate for the very irregular shape of numerous natural geometrical objects, so a need to assign some intermediate dimension, called fractal dimension, to such objects. Fractals have been applied successfully in the fields of medicine, metallurgy, geology and material science. Li et al [15] argued that fractals provide an automatic, quantitative, and objective measurement technique for characterizing aggregate shape and a potential to be implemented in automatic quality control of aggregates.

Several researchers have attempted to evaluate the effect of aggregate angularity on rutting resistance of asphalt mixtures. Most agree that aggregate angularity has a significant influence on rutting resistance of mixtures. An emphasis is placed by many authors on the angularity of fine aggregates (sand- size particles) as a major factor affecting the resistance to plastic deformation of asphalt mixtures.

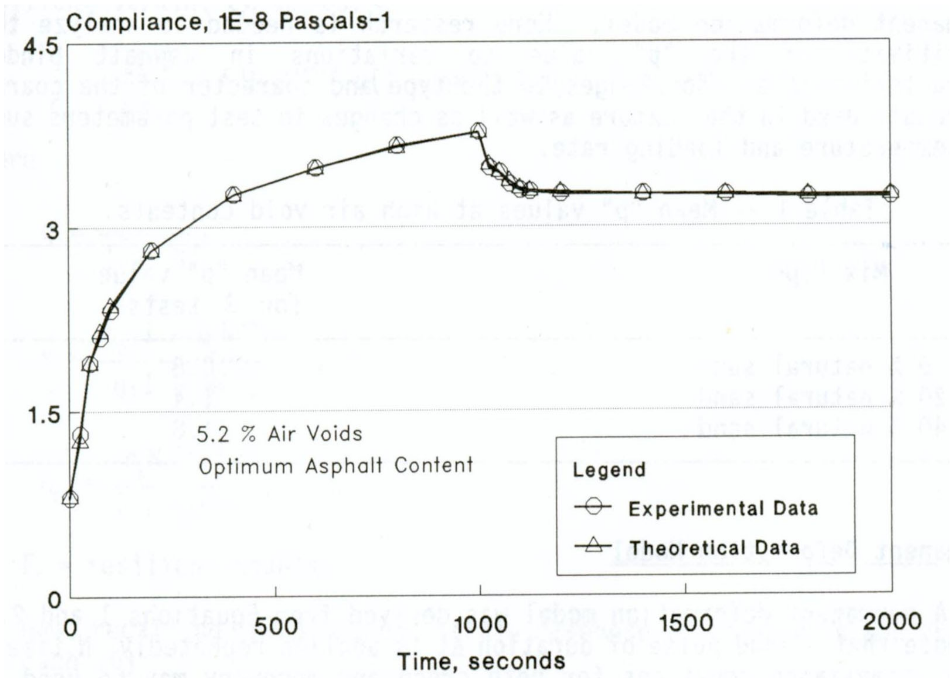
Perdomo et al [16] investigated the influence on resistance to plastic deformation of replacing rounded, smooth, sand-sized aggregates with rough, angular, porous particles while keeping the other aggregates and total gradation unchanged. They used fractal dimension analysis as a method of quantifying aggregate angularity and surface texture. Their laboratory investigation showed that mixes containing 40% natural sand had significantly lower resistance to plastic deformation than those mixes prepared without using natural sand (i.e, using angular and rough particles). Figure 3.3 shows the result of their study. They further argued that fractal dimension analysis is a practical method to quantify aggregate angularity and surface texture.

Sanders And Dukatz [17] evaluated percent fracture of hot mix asphalt gravels in Indiana, USA. The study involved evaluation of aggregate gradation and angularity for four pavement sections, which exhibited various levels of rutting, and an extensive literature review on the effect of percent fracture on rutting resistance. Many of the authors quoted in their literature review indicated that it is the angularity of the fine aggregate (sand particles) that has significant effect on rutting resistance than the angularity of the coarse aggregate. In fact many argued that requiring more than 50% fractured faces for coarse aggregate only increases cost without substantial improvement in rutting resistance. Sanders and Dukatz found out that the pavement section, which prematurely rutted while carrying the lowest traffic and being in service for shortest period compared to other sections in the study, had a natural sand hump in its aggregate gradation. They concluded that gradation and fine aggregate angularity appeared to be the most important factors influencing the rutting resistance.

Kobayashi et al [18] conducted an extensive study of the effect of fine aggregate shapes on characteristics of asphalt mixtures in Japan. Image processing method was employed to quantify the angularity of fine aggregates and several parameters were defined to describe the shape of fine aggregates. The study also proposed a simple method, known as dry viscosity test and involving measurement of the time during which 500 gm of single-grained fine aggregate run through a funnel, for the purpose of quantifying aggregate angularity. The conclusion of the study was that the use of angular fine aggregate can significantly enhance performance of pavements with regard to rutting.



(a)



(b)

Figure 3.3 Effect of fine aggregate angularity on rutting resistance[16]

Kim et al [19] studied the effect of aggregate type (angularity) and gradation on permanent deformation of asphalt concrete and concluded that aggregate angularity has a significant effect on permanent deformation resistance of asphalt concrete, indicating better performance from mixture comprised of aggregates with rough surface texture and angular shape. Dukatz[11] studied aggregate properties in relation to pavement performance and argued that angular particles greatly reduce the deformation under load.

Thus, available evidence clearly indicate that aggregate angularity is an important factor to be considered in selecting materials for rut resistant mixtures. It has been mentioned before that the shearing strength of asphalt mixtures greatly depend on the degree of mechanical interlock of the aggregate structure and friction that develops at contact points. Obviously, the aggregate angularity plays an important role in providing a strong aggregate interlock. However, aggregate angularity is just one factor among many that affect rutting resistance of asphalt mixtures. Thus, use of angular aggregates, by itself, may not produce a rut resistant mixture and one needs to consider all material properties that affect the resistance to rutting.

3.2.3 Mineral Fillers

Fillers are usually defined as material passing the # 200 (0.075mm) sieve. Fillers are added to paving mixtures to impart greater stability and strength. Two theories have been developed in an attempt to explain the stabilizing effect of fillers. According to the first one, fillers serve to fill the voids between aggregate particles, there by increasing the density and strength of the compacted mixture. The second theory presumes that the fine particles of the filler become suspended in the asphaltic binder forming a mastic. The suspended filler particles absorb binder components, there by increasing the viscosity of the binder and, consequently, the toughness of the mixes. The later theory have been supported by a research work conducted at the Danish National Road Laboratory as part of SHRP Idea Project, SHRP AIIR-13, Microscopic Analysis of Asphalt Aggregate Mixtures Related to Performance [20]. However, it is believed that fillers may play both roles simultaneously.

At higher field temperatures, where the deformation behaviour of asphalt pavement becomes critical, the highest possible viscosity of the filler-bitumen mastic is desirable as this will have a favourable influence up on the deformation resistance of the pavement. It has been indicated [21, 22] that at a given temperature an increase in viscosity can be achieved either with more filler or with the use of effective filler. However, there is a limit to the amount of filler that one can add to mixtures as high proportion of filler requires more binder to cover the extra surface area, may create problems in achieving the required compaction, and may aggravate aging and cracking.

Several researchers have shown that different fillers will stiffen an asphalt binder differently [21, 22, 24]. Changes in penetration, viscosity, ductility, and softening point temperatures have been used to show the stiffening potential of fillers. Two of the more commonly used methods to express this stiffening potential are:1) a stiffening ratio using kinematic viscosity of a mastic and neat asphalt binder, and 2) the increase in the ring and ball softening point temperature due to the addition of fillers

Anderson et al [20] studied rheological properties of mineral filler-asphalt mastics in relation to pavement performance. The mastics were prepared using four types of SHRP binder and two filler types (calcite and quartz). The rheological measurements were made using techniques developed under the SHRP research and were used to construct rheological master curves. The rheological measurement involved measurements of dynamic shear modulus, phase angle and creep properties. Anderson et al observed that the relative stiffening effect of fillers depends on the type of binder. They concluded that fillers are expected to significantly improve rutting resistance as a result of increasing moduli and this effect is expected to be asphalt-filler specific.However, the authors did not specify whether this effect also depend on the proportion of the filler in the mix.

Al Suhaibani et al [21] conducted a research on the effect of filler type and content on properties of asphalt concrete mixes. Three filler types; limestone dust, hydrated lime, and protland cement were considered. The effect of filler type and content on rutting potential of asphalt concrete was investigated using wheel tracking test.Figure 3.4 is a plot of rut depth after 10000 cycles in wheel track test against lime stone dust content. The conclusions from this study with regard to rutting were:

- rutting in asphalt concrete is highly dependent on the softening point of the mastic, i.e., binder plus filler, which in turn depends on the type and proportion of filler,
- rutting is highly affected by the amount of hydrated lime but not by the portland cement, and
- mixes made with limestone dust exhibit lower rut depth than those containing either portland cement or hydrated lime.

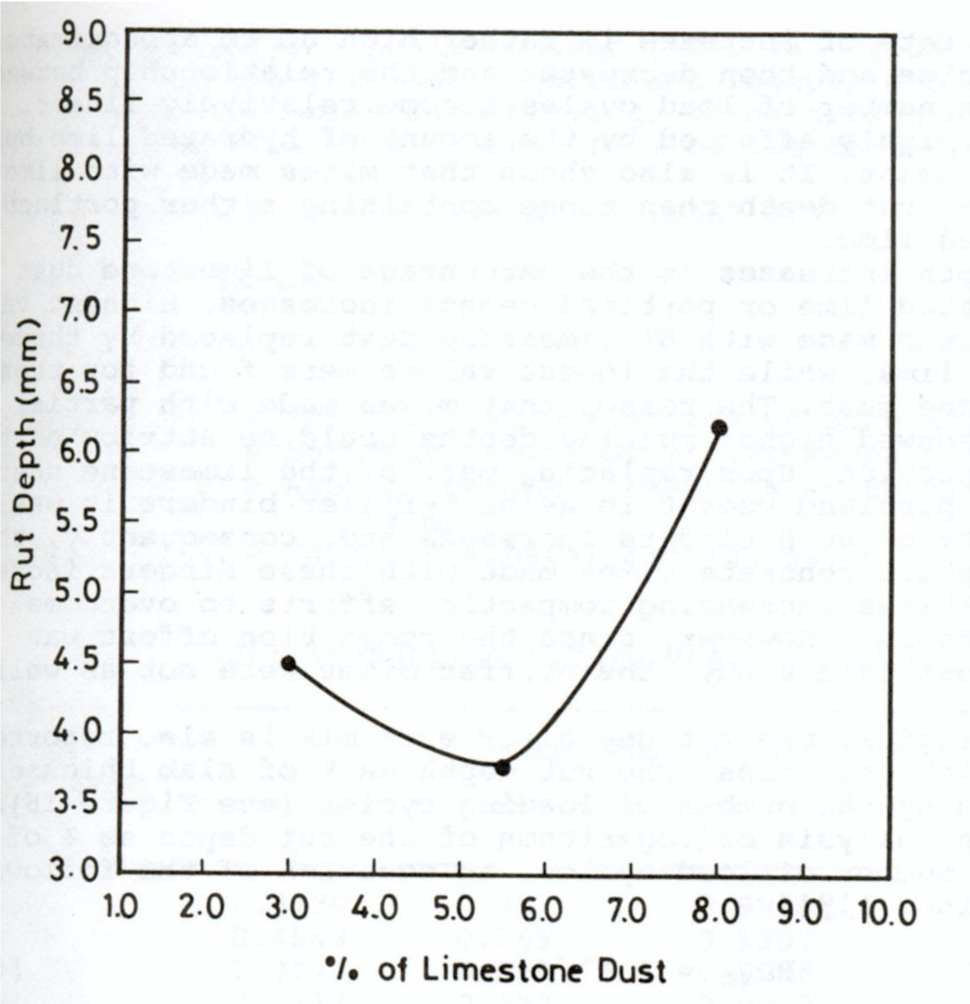


Figure 3.4 Effect of limestone dust content on rut depth[21]

Mohammad and Gokman [23] studied the performance of hot asphalt mixes with hydrated lime filler. The study involved comparative evaluation of conventional asphalt concrete mixture and one modified with hydrated lime. Hamburg wheel track testing device was used to evaluate the resistance of the mixtures to rutting and striping. The authors pointed out that the larger size fraction in the hydrated lime performs as a filler and increases the stiffness of the bituminous mixture while the smaller size fraction increases binder film thickness, enhancing viscosity of the binder, and improving the binder cohesion. The study revealed that mixes with hydrated lime showed improved performance compared to mixes with no lime.

Kavussi and Hicks [25] made an extensive study of properties of bituminous mixtures containing different fillers. Limestone, quartz, fly ash, and kaolin fillers with different physical properties were evaluated. The study involved both the rheological evaluation of the bitumen-filler mastic and an evaluation of the mechanical properties of mixtures, including a toughness parameter (area under the stress-strain curve in flexural testing) and Marshal parameters. The authors concluded that:

1. Viscosity of a filler-bitumen mixture is directly related to the particle size of the filler
2. The filler type and amount have a considerable effect on the flexural properties of a mix. As a result of an increase in the filler content, there is an increase in flexural stiffness.
3. With respect to mixture toughness, there is an optimum filler content corresponding to maximum toughness. For the fillers considered, the maximum toughness corresponded to a range of filler bitumen ratio between 0.25 and 0.75.

Thus the literature reviewed above indicate that the importance of fillers in improving the performance of asphalt concrete mixtures has been well recognised. However, it is worth noting that different fillers have different effects, some are bitumen extenders (increase in volume) but others show stiffening effect. While many agree that fillers have favourable influence on rut resistance of asphalt mixtures, other considerations such as workability, susceptibility to cracking, and moisture susceptibility may dictate the proportion and type of fillers to be used in asphalt mixtures.

3.3 Effect of Binder on the Permanent Deformation Response of Asphalt Mixtures

Bituminous binders form another important component of asphalt concrete. Bitumens are hydrocarbons which are soluble in carbon disulphate. They are usually fairly hard at normal temperatures, but when heated they soften and flow. When mixed with aggregates in their fluid state, and then allowed to cool, they solidify and bind the aggregates together, thus the name binder. Asphalt binders are visco-elastic materials whose resistance to deformation under load is very sensitive to loading time and temperature. A visco-elastic material combines elastic behaviour, in which material recovers to its initial state after removal of the applied load, and viscous behaviour, in which the material deforms constantly under applied loads. Figure 3.5 illustrates the response of elastic, viscous and visco-elastic materials. At any combination of time and temperature, linear visco-elastic properties must be characterized by at least two properties: the total resistance to deformation and the relative distribution of that resistance between an elastic part and a viscous part.

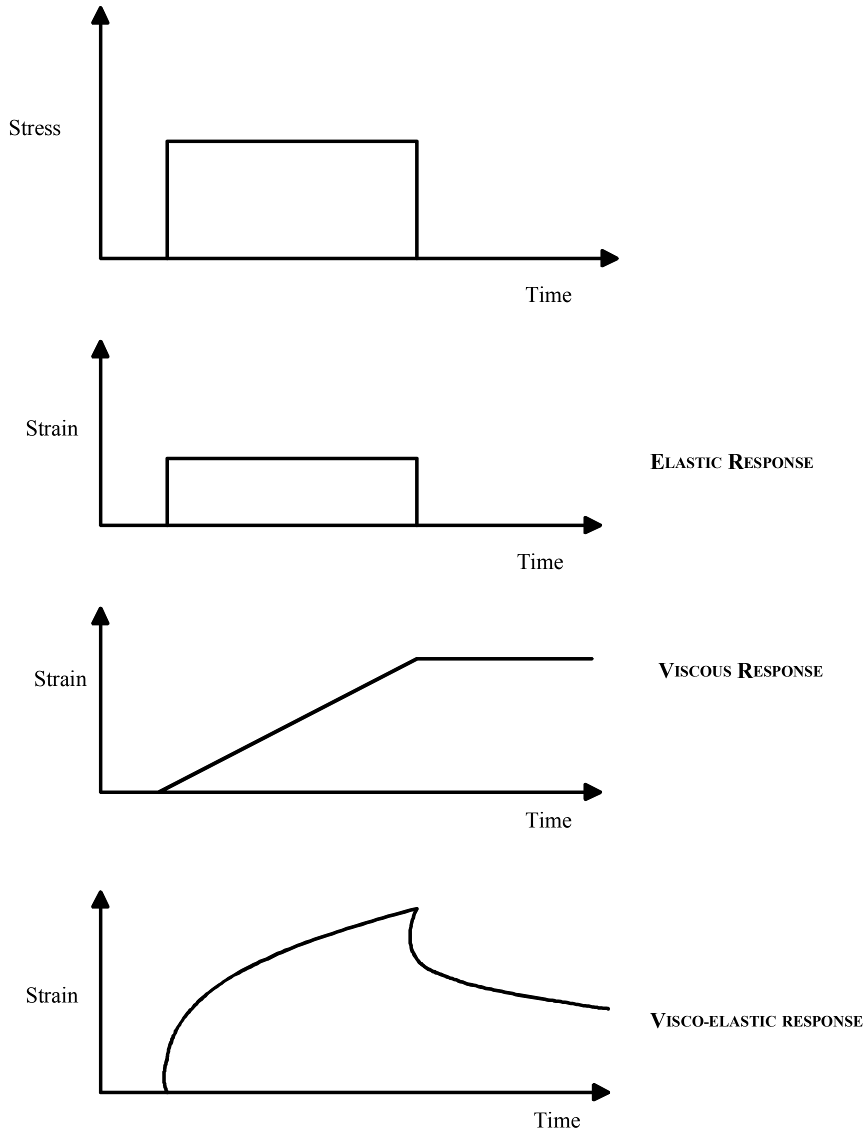


Figure 3.5 Idealized response of elastic, viscous and visco-elastic materials

3.3.1 Effect of Binder Content

The binder content is a key mixture design parameter. It is known that the amount of binder in the mixture affects the durability and performance of the mixture. Most of the studies on the effect of binder on permanent deformation response of mixtures found in the literature centred on the effect of binder type (or grade) rather than the binder content. However, the studies that considered the effect of binder content found that it has significant influence on resistance to permanent deformation of mixtures.

As a part of the Strategic Highway Research Program (SHRP), tests were conducted on asphalt concrete specimens having different binder contents and compacted to different void levels[26]. Testing was conducted using the SHRP's simple shear with constant height test. Figure 3.6 shows linear models fitted to the data. The general trend indicated by the data is that an increase in binder content results in a decrease in resistance to permanent deformation, as measured by the number of cycles to 5% strain.

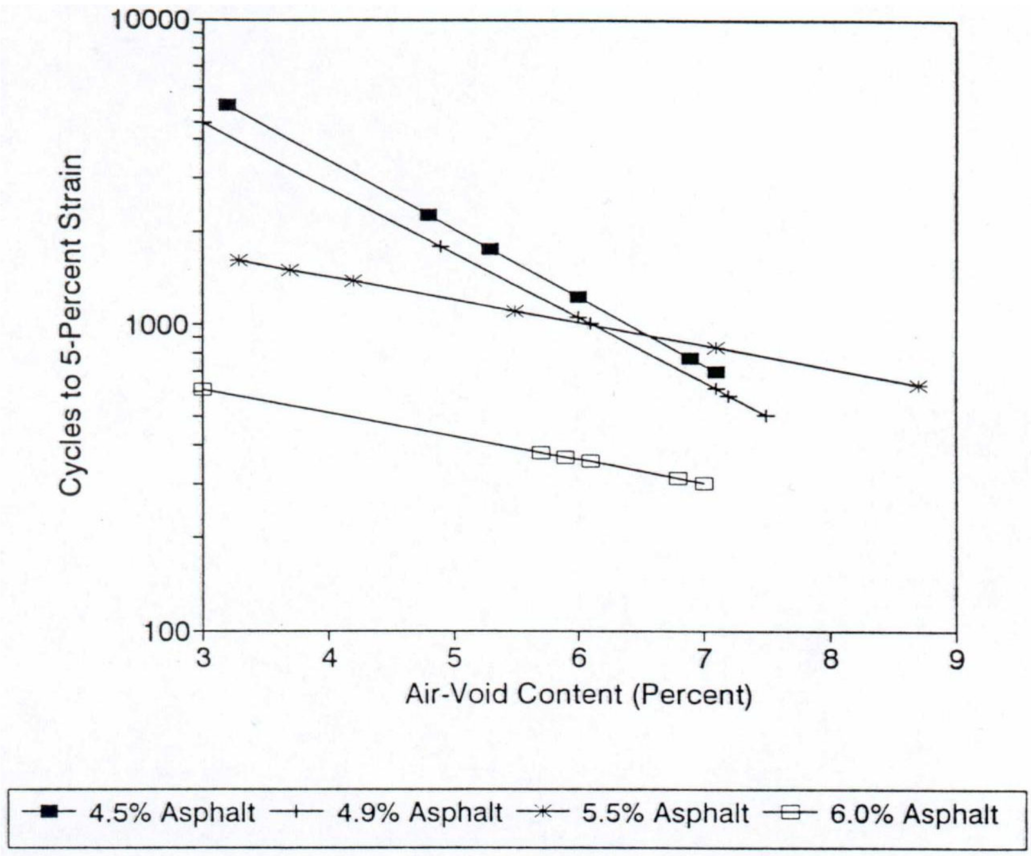


Figure 3.6 Effect of asphalt content[26]

Lee and Al-Dhalaan [27] conducted a study on rutting and asphalt mix design on a test road in Saudi Arabia, which showed severe rutting. They concluded that the possible reason for the rutting was over asphalting, i.e., high binder content. However, their study did not involve comparison of performance of mixtures with different levels of binder content. Brown and Cross[2] considered the effect of the properties and the amount of binder on rutting in their national study of rutting mentioned earlier. They concluded that the amount of binder is extremely important. Their study showed that an increase in voids filled with asphalt leads to increase in rut depth.

Coree and Button[28] reported on full scale rutting tests of large-stone asphalt mixtures. The mixtures had a nominal maximum size of 38 mm. Three levels of binder contents were considered.The results of their study showed that an increase in binder content leads to an increase in rutting.

One of the objectives of this thesis work is to evaluate the effect of binder content on the permanent deformation characteristics of asphalt concrete mixtures. Asphalt concrete specimens with three different levels of binder content were prepared and tested in cyclic load triaxial test. The binder content was found to significantly influence the permanent deformation behaviour of mixtures. This is discussed in detail in chapters 6 and 7.

3.3.2 Effect of Binder Properties

Several researchers have attempted to evaluate the effect of the binder's mechanical properties on permanent deformation behaviour of asphalt mixtures. Asphalt binder properties have been expressed using various measures, some of which are empirical and others fundamental. These measures of binder properties will be discussed first followed by review of the effects of the properties on permanent deformation behaviour.

Measures of Binder Properties

The findings of SHRP asphalt binder research indicated that to better select asphalt binders one needs to opt for its fundamental rheological and failure characterization [29]. However, rheological characterization of asphalt binder is not an easy task; asphalt binder is a difficult material and the use of rheological test equipment is expensive and complicated. These difficulties have led many asphalt researchers to simplify measurement of asphalt properties and to rely on a number of properties and empirical parameters that have been measured and correlated randomly with various indicators of pavement performance. Bahia and Anderson [29] have classified these parameters and properties into three groups: empirical single-point measurements, the viscosity measures and susceptibility parameters.

The empirical measures include penetration, ductility, and softening point. The fact that these empirical measures are not capable of characterizing the rheological properties of asphalt binders have been pointed out by many researchers[29]. Empirical as they are, they can not be expressed in engineering units and, therefore, can't be directly related to any of the required rheological properties of asphalt binders. They do not give an indication of whether a particular binder at the test temperature is more elastic or more viscous.

The viscosity measures basically involve the use of coefficient of viscosity which is a fundamental material property expressed in engineering units. The commonly used measures of viscosity are the absolute viscosity and apparent viscosity. The absolute viscosity is a fundamental measure of Newtonian fluids whose properties are not affected by rate of loading or stress level. However, asphalt binders exhibit Newtonian behaviour only at high temperatures (above the softening point) or at a very low loading rates. Thus this measure can not describe properties of asphalt binders at lower temperatures and short loading times. Apparent viscosity is a one- point measurement that represents a material behaviour only at a selected rate, strain or stress. It was thought that apparent viscosity could represent the visco-elastic properties of binders, where viscosity is not an absolute value but varies with the shear rate and shear stress at which it is calculated. However, the question of where, in the domains of time, stress and temperature, should the apparent viscosity be measured has challenged many researchers. Further more, complications related to methods of measurement of apparent viscosity, have raised many conceptual and practical issues which indicate several limitations of the use of apparent viscosity to fundamentally characterize asphalt binders. Apparent viscosity is measured using incremental creep test.

Susceptibility parameters were proposed in an attempt to describe the visco-elastic properties of binders with in the time-temperature domain. The susceptibility parameters may be grouped in to two types: Temperature susceptibility parameters and shear susceptibility parameters. Temperature susceptibility parameters include parameters such as temperature required to change the penetration by a certain amount, slope of the logarithmic plot of penetration versus temperature, viscosity changes as a function of temperature, penetration index, penetration-viscosity- number, etc. These parameters all carry the problem of empiricism, most of them do not take the time dependency of asphalt binder properties in to account, and as the time and temperature dependency of asphalt is not linear, the parameters are not constant material properties.

Shear susceptibility parameters include the degree of complex flow (C) and the shear index [29, 30]. The degree of complex flow was defined by:

$$M = \frac{T}{S^c} \qquad 3.13$$

Where M is a constant, T is the shear stress, S is shear rate, and C is the degree of complex flow. For $C = 1$, the asphalt is a Newtonian fluid and M is a steady-state coefficient of viscosity. Therefore, C was considered a good measure of non-Newtonian behaviour. Bahia and Anderson [29] pointed out that the degree of complex flow is misleading, though looks attractive for study of asphalt aging, for a number of reasons including:

1) It is calculated based on the assumption that the relation between shear stress and shear rate is linear. This may be true only over a small range of stress or strain rates. The relation between stress and strain rates is, therefore, not linear and the value of C will be arbitrary if it does not refer to a certain range of stress and strain.

2) The parameter depends on stress history (the way in which the sample is loaded). Researches have shown that changing the loading sequence from load increments that are added in decreasing sequence to load increments that are added in increasing sequence will result in dramatic change in the C value.

3) Constructing the stress- shear rate relationship suffers from the same fundamental problems associated with apparent viscosity. At each stress level, there are an infinite number of shear rates because of delayed elasticity. This may make the shear rate used in calculation of the degree of flow C arbitrary and may provide misleading results.

The other shear susceptibility parameter, shear index, is calculated by plotting apparent viscosity versus shear rate on a log-log scale. The shear index is then taken as the slope of the relationship between two different shear rates. Since this parameter is based on apparent viscosity, it is associated with limitations mentioned for apparent viscosity and the degree of flow.

The foregoing discussion indicate that conventional measures of asphalt binder properties are not capable of characterizing its fundamental rheological behaviour. This has led, in recent years, to the definition and development of new measures that are believed to enable characterization of the fundamental rheological properties of asphalt binders. At forefront of this development is the SHRP research on asphalt binders. SHRP researchers claimed that the properties that are proposed for the new SHRP binder specification were derived and selected by addressing each type of pavement failure, understanding the failure mechanisms, understanding the contribution of the binder to resistance of that failure, and selecting the required measure that will best reflect that contribution of the binder [30]. The critical pavement distress modes in which binder plays an important role were identified as rutting, fatigue cracking, and thermal cracking. Oxidative aging and physical hardening were also considered as durability factors that cause changes in properties of binders and thus affect performance. Four types of tests were selected [29, 30]:

1. The *rotational viscometer*, to measure flow properties at temperatures that mimic temperatures at which the pumping and mixing of binders occurs
2. The *dynamic shear rheometer*, to measure properties at temperatures that mimic high and intermediate pavement temperatures, and to mimic loading rates typical of traffic loading.
3. The *bending beam rheometer*, to measure properties at the lowest pavement temperatures and to mimic loading conditions that result from thermal cooling.
4. The *direct tension test*, to measure failure properties at the lowest pavement temperatures and mimic loading that results from thermal cooling.

Dynamic (oscillatory) testing is considered to be the best technique to describe the behaviour of visco-elastic materials. Using this technique in the shear mode, the complex modulus (G^*) and the phase angle (δ) are measured. G^* represents the total resistance to deformation under load, while δ represents the relative distribution of this total response between the in- phase component and the out-of-phase component. The in-phase component is an elastic component and can be related directly to energy stored in a sample for every loading cycle while the out- of- phase component represents the viscous component and can be related directly to energy dissipated per cycle in permanent flow. The relative distribution of these components is a function of the composition of the material, loading time, and temperature. Rheological properties are usually represented by a master curve, which is the variation of G^* and δ with frequency at a constant temperature or isochronal curve, which is the variation of G^* and δ with temperature at a selected frequency. Figure 3.7 shows examples of these rheological curves.

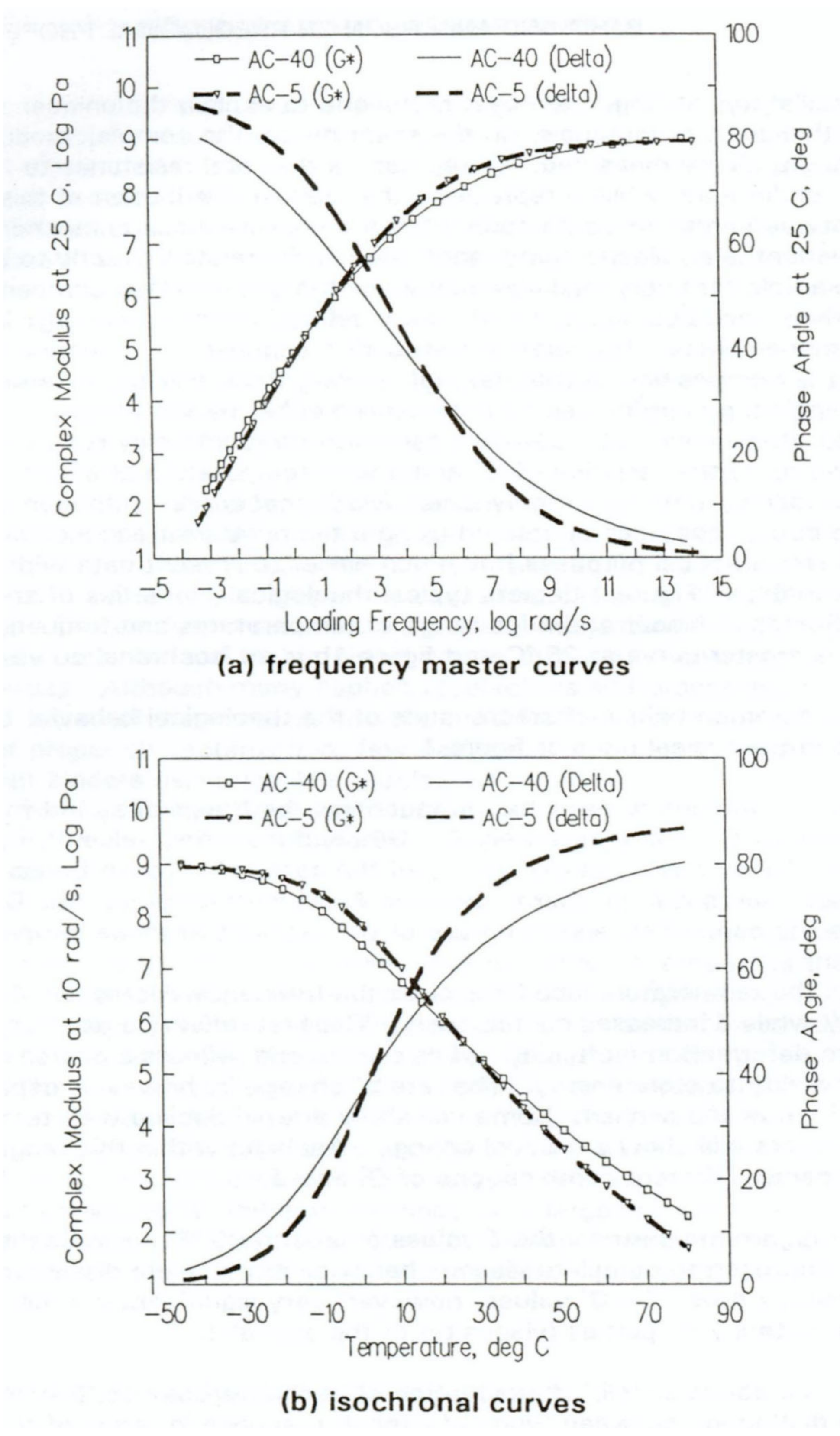


Figure 3.7 Typical rheological curves for asphalt binders[29]

Contribution of Binder to Rutting Resistance

While aggregate properties make important contribution to rutting resistance as discussed earlier, one should not ignore the contribution of the binder.Many researchers have attempted to evaluate the contribution of binder to rutting resistance and have come up with varying conclusions. The recognition of the importance of the binder in mitigating rutting has led many engineers to specify harder binders or polymer-modified binders for pavements in hot climates and heavy-duty pavements. As mentioned earlier rutting is caused by accumulation of permanent deformations resulting from the repeated applications of traffic loading. Based on the assumption that pavement rutting is mainly caused by deformations of the surface layer, Bahia and Anderson [29] argued that rutting can be considered as a stress controlled, cyclic loading phenomenon. Accordingly, during each cycle of traffic loading, a certain amount of work is being done in deforming the surface layer. Part of this work is recovered in elastic rebound of the surface layer, while the remaining work is dissipated in permanent deformation and heat. To minimize rutting, the work dissipated during each loading cycle should be minimized. For a visco-elastic material, the work dissipated per cycle (W_c) is given by:

$$W_c = \pi\sigma\varepsilon\sin\delta \qquad \textit{3.14}$$

Where σ and ε represent the stress and strain respectively. Since rutting in asphalt concrete surfacing layer is considered as a stress controlled (σ_o) cyclic loading phenomenon, the following substitution can be made:

$$W_c = \pi\sigma_0\varepsilon\sin\delta \qquad \textit{3.15}$$

$$\varepsilon = \frac{\sigma_0}{G^*} \qquad 3.16$$

Therefore,

$$W_c = \pi\sigma_0^2\left[\frac{1}{G^*/\sin\delta}\right] \qquad \textit{3.17}$$

Thus, according to equation 3.17, the work dissipated per loading cycle is inversely proportional to the parameter G*/sinδ, which is the parameter selected for SHRP specification. The parameter combines the total resistance to deformation as reflected by G* and the relative non-elasticity of binder as reflected by sinδ. SHRP researchers have also defined other parameters related to the complex modulus G*, which include the storage modulus (G') and the loss modulus (G''). The relationships between the complex modulus, storage modulus, loss modulus, and phase angle can be visualized through the trigonometry of a right triangle as shown in Figure 3.8.

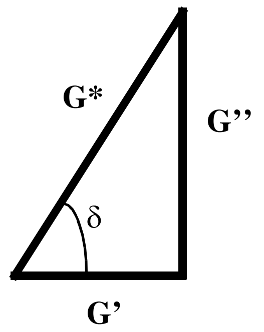


Figure 3.8 Relationship between the complex modulus, storage modulus, loss modulus, and phase angle

At a particular frequency (ω), the storage and loss moduli can be calculated using equations 3.18 and 3.19 respectively.

$$G'(\omega) = G^*(\omega)\cos\delta \qquad \textit{3.18}$$

$$G''(\omega) = G^*(\omega)\sin\delta \qquad \textit{3.19}$$

As can be seen from Figure 3.8, sinδ is the ratio of loss modulus (G'') to the complex modulus (G*). G'' is directly related to the work dissipated during loading cycle and thus, its ratio to G* gives a relative measure of the non-elastic (permanent) component of the total resistance to deformation. According to this logic, the contribution of the binder to rutting resistance can be increased by increasing its total resistance to deformation (G*) and or decreasing its non-elasticity (sinδ).

But Chritensen and Anderson [31] warn against misinterpretation of the storage and loss moduli as the elastic and viscous moduli. In reality, the elastic component of the response only represents part of the storage modulus and the viscous response only part of the loss modulus because most real visco-elastic materials exhibit a significant amount of delayed elastic response, which is time dependent but completely recoverable. Therefore, both the storage and loss moduli reflect a portion of the delayed elastic response.This casts doubt on the use of sin δ (G''/G*) as a measure of the non-elastic component of the total resistance.

Philips and Robertus [32] gave a critical analysis of the contribution of asphalt binder to pavement permanent deformation based up on the concept of *Zero-Shear-Viscosity*.The *Zero-Shear-Viscosity,* which is sometimes termed the *Newtonian* viscosity, is the viscosity in the linear regime, where stress is proportional to the shear rate. In the linear visco-elastic regime, stress and strain are linearly proportional to each other, although both are time dependent. In this regime the shear relaxation modulus and the stiffness modulus are independent of the stress and strain level, and the resulting viscosity is independent of shear rate. Philips and Robertus argued that, since rational pavement design methods are based on the premise that wheel loading occurs in the linear regime, a linear viscosity, i.e., the *Zero-Shear-Viscosity* is required for the purpose of being consistent. However, as discussed in chapter 4, available evidence indicates that under service conditions asphalt concrete materials exhibit a non-linear viscoelasto-plastic behaviour. Therefore, it might be argued that the assumption of linearity itself that is made in pavement design is not realistic and the need to be consistent with this assumption might not be a strong point in favour of the use of *zero-shear-viscosity*. It has also been argued that, at macroscopic (continuum) level, asphalt pavement deformation is the combined result of asphalt compaction and asphalt displacement, and arises from binder permanent deformation and particle slip. The permanent (non-recoverable) part of the asphalt deformation arises from processes which are purely dissipative, and is described by a viscosity or friction coefficient. This assertion follows from the second law of thermodynamics which states that the mechanical work expended in generating permanent deformation is dissipated as heat, and hence the process is irreversible. Following this line of reasoning, Philips and Robertus [32] argued that the binder's contribution to the rutting process is a permanent deformation which arises solely from a dissipative process, a viscosity. A logical question is which viscosity is to be used. Based on the assumption of linearity used in pavement design methods, Philips and Robertus conclude that the so-called *Zero-Shear-Viscosity* (η_o) might be appropriate.

The deformation of the binder and asphalt during wheel loading is related to the visco-elastic (creep) compliance, J(t), which is inversely proportional to the stiffness modulus. The time dependent response on and after wheel loading can be converted into the corresponding frequency response by means of an integral transform:

$$J''(\omega) - \int_0^{\infty} \omega \, [J_{de}(\infty) - J_{de}(t)] \cos \omega t \, \delta t = 1/\omega\eta_o \qquad 3.20$$

Where:

$J''(\omega)$	= loss compliance
η_o	= the Zero-shear-Viscosity
ω	= frequency
J_{de}	= The delayed elastic component of J(t)

This integral equation is valid for all visco-elastic materials in the linear range. It is a positive quantity which is larger for binders with more delayed elasticity and tending to zero at zero frequency.The SHRP rutting parameter ($G^*/\sin\delta$) is the inverse of J''(i.e, $1/J''$).The integral equation (equation 3.20) quantifies the difference between instantaneous deformation produced by traffic and the permanent deformation, in terms of the amount of elasticity and therefore reflects the rational for selection of $G^*/\sin\delta$ as a performance parameter by SHRP researchers. However, by correlating $G^*/\sin\delta$ at $\omega = 10$ radians with dynamic creep rate on iso-viscosity basis, Philips and Robertus found out that delayed elasticity reduces the value of SHRP rutting parameter and argued that the *Zero-Shear-Viscosity* can better describe the contribution of binder to rutting. This is illustrated in Figure 3.9.

The difference between η_o and SHRP's $G^*/\sin\delta$ arises from delayed elastic recovery. $G^*/\sin\delta$ is measured at short loading times representative of traffic loading while recovery can be a slow process. Thus η_o measures the binder modulus which exists after traffic loading and delayed elastic recovery. An objection to η_o is that it does not exist for materials with yield stress, such as gelled materials. The appropriate viscosity to be used in the non-linear regime can not be decided from linear theory alone.

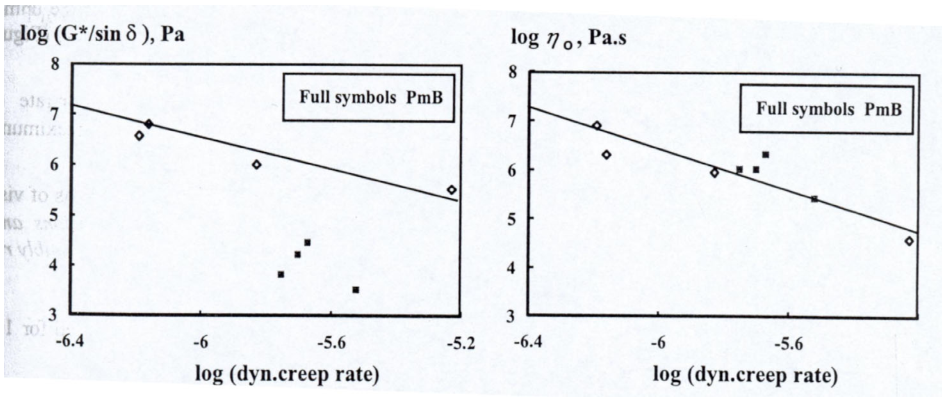


Figure 3.9 Correlation of G*/sinδ and η_0 with asphalt permanent deformation in dynamic creep[32]

Sybilski [33] measured binder viscosity, η, at various shear rates, and obtained the *Zero-Shear-Viscosity*, η_o by fitting the data to a simplified Cross equation (equation 3.21):

$$\eta = \frac{\eta_0}{\left[1 + \left(K\frac{d\gamma}{dt}\right)^m\right]} \qquad 3.21$$

Where:

η	= apparent viscosity (MPa)
η_o	= Zero-shear-viscosity (MPa)
$d\gamma/dt$	= shear rate
K, m	= constants

Sybilski found that correlation of rutting at 45°C in a wheel tracking test with η_o at 60°C is good ($R^2 = 0.82$). However, the corresponding correlation with softening point is found to be less satisfactory ($R^2 = 0.56$). The author concluded that the rutting resistance of polymer modified bitumen made from a highly modified, soft, base bitumen may be over- estimated by *zero-shear-viscosity* and softening point mainly because of equipment limitation to measure the *zero-shear-viscosity* at sufficiently low shear rates.

Oliver [34] reported the result of a major study, which was undertaken to develop relationships between binder rheological properties and the rutting resistance of asphalt mixes as indicated by laboratory wheel tracking tests. Ten binders involving plain binders of different grades and modified binders were used and were characterized by a number of procedures including dynamic shear rheometer, elastometer consistency and softening point. A single asphalt composition (grading and binder content) was employed in the study so that differences in rutting behaviour could be directly related to the binders used. Roller compacted samples of asphalt were prepared using each of the binders and tested in wheel tracking machine at 45^{o}C and 60^{o}C. Oliver concluded from this study that there is a clear linear relationship between log wheel tracking rate and log G*/ sinδ for those binders not containing added polymer. However, the polymer modified binders all fell below the line. Only elastometer consistency provided a satisfactory correlation for both modified and unmodified binders, with a plot of log consistency against log wheel tracking rate giving an R^2 value of 0.89.

Ramond et al [35] studied two mixes made with unmodified and polymer modified bitumen. They found a good correlation, with R^2 varying from 0.85- 0.92, of G*/ sinδ measured before RTFOT (**R**olling **T**hin **F**ilm **O**ven **T**est) with rut depth. But after RTFOT, the correlation was found to be very poor. The authors argued that, although specifying a minimum binder modulus provides a good starting point, it is insufficient to prevent rutting since mix properties including aggregate, mix composition and additives play an important role.

Collop and Khanzada [36] carried out a wheel tracking test and repeated load axial test on two idealized bituminous mixtures over a range of temperatures and stress levels. They also performed rheological measurements on the bitumen using the Dynamic Shear Rheometer (DSR) in terms of SHRP's rutting parameter (G*/sin δ) and correlated it to the results from the wheel tracking and repeated load axial tests.They concluded that there is a considerable scatter caused mainly by non-linearities not accounted for in the SHRP rutting parameter. Figure 3.10 shows some of the results of their study. Other authors including Chabert et al [37], Michel and Bernand [38] reported a poor correlation of rut depth in wheel tracking test at 60^{o}C with G*/sin δ.

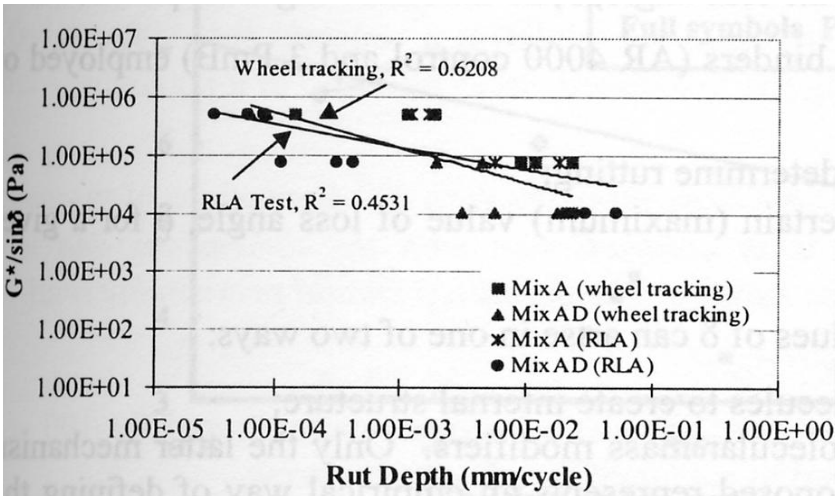


Figure 3.10 Correlation of G*/sinδ to rutting in wheel tracking test[36]

It appears that, except for some unmodified binders, SHRP's rutting parameter might not be a good indicator of the binders contribution to the resistance to deformation of asphalt mixtures. Many of the studies mentioned in the preceding paragraphs indicate that the correlation between SHRP's parameter and rutting is poor. However, it has to be pointed out that most of these correlations are based on wheel tracking test. Wheel tracking test is itself an empirical test, the correlation with field performance of which have come under question as discussed in chapter 5.There is also a theoretical objection to G*/sinδ arising from the fact that the parameter can increase with an increase in temperature (or decrease in frequency) for some binders.Therefore, there appears to be a need to explore other parameters that might be used as measures of the binder's contribution to rutting resistance of mixtures.However, the fact that the properties of the binder have a significant influence on rutting resistance is not disputed.

3.4 Effect of Void Content

The amount of voids in asphalt mixture is one of the factors that significantly affect performance throughout the life of an asphalt pavement. It is known that low voids lead to rutting while high voids are associated with permeability of water and air, resulting in moisture damage, oxidation, ravelling and cracking. Voids in the pavement are controlled by asphalt content, compactive effort during construction, and additional compaction under traffic. The voids in asphalt mixture are directly related to density, thus it is necessary to closely control the density in order to ensure that the voids stay within acceptable range. Two commonly used methods for density specification are the percent of laboratory density and percent of theoretical maximum density. A problem with density is the method of measurement.The two commonly used methods include measurement of bulk density of core taken from the in-place pavement, and use of a nuclear gauge to measure the in-place density. The later method is not considered to be as accurate as the former.

Ford [39]studied the effect of air voids on pavement performance. He measured voids in cores taken from field and correlated it to measured rutting. The author concluded that air voids have significant effect on rutting with correlation coefficient of 0.674, and that rut depth increases with decreasing air voids. Figure 3.11 shows the relationship between rut depth and air voids. This study also showed that pavements with air void content of 2.5 or less had excessive rutting.

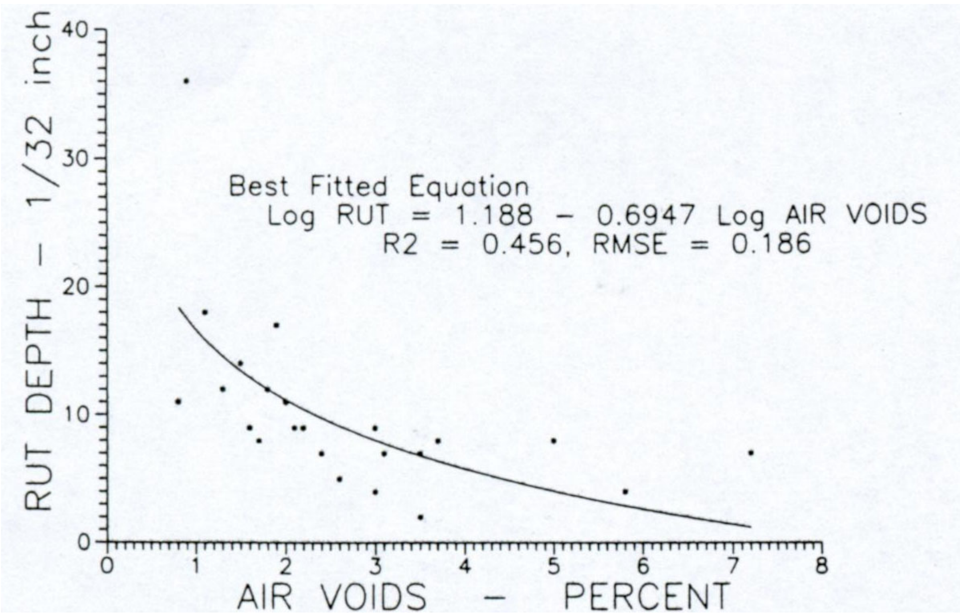


Figure 3.11 Effect of void content on rutting[39]

Kandhal et al [40] argued that mixtures with low voids in mineral aggregate and high asphalt contents have a tendency to have very low air void contents after densification by traffic and loose stability, i.e., start to rut and shove after reaching a critical compaction level. Brown and Cross [2] concluded, from their study on rutting in hot mixed asphalt, that in-place air voids above approximately 3% are needed to reduce the chances of premature rutting. Mallick et al [41] also argued that there is a critical air void of approximately 3% below which strain increases rapidly.

The mechanism of rutting at low air voids is identified as dialtion, i.e., an increase in volume. At low voids the coated aggregate particles will climb over each other under the action of the load resulting in an increase in volume and shoving. This behaviour has also been observed in the field in that pavements with insufficient compaction experience densification under traffic and mixtures with low voids often increase in voids when shoving takes place [41].

Evaluation of the effect of void content on permanent deformation response of asphalt concrete mixtures is one of the objectives of this thesis work as mentioned earlier. Several specimens with three levels of void content were tested. The results indicate that void content has a significant influence on permanent deformation characteristics of asphalt concrete mixtures. The fact that low void content leads to dialation is also demonstrated using the measured volumetric strain. A detailed discussion of the effect of void content on permanent deformation response is given in chapters 6 and 7.

CHAPTER 4: REVIEW OF MODELS FOR DEFORMATION OF ASPHALT CONCRETE MIXTURES

It is now a well established fact that the response of asphalt concrete mixtures depends heavily on temperature and time of loading. At low temperatures (or short loading times) asphalt concrete may behave as linear visco-elastic material but at high temperatures (long loading times) it behaves as a non-linear elasto-viscoplastic material. Furthermore, asphalt concrete exhibits the properties of dilatancy and hardening. The effect of aggregates on the response of asphalt concrete may also be much more pronounced at high temperatures.

In the case of linear visco-elasticity, one may find an appropriate physical model (such as Maxwell, Kelivin, etc.) components that may be arranged in series or in parallel to best fit test results. However, since rutting is generally considered to be a high temperature problem, a realistic model for prediction of rutting or rutting resistance may have to be based on elasto-viscoplasticity in order to account for the behaviour of asphalt concrete at high temperatures. In an elasto-viscoplastic material four components of strain may be identified; i.e, elastic, viscoelastic, viscoplastic, and plastic.

The strains due to elasticity are fully recoverable and independent of time and hence, in a loading/unloading cycle, no permanent strains are generated and the recoverable strains are independent of the rate of loading and unloading. The strains due to viscoelasticity are time dependent and recoverable. The strains due to viscoplasticity and plasticity are permanent. The strains due to viscoplasticity are time dependent while the strains due to plasticity are time independent. In asphalt concrete, the total strain is expected to contain these four components. A method of partitioning the strain into these four components will be discussed in chapter 7.

The permanent deformation response of asphalt concrete mixes to loading may have to be characterized by a constitutive model that is comprehensive and is compatible with the pavement structural analysis methods. The finite element method is currently considered to be suitable for analysis of pavement structures. Thus an ideal constitutive model for asphalt concrete might be one that is compatible with the finite element idealization of the pavement structure.

This chapter will review models for description and prediction of deformation properties of asphalt concrete. Viscoelastic models are briefly reviewed followed by discussion on the application of these models to characterize asphalt concrete. Some background on the theory of elasto-viscoplasticity is also given and the possibilities and difficulties that may be encountered in applying this theory to model the properties of asphalt concrete are discussed.

4.1 Rheology of Asphalt Concrete Mixtures

Permanent deformation in asphalt concrete layer is caused by densification and shear deformation under repeated axle loads. It develops gradually with increasing number of load applications. Figure 3.1 Shows a schematic permanent deformation curve that may be obtained by monitoring road pavements or conducting simulative experiments.The initial range, referred to as primary creep range represents a compaction regime where the material is considered to experience additional compaction under traffic loading. The compaction may result in an improved aggregate interlock and consequently, the rut rate (the slope of the permanent deformation curve) decreases. In the second range, known as the secondary creep range, the rate of deformation is slower but constant. The third range, called the tertiary creep range, is a catastrophic range and is reached when the rut rate begins to increase again. The last range usually involves large scale aggregate movements.

The main attributes of asphalt mixes that can be associated with permanent deformation are discussed by Sousa and Wiessman [42]. These attributes include; rate and temperature dependency, dialation, and air void dependent behaviour. The response of asphalt concrete is also dependent on loading history and path. Permanent deformation properties of mixtures are also dependent on the binder content and properties of the component materials as discussed in the previous chapter. Thus, permanent deformation of asphalt mixes is a complex phenomenon where aggregate, asphalt, and asphalt-aggregate interface properties control the overall performance. This properties may change over time as a result of, for instance, aging or moisture damage to the asphalt-aggregate interface.

As a result of the influence of the binder, the response of asphalt mixes is dependent on temperature and rate of loading. This property is of mixes has been well documented and has been generally presented in terms of master curves for dynamic modulus and phase angle.

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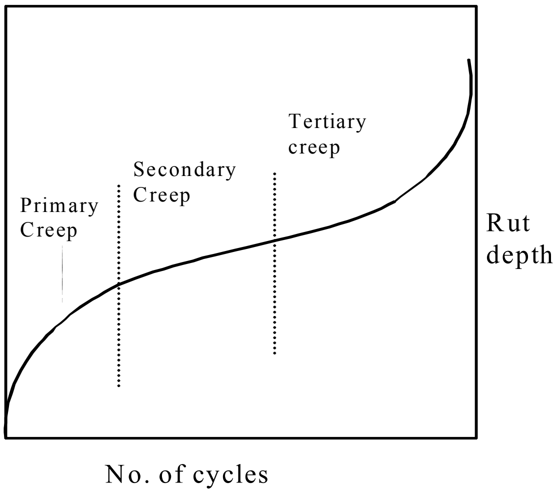


Figure 4.1 Schematic permanent deformation curve

Available evidence indicates that asphalt mixes may dialate when subjected to shearing deformations. This indicates that there is volumetric deviatoric coupling, i.e., deviatoric stresses may lead to volume/ pressure change and increased hydrostatic pressure leads to deviatoric stiffening. Figure 4.2 illustrates the phenomenon of dialation. It shows the results of a study in which specimens 10 cm in diameter and 5 cm high were subjected to a sustained shear stress of 35 kpa while the axial stress was maintained at a constant value of 17 KPa [26]. It can be seen from the figure that the rate of dialation is dependent on aggregate structure resulting from different methods of compaction. The same study also indicated that dialation is dependent on state of stress. Further evidence of dialation is presented in chapter 7 based on triaxial test conducted as part of this thesis work.

Axial creep test conducted at different confining pressures provided evidence of stress hardening with confining pressure. Figure 4.3 shows the result of such a test. It can be observed that with increase in confining pressure the permanent deformation was significantly reduced. This is caused by an increase of shear moduli resulting from the increase in confining stress.

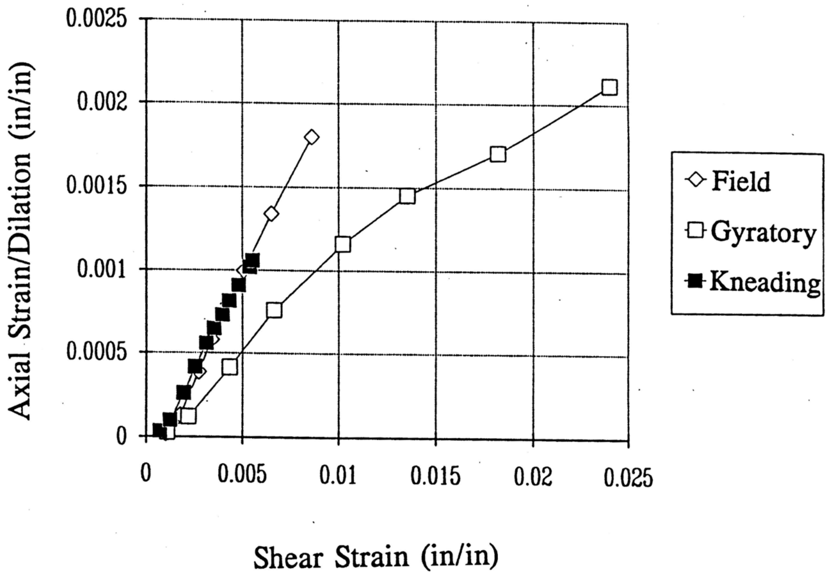


Figure 4.2 Comparative dilational response of asphalt concrete specimens to shear creep loading[26]

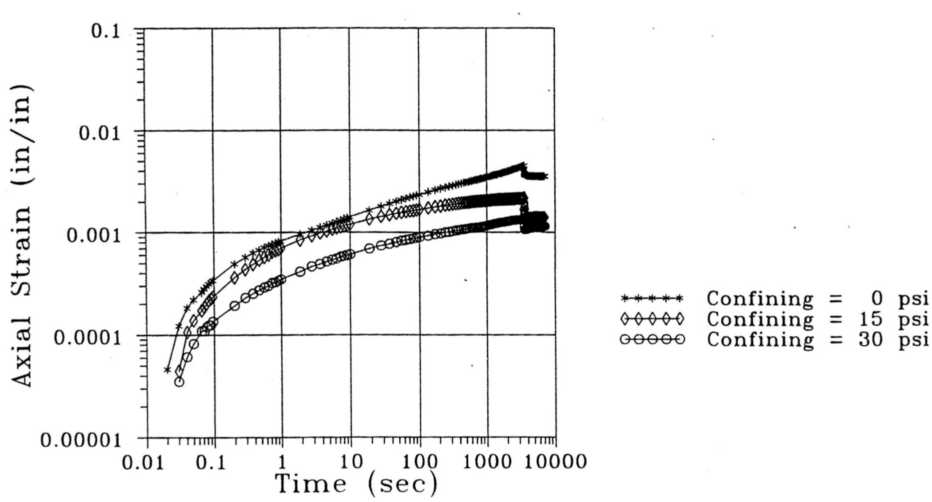


Figure 4.3 Effect of confining pressure on creep behaviour at 40°C[26]

The dependence of permanent deformation response on void content has been discussed in chapter 3. Further evidence is presented in chapter 6 from test results.A comprehensive constitutive model should accommodate all of the attributes discussed above in order to take the rheological properties of the mixture into account. Available models for deformation of asphalt concrete should be evaluated on the basis of whether or not they take the rheological properties into account as well as on the basis of their correlation with observed results from field. In the following sections, available models and their theoretical basis are reviewed.

4.2 Viscoelastic Models

A number of mechanical models have been proposed to study the stress-strain-time relation of visco elastic materials. The basic elements in all linear viscoelastic models are linear springs and linear viscous dashpots. This section gives an overview of the commonly used viscoelastic models mainly based on Findley et al [43].

In a linear spring, the stress-strain relation may be expressed as:

$$\sigma = R\varepsilon \tag{4.1}$$

Where R can be interpreted as a linear spring constant or elastic modulus. The spring element exhibits instantaneous elasticity and recovery. The stress can be related to strain rate in linear viscous dashpot as:

$$\sigma = \eta\dot{\varepsilon} \tag{4.2}$$

where $\varepsilon^{\cdot}$ is the strain rate and η is the coefficient of viscosity. Equation 4.2 states that the strain rate is proportional to the stress, i.e., the dashpot will deform continuously at a constant rate when subjected to a step of constant stress. But if a step of constant strain is imposed on the dashpot, the stress will have an infinite value at the instant when the constant strain is imposed and it will rapidly diminish with time to zero and will remain zero. However, an infinite stress is impossible in reality. It is therefore impossible to impose instantaneously any finite deformation on the dashpot.

4.2.1 Maxwell Model

The Maxwell model consists of a linear spring and a linear viscous dashpot connected in series as shown in fig 3.3.

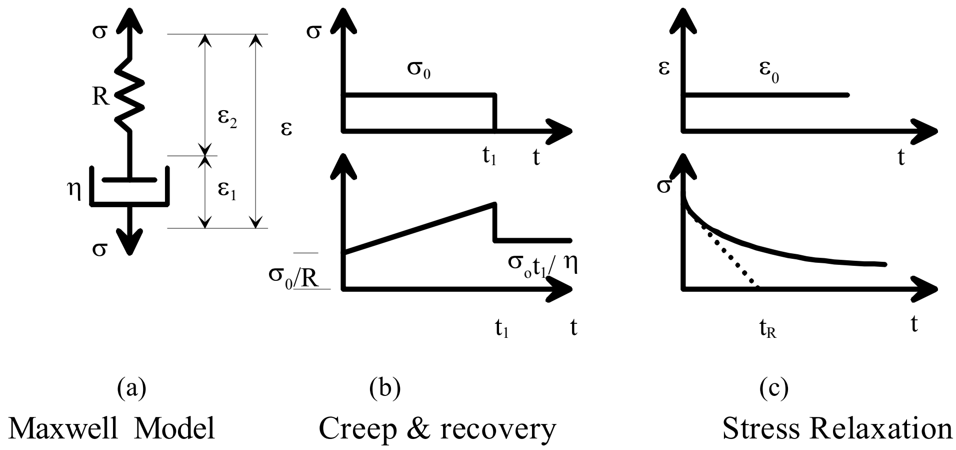


Figure 4.4 Behaviour of Maxwell Model

The stress - strain relations for the spring and dashpot are given respectively by:

$$\sigma = R\varepsilon_2 \qquad 4.3$$

$$\sigma = \eta\dot{\varepsilon}_1 \qquad 4.4$$

Since the spring and dashpot are connected in series, the total strain and the total strain rate are given respectively by:

$$\varepsilon = \varepsilon_1 + \varepsilon_2 \qquad 4.5$$

$$\dot{\varepsilon} = \dot{\varepsilon}_1 + \dot{\varepsilon}_2 \qquad 4.6$$

Inserting equation 4.4 and time derivative of equation 4.3 in to equation 4.6, the internal variables ε_1 and ε_2 can be eliminated and the following stress strain relationship for Maxwell model is obtained:

$$\dot{\varepsilon} = \frac{\dot{\sigma}}{R} + \frac{\sigma}{\eta} \qquad 4.7$$

By solving this differential equation (4.7), the strain time relations under various stress conditions or the stress - time relations under a given strain input may be obtained. For a constant stress applied at time t = 0 for instance, equation 4.7 becomes a first order differential equation of ε, which can be integrated (with initial condition $\sigma = \sigma_0$ at $t = t_0$) to obtain:

$$\varepsilon(t) = \frac{\sigma_0}{R} + \frac{\sigma_0}{\eta}t \qquad 4.8$$

This result is shown in Figure 4.4b. If the stress is removed from Maxwell model at time t_1, the elastic strain σ_0/R in the spring returns to zero at the instant the stress is removed, while $(\sigma_0/\eta)t_1$ represents a permanent strain which does not disappear. If the Maxwell model is subjected to a constant strain ε_0 at time t = 0, for which the initial value of stress is σ_0, the stress response can be obtained by integrating equation 4.7 for these initial conditions resulting in:

$$\sigma(t) = \sigma_0 e^{-R(t/\eta)} = R\varepsilon_0 e^{-R(t/\eta)} \qquad 4.9$$

Equation 4.9 describes the stress relaxation phenomenon for Maxwell model under constant strain which is shown in Figure 4.4c. Derivative of equation 4.9 gives the rate of stress change as follows:

$$\dot{\sigma} = -\left(\sigma_0\frac{R}{\eta}\right)e^{-R\frac{t}{\eta}} \qquad 4.10$$

Thus the initial rate of change in stress at t = 0+ (where 0+ refers to the time just after the application of the strain) is:

$$\dot{\sigma} = -\sigma_0\frac{R}{\eta} \qquad 4.11$$

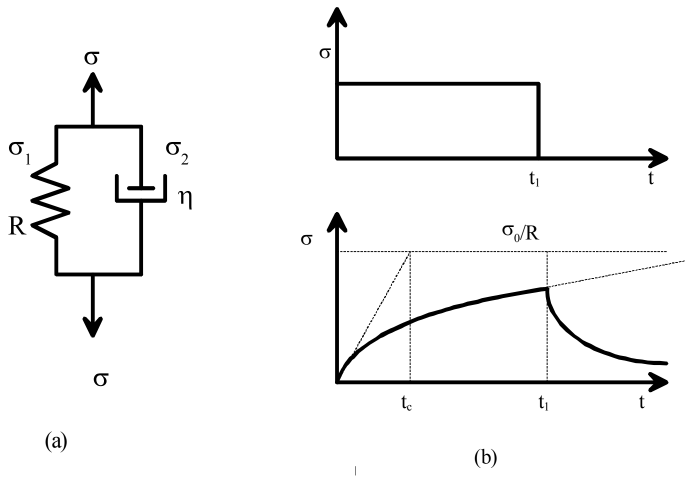
If the stress were to decrease continuously at this initial rate, the relaxation equation would be of the following form:

$$\sigma = -\left(\sigma_0 R\frac{t}{\eta}\right) + \sigma_0 \qquad 4.12$$

According to equation 4.12, the stress would then reach zero at time $t_R = \eta/R$, which is called the relaxation time of the Maxwell model.The relaxation time characterizes one of the viscoelastic properties of a material. At the relaxation time only 37% of the initial stress remains.

4.2.2 Kelvin Model

The kelvin model is another two element model in which a spring element and dashpot element are connected in parallel. Figure 4.5 shows the behaviour of the Kelvin model.



(a) Kelvin Model

(b) Creep and Recovery

Figure 4.5 Behaviour of Kelvin Model

The stress and strain relations for the spring and dashpot are:

$$\sigma_1 = R\varepsilon \qquad 4.13$$

$$\sigma_2 = \eta\dot{\varepsilon} \qquad 4.14$$

The total stress is given by:

$$\sigma = \sigma_1 + \sigma_2 \qquad 4.15$$

Elimination of σ_1 and σ_2 among equations 4.13, 4.14 and 4.15 yields the following equation between the stress σ and strain ε:

$$\dot{\varepsilon} + \frac{R}{\eta}\varepsilon = \frac{\sigma}{\eta} \tag{4.16}$$

The solution of equation 4.16 can be shown to have the following form for creep under constant stress σ_0 applied at t = 0,

$$\varepsilon = \frac{\sigma_0}{R}\left(1 - e^{-R\frac{t}{\eta}}\right) \tag{4.17}$$

The strain described by equation 4.17 increases with a decreasing rate and approaches the value of σ_0/R asymptotically when t tends to infinity. The response of this model to an abruptly applied stress is that the stress is carried entirely by the viscous element, η, at first but as the viscous element elongates, greater and greater portion of the load is transferred to the elastic element, R. Thus finally the entire stress is carried by the elastic element. This behaviour is called delayed elasticity. The strain rate for Kelvin model in creep under constant stress can be found by differentiating equation 4.17:

$$\dot{\varepsilon} = \frac{\sigma_0}{\eta}e^{-R\frac{t}{\eta}} \tag{4.18}$$

Thus the initial strain rate is finite with a value of σ_0/η, and the strain rate approaches the value of zero asymptotically when t tends to infinity.If the strain were to increase at its initial rate, it would cross the asymptotic value σ_0/R at time $t_c = \eta/R$, called the retardation time. Actually, 63% of the total strain occurs within the retardation time period.

The kelvin model does not show a time-dependent relaxation. Because of the presence of the viscous element an abrupt change in strain can be accomplished only by an infinite stress. Even if the change in strain is achieved, say by low application of strain, the stress carried by the viscous element drops to zero but a constant stress remains in the spring.

4.2.3 Burgers Model

Burgers model consists of a Maxwell model and a Kelvin model connected in series. The constitutive equations for Burgers model can be derived by considering the strain response under constant stress of each of the elements and using mathematical techniques such as Laplace transforms and is given by the following equation:

$$\sigma + \left(\frac{\eta_1}{R_1} + \frac{\eta_1}{R_2} + \frac{\eta_2}{R_2}\right)\dot{\sigma} + \frac{\eta_1\eta_2}{R_1R_2}\ddot{\sigma} = \eta_1\dot{\varepsilon} + \frac{\eta_1\eta_2}{R_2}\ddot{\varepsilon} \qquad 4.19$$

where the subscripts 1 and 2 refer to the Maxwell and Kelvin components respectively and the double dots indicate second time derivative of stress and strain. This differential equation may be solved for given initial conditions to obtain the creep behaviour of Burgers model. It can be shown that the creep behaviour of the burgers model is the sum of the creep behaviour of the Maxwell and Kelvin models. The creep behaviour of Burger's model is described by the following equation:

$$\varepsilon(t) = \frac{\sigma_0}{R_1} + \frac{\sigma_0}{\eta_1}t + \frac{\sigma_0}{R_2}\left(1 - e^{-R_2\frac{t}{\eta_2}}\right) \qquad 4.20$$

The first two terms on the right side of equation 4.20 represents instantaneous elastic strain and viscous flow, and the last term represents delayed elasticity. The creep rate can be found by differentiating equation 4.20 and is expressed as follows:

$$\dot{\varepsilon} = \frac{\sigma_0}{\eta_1} + \frac{\sigma_0}{\eta_2}e^{-R_2\frac{t}{\eta_2}} \qquad 4.21$$

Thus the creep rate starts with a finite value of $\sigma_0(1/\eta_1 + 1/\eta_2)$ and approaches asymptotically to the value of σ_0/η_1 . If the stress σ_o is removed at time t_1, the recovery behaviour of the Burgers model can be obtained from equation 4.20 and the superposition principle by considering that at $t = t_1$, a constant stress $\sigma = -\sigma_o$ is added. This gives, for $t > t_1$:

$$\varepsilon(t) = \frac{\sigma_0}{\eta_1}t_1 + \frac{\sigma_0}{R_2}\left(e^{\frac{R_2t_1}{\eta_2}} - 1\right)e^{-R_2\frac{t}{\eta_2}} \qquad 4.22$$

As can be seen from equation 4.22, the recovery has an instantaneous elastic component followed by creep recovery at a decreasing rate. The second term of equation 4.22 decreases towards zero for large times, while the first term represents a permanent strain due to a viscous flow of η_1.

4.2.4 Generalized Maxwell and Kelvin models

If several Maxwell models are connected in series or several Kelvin models are connected in parallel, the resulting models describe the same mechanical behaviour as the single Maxwell model or a single Kelvin model respectively. On the other hand several Maxwell models connected in parallel represent instantaneous elasticity, delayed elasticity with various retardation times, stress relaxation with various relaxation times and also viscous flow. The generalized Maxwell model is convenient for predicting the stress associated with a prescribed strain variation, since the same prescribed strain is applied to each individual element and also the resulting stress is the sum of the individual contributions. The contribution of the i-th element can be described as:

$$D\varepsilon = \frac{D\sigma_i}{R_i} + \frac{\sigma_i}{\eta_i} \tag{4.23}$$

where D is a differential operator with respect to time D = d/dt, from which,

$$\sigma_i = \frac{D}{\frac{D}{R_i} + \frac{1}{\eta_i}}\varepsilon \tag{4.24}$$

The sum of both sides of 4.24 yields, for a-elements:

$$\sigma = \sum_{i=1}^{a} \sigma_i = \left(\sum_{i=1}^{a} \frac{D}{\frac{D}{R_i} + \frac{1}{\eta_i}} \right) \varepsilon \tag{4.25}$$

Similarly if several Kelvin models are connected in parallel, they do not exhibit any different behaviour than an equivalent Kelvin model. However, several Kelvin models connected in series will result in generalized Kelvin model, which better represents the behaviour of viscoelastic materials. The strain contribution of i-th element in the series can be expressed as:

$$\varepsilon_i = \frac{1}{D\eta_i + R_i}\sigma \tag{4.26}$$

The sum of the strain contributions of a-elements is:

$$\varepsilon = \sum_{i=1}^{a} \varepsilon_i = \left(\sum_{i=1}^{a} \frac{1}{D\eta_i + R_i} \right) \sigma \tag{4.27}$$

The generalized Maxwell model is more convenient than generalized Kelvin model in viscoelastic analysis where the strain history is prescribed whereas the generalized Kelvin model is more convenient in cases where the stress history is prescribed.

The creep strain of the generalized Kelvin model under constant stress, σ_0, can be obtained directly, instead of solving 4.27, by considering that the total strain is the sum of the creep strain of each individual Kelvin model. Thus, the creep strain of the generalized Kelvin model has the following form:

$$\varepsilon(t) = \sigma_0 \sum_{i=1}^{a} \varphi_i \left(1 - e^{-\frac{t}{t_c^i}}\right) \qquad 4.28$$

where $\varphi_i = 1/R_i$ is creep compliance, and $t^i_c = h_i/R_i$ is the retardation time.If the number of Kelvin elements in the generalized Kelvin model increases indefinitely, the creep strain becomes:

$$\varepsilon = \sigma_0 \int_0^{\infty} \varphi(t_c)\left(1 - e^{-\frac{t}{t_c}}\right) dt_c \qquad 4.29$$

where $\varphi(t_c)$ is a distribution function of retardation times called the retardation spectrum.

Similarly the stress relaxation of generalized Maxwell model under constant strain, ε_0, can be expressed as:

$$\sigma(t) = \varepsilon_0 \sum_{i=1}^{a} R_i e^{-\frac{t}{t_R^i}} \qquad 4.30$$

Where $t^i_R = \eta_i/R_i$ is the relaxation time. For a continuous distribution of relaxation times equation 4.30 will have the following form:

$$\sigma(t) = \varepsilon_0 \int_0^{\infty} R(t_R) e^{-\frac{t}{t_R}} dt_R \qquad 4.31$$

where $R(t_R)$ is called the relaxation spectrum, which is a distribution function of relaxation times.

4.3 Use of Viscoelasticity to Model Asphalt Concrete Properties

A material is said to be linearly visco-elastic if stress is proportional to strain at a given time, and the linear superposition principle holds. Findley et al [43] stated these linear requirements mathematically using the following two equations

$$\varepsilon[c\sigma(t)] = c\varepsilon[\sigma(t)] \qquad 4.32$$

$$\varepsilon[\sigma_1(t) + \sigma_2(t - t_1)] = \varepsilon[\sigma_1(t)] + \varepsilon[\sigma_2(t - t_1)] \qquad 4.33$$

in which ε and σ are the strain output and stress input, respectively, and c is a constant. The second requirement is usually referred to as Boltzmann superposition principle. Most materials including asphalt concrete are nearly linear over certain range of the variables stress, strain, time, temperature and non-linear over larger ranges of some of these variables. The boundary between linear region (i.e. where an assumption of linear behaviour is acceptable) and non-linear is in some cases arbitrary.

The principles of viscoelasticity has been successfully used to explain the mechanical behaviour of polymers and similar materials and much of the basic information has been developed in this area. Research on the application of viscoelasticity in characterization of polymers has served as a valuable background for asphalt technologists in establishing the viscoelastic nature of asphalt and asphalt concrete. In the last few decades several attempts have been made to characterize the time and temperature dependence of the mechanical properties of asphalt mixtures within the frame work of linear viscoelasticity. In most cases the linear viscoelastic behaviour of the asphalt concrete is assumed and the linear viscoelastic material properties in terms of creep compliance, relaxation modulus, complex compliance, and complex modulus were obtained.

Papazian[44] defined the general stress-strain equation of linear, viscoelastic materials in frequency domain and demonstrated its application to characterize asphalt concrete. He argued that asphalt concrete behaves as a linear, viscoelastic material at levels of stress sufficiently low compared with its ultimate strength, and for small strains. But neither a specific definition of ultimate strength nor an indication of the magnitude of stress that can be considered comparatively low was provided. It was attempted in the study to define the rheological properties of asphalt concrete in terms of its complex moduli using algebraic equations which are independent of time. The equations were identical in form to the stress-strain equations of classical elastic body. A given complex modulus was interpreted as an impedance of a mechanical model which represents the response of the material. The response of asphalt concrete was represented by a mechanical model made up of one Maxwell model in series with a number of Kelvin models. The dashpot in the Maxwell element was used to represent the non-recoverable (permanent) deformation.

Monismith and Secor[45] conducted research on the viscoelastic behaviour of asphalts and asphalt mixtures, which demonstrated the time dependence of stress-strain relationship. They used a four element mechanical model to fit the results of triaxial compression tests, i.e., to demonstrate instantaneous elastic deformation, retarded elastic deformation and viscous flow. They used the data from triaxial tests to develop solutions for time dependent deflection of viscoelastic plate on elastic foundation under static loading. They reported that the agreement between the actual data and that predicted by the four- element model was not perfect but the four-element model appeared to reflect the characteristics of the mixture to a marked degree over a very wide range of loading conditions. Comparisons of the predicted and measured time -dependent deflection of the viscoelastic plate indicated that the deflection profiles had the same general shape and time dependence but the measured values had magnitude considerably greater than those given by theory with the deviations between the two sets of values increasing with increased temperature. The authors argued that the cause of the discrepancy between theory and actual behaviour could be the assumption of equal properties in tension and compression of asphalt concrete but it may be argued that the discrepancy could be the result of the inability of the viscoelastic model to represent the time independent plastic deformation of asphalt concrete.

Ishihara and Kimura[46] developed a solution for analysis of two layer pavement systems based on viscoelastic theory considering asphalt concrete as Maxwell type viscoelastic material. The purpose of their study was to develop a methodology for analysis of stresses and strains in pavement systems and not as such to describe the deformation characteristics of asphalt concrete. They reported that there was a qualitative agreement between the result of their analysis and observed behaviour. Perloff and Moavenzadeh [47] studied deflection of viscoelastic medium under moving load. They used the principle of correspondence, in which time- dependent (viscoelastic) problems can be reduced to an equivalent time-independent (elastic) problems when the boundary conditions and the geometry of the body remain unchanged. Using existing solutions for elastic deflection of homogeneous, isotropic halfspace under a point load and Laplace transforms they developed a time dependent solution for deflection of homogeneous, isotropic viscoelastic halfspace. Their numerical solution indicated that a possibility exists for deflections to continue to accumulate until failure though the particular model they employed show limiting deflection. However, whether or not the model adequately describes the behaviour of viscoelastic pavement materials was not demonstrated.

Gardiner and Skok [48] used viscoelastic concepts to evaluate laboratory test results and field performance of asphalt mixtures. They employed the four element viscoelastic model used earlier by Monismith and Secor. They tested asphalt concrete specimens under constant stress and repeated load using special testing equipment. The main conclusion they made from the constant stress test was that strain at any time was not directly proportional to the stress. Conclusions drawn from the repeated load test include:

- the strain envelope (strain-time relation) is highly dependent on loading history,
- the strain envelope is dependent on frequency (rate of loading), the higher the frequency the smaller the strain,
- an increase in loading time results in an increase in magnitude of both the total strain and irrecoverable strain, and
- the lateral confining pressure has also a pronounced effect on the strain envelope.

Their conclusions indicated that they have observed some of the basic properties of asphalt concrete that are known today. However, the four element model might not be capable of fully describing the response of asphalt concrete to a given loading as the effects of non-linearity, and time independent plastic deformation were not considered.

Lai and Anderson [49] studied the properties of asphalt concrete using a series of uniaxial compression creep tests under constant loading, multiple step loading and repeated loading. They found that the response of asphalt concrete is non-linear and the total strain is mainly composed of the irrecoverable (permanent) strain. Their attempt to use the modified superposition principle, which is used to describe the non-linear creep behaviour of viscoelastic materials, was not successful as the use of the principle indicated that most of the strain is recoverable while the test results show substantial permanent deformation. They applied what is called strain hardening theory, in which the rate of permanent strain is related to the current stress and permanent strain to describe the creep behaviour of asphalt concrete. They reported that the agreement between theory and test results was satisfactory for constant stress and multi-step loading creep tests. But for repeated loading satisfactory agreement was found only up to 60 cycles.

Lytton et al [50] developed a low temperature cracking model for asphalt concrete based on linear visco-elasticity. Hopman et al [51] considered asphalt as momentarily (i.e., during passage of a wheel load) linearly visco-elastic material in their multi-layer model called VEROAD, which is based on Burgers model.

Sousa et al [42] developed a non-linear viscoelastic model, which incorporates damage parameter, to predict permanent deformation of asphalt aggregate mixes. The model consists of a number of three dimensional Maxwell elements in parallel and each Maxwell element is composed of a non-linear spring and dashpot. The dilatant and hardening properties of asphalt concrete, which are known to be due to aggregate skeleton, were associated with the spring while the temperature and rate dependency were associated with the dashpot. The model incorporates some 11 material parameters to be determined from a series of laboratory tests including Simple shear, uniaxial strain, volumetric, frequency sweep, and strain sweep tests. The authors reported that the model was capable of capturing the most important aspects of the permanent deformation response of asphalt aggregate mixes. But it was later reported by the same authors that this model failed to provide a good representation of mixture behaviour under cyclic loading. In addition the model involves too many parameters, which makes it difficult to apply in practice.

Lee and Kim [52] developed a viscoelastic constitutive model for asphalt concrete under cyclic loading using Schapery's elastic-viscoelastic correspondence principle. Their model also included damage parameter as an internal state variable in the form of a generalized microcrack growth law. The authors conducted a uniaxial cyclic tensile loading test and reported that their model satisfactorily predicts damage accumulation in asphalt concrete. They argued that the model also satisfactorily predicted the stress-strain behaviour of asphalt concrete under controlled stress and monotonic loading up to failure. However, they did not demonstrate the applicability of the model to predict deformation under more realistic repeated loading conditions.

With regard to permanent deformation, the assumption of linear visco-elasticity is a rough approximation. Recent findings from extensive laboratory test such as those conducted under the SHRP have indicated that asphalt concrete undergoes a considerable plastic deformation when loaded especially at higher temperatures [26]. Drescher et al [53] analysed the instantaneous strain in asphalt concrete in a laboratory study in terms of the plastic and the elastic strain components. Figure 4.6 shows the results of their analysis, which clearly indicates the existence of temperature dependent plastic strains at the beginning of loading. Ramsamooji et al [54] developed a rutting prediction model for asphalt concrete using plasticity theory.Their study indicated that for temperature up to 32^{o}C, plasticity accounts for 74% of the rutting while viscous creep and microcracking account only for 23% and 3% of the rutting respectively after 300,000 cycles in triaxial compression test.

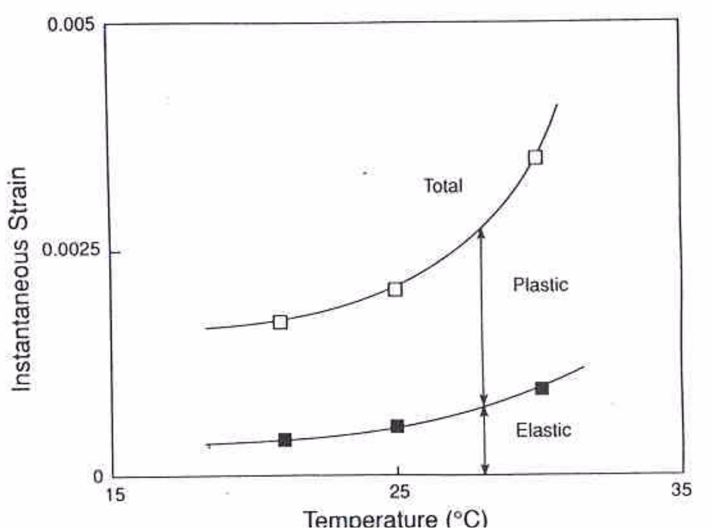


Figure 4.6 Elastic and plastic components of Instantaneous strain[53]

This plastic deformation can not be explained by the theory of visco-elasticity. Further more the deformation of asphalt concrete is both stress and rate dependent and therefore the stress-strain relationship is highly non-linear.Even the linearity at low temperatures is disputed by some researchers [55]. Sides et al [56]conducted an extensive study to characterize sand-asphalt mixture under compresive and tensile cyclic loading. They found that the elastic, plastic, viscoelastic, and viscoplastic strain components are simultaneously produced during loading process as illustrated in Figure 4.7. They proposed an elasto-viscoplastic model to characterize the sand-asphalt mixture. Thus, it can be concluded that the most realistic approach to characterize the response of asphalt concrete to arbitrary loading is perhaps the theory of elasto-viscoplasticity. The next section gives an overview of this theory and models developed based on it to characterize asphalt concrete.

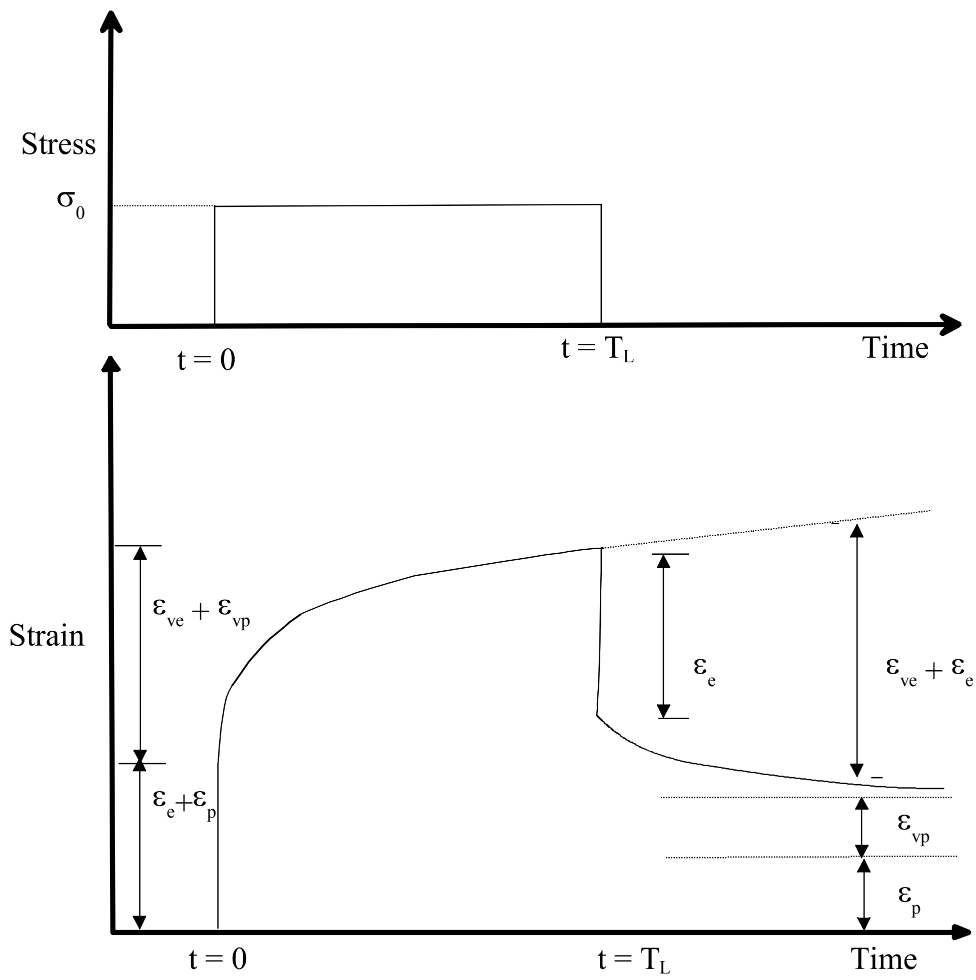


Figure 4.7 Components of asphalt concrete strain (ε_e =elastic strain, ε_p =plastic strain, ε_{ve} = viscoelastic strain, ε_{vp} = viscoplastic strain)

4.4 Elasto-Viscoplastic Models

As stated in the previous section the response of asphalt concrete to load involves elastic, viscoelastic, plastic, and viscoplastic components.Further the response depends on both the time history and load history (loading path) of the loading process, i.e. different results will be obtained for different loading paths and different duration of the process. Therefore, the theory of viscoelasticity alone can not adequately describe the deformation behaviour of asphalt concrete.

In theory of viscoplasticity, a distinction is made between an *elastic-viscoplastic* material and an *elastic/ viscoplastic* material [57, 58]. *Elastic-viscoplastic* materials show viscous properties in both the elastic and plastic regions while *elastic/ viscoplastic* materials show viscous properties in the plastic region only. In view of the analysis made by Sides et al [56], shown in Figure 4.7, asphalt concrete can be classified as an *elastic-viscoplastic* material as its viscous strain has both recoverable (elastic) and non- recoverable (plastic) components. The determination of yield criterion for *elastic-viscoplastic* material is extremely difficult as discussed by Naghdi and Murch [58]. The assumption of elastic/viscoplastic behaviour is an idealization that simplifies this problem.

Perzyna [57] represented the strain tensor for *elastic-viscoplastic* body using the following expression:

$$\varepsilon_{ij} = \varepsilon^{e}_{ij} + \varepsilon^{v}_{ij} + \varepsilon^{p}_{ij} \qquad 4.34$$

where $\varepsilon^{e}{}_{ij}$, $\varepsilon^{v}{}_{ij}$, $\varepsilon^{p}{}_{ij}$ denote the elastic, viscous, and plastic strain components respectively. The first two components of the strain tensor can be determined using the theories of elasticity and viscoelasticity. In order to determine the plastic component of the strain tensor and establish a general constitutive model for *elastic-viscoplastic* body, one needs to describe the yielding behaviour of viscoelastic materials.It is however very difficult to define the yield condition as illustrated in Figure 4.8. In the theory of elasto-plasticity, the plastic state produced by the loading path OP is represented by the same point P independent of the time in which the state represented by P is reached. If the material is *elastic-viscoplastic*, the plastic state may be reached at different points, say P_1 and P_2, depending on the time in which the load path is through.This property is caused by the viscosity of the material and the dependence of the load history on time. Further, by traversing the path OP in the same overall time but with different strain rates at the same points of the path different yield limits will be obtained.

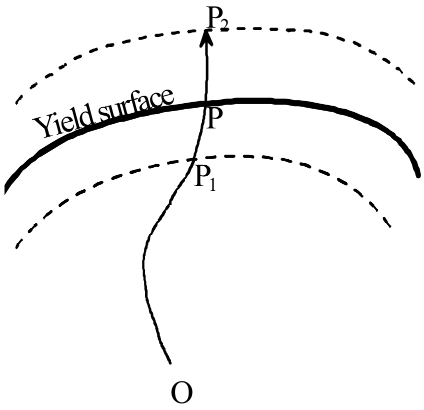


Figure 4.8 Load path and yield surface for viscoelastic material

Naghdi and Murch [58] introduced the notion of flow surface in order to describe the behaviour of *elastic-viscoplastic* materials.The flow surface is expressed as:

$$f = f(\sigma_{ij}, \varepsilon^{p}_{kl}, \kappa, \beta) \tag{4.35}$$

Elastic-viscoplastic state is determined by the condition f = 0, while the viscoelastic states correspond to the condition f < 0. The function f depends on the state of stress σ_{ij}, the state of plastic strain $\varepsilon^{p}{}_{kl}$, the parameter $\beta = \beta\ (e^{v}{}_{kl})$ which represents viscous effects and the strain hardening parameter κ. The strain hardening parameter depends, in turn, on the plastic strain and is defined in the same manner as in the theory of plastic flow describing isotropic strain hardening of the material. Based on this flow surface, mathematical conditions for loading, unloading and neutral states were defined. A rather complicated constitutive equation was also proposed for elastic-viscoplastic bodies by Naghdi and Murch [58]. But as commented by Perzyna [57], the variability of flow surface as a result of rheologic effects introduces certain indeterminacy, i.e., the position of the instantaneous flow surface and the point at which the plastic state is attained are unknown. Further, the direction of the plastic strain vector can not be uniquely determined. In the conventional theory of plasticity the direction of the plastic strain vector is perpendicular to the flow surface.

Owing to the difficulties mentioned above, many simplifying assumptions are needed in order to develop constitutive models of practical significance. One of these simplifying assumptions is the consideration of the material as *elastic/viscoplastic* instead of *elastic-viscoplastic*. Based on this assumption, Perzyna [57] provided a general framework which was later used by Desai and Zhang [59] to develop a viscoplastic model for geologic materials. Desai's model was applied by Alkhoury et al [60] to characterize asphalt concrete damage.

Perzyna's formulation for elastic/viscoplastic bodies assumes that the strain rate can be resolved into an elastic and inelastic part, i.e.,

$$\dot{\varepsilon}_{ij} = \dot{\varepsilon}^{e}_{ij} + \dot{\varepsilon}^{p}_{ij} \qquad 4.36$$

where the inelastic strain rate ($\dot{\varepsilon}^{p}_{ij}$) represents combined viscous and plastic effects. Since the material is assumed to have no viscous properties in the elastic region, the choice of yield criterion will be much simpler as compared to the case of elastic-viscoplastic materials. The initial yield condition, which was referred to as static yield criterion by Perzyna [57] is the same as the yield criterion in the classical theory of plasticity. Perzyna introduced a static yield function of the form:

$$F(\sigma_{ij}, \varepsilon^{p}_{kl}) = \frac{f(\sigma_{ij}, \varepsilon^{p}_{kl})}{\kappa} - 1 \qquad 4.37$$

where the function $f(\sigma_{ij}, \varepsilon^{p}_{kl})$ depends on the state of stress, σ_{ij}, and the state of plastic strain, ε^{p}_{kl}. The parameter κ (the strain hardening parameter) is defined by the expression:

$$\kappa = \kappa(W_p) = \kappa\left(\int_0^{\varepsilon^{p}_{kl}} \sigma_{ij} d\varepsilon^{p}_{ij}\right). \qquad 4.38$$

The following constitutive equation was then proposed for work-hardening and rate sensitive plastic materials by Perzyna.

$$\dot{\varepsilon}_{ij} = \frac{1}{2\mu}\dot{s}_{ij} + \frac{1-2\nu}{E}\dot{s}\delta_{ij} + \gamma\langle\Phi(F)\rangle\frac{\partial F}{\partial\sigma_{ij}} \qquad 4.39$$

In which:

$$\langle\Phi(F)\rangle = \begin{cases} 0 & \text{for} \quad F \le 0 \\ \Phi(F) & \text{for} \quad F > 0 \end{cases} \qquad 4.40$$

and where:

$s^{\cdot}_{ij}$ = deviatoric stress rate

$s^{\cdot}$ = mean stress

μ = shear modulus

E = Elastic modulus

δ_{ij} = Kronecer delta

γ = viscosity constant of the material

The above constitutive equation involves the assumption that the rate of increase of the inelastic components of the strain tensor is a function of the excess stress above the static yield criterion. This function of stresses above the static yield criterion generates the inelastic strain rate according to a viscosity law of the Maxwell type. Also the elastic components of the strain tensor are assumed to be independent of the strain rate.

4.5 Application of Viscoplasticity for Modelling the Behaviour of Asphalt Concrete

The previous sections highlighted the need for modelling the behaviour of asphalt concrete based on the theory of elasto-viscoplasticity. However, as already mentioned above, elasto-viscoplasticity theory is rather complicated and requires substantial effort in mathematical modelling and material testing. It is perhaps because of this complexity that not many attempts have been made, so far, to characterize asphalt concrete based on this theory. A review of the few elasto-viscoplasticity based modelling attempts is provided in the following paragraphs.

Sides et al [56]developed a comprehensive elasto-viscoplastic model for sand asphalt under compresive and tensile cyclic loading.The model incorporates the elastic, viscoelastic, plastic and viscoplastic strain components, i.e., the total strain was expressed as

$$\varepsilon_t = \varepsilon_e + \varepsilon_p + \varepsilon_{ve} + \varepsilon_{vp} \qquad 4.41$$

where:

ε_e = elastic strain, recoverable and time independent

ε_p = plastic strain, irrecoverable and time independent

ε_{ve} = viscoelastic strain, recoverable and time dependent; and

ε_{vp} = viscoplastic strain, irrecoverable and time dependent.

A series of repeated creep and creep recovery tests were made to determine the parameters of the model and quantify the various strain compnents.The elastic strain was obtained from the recovery curves and it was taken to be equal to the instantaneous decrease in the total strain which occurs at the moment the load is removed. The elastic strain component is then related to the stress using the elastic modulus. The plastic strain component was determined from the creep curve as a difference between the instantaneous strain which occurs at the beginning of loading and the elastic strain. The relationship between the plastic strain and stress was established using the plastic modulus.

The viscoelastic strain component, i.e., the recoverable part of time dependent strain was represented as a product of power function of time and a function of stress, the parameters of which were determined from the test data. The authors pointed out that the viscoelastic strain accumulated during the loading period was not completely recovered during the creep recovery phase and hence a residual viscoelastic strain was building up. This residual strain was found to depend on the ratio of recovery time to loading time and was modelled as a product of a function of number of load repetitions, a function of stress and a power function of time.It was found that for number of repetitions greater than 100, the residual strain was almost constant, indicating that for large values of the number of repetitions, the residual strain depends on the total effective duration of loading rather than on the total number of load repetitions. The viscoplastic strain component was obtained by subtracting all other components from the total strain. The viscoplastic strain was found to be a function of time, stress level, and number of load cycles.

Thus the authors have provided a framework for obtaining the components of the total strain. However, the modelling methodology is not strictly based on theory of viscoplasticity, i.e. no description of the yield condition and the stress history effects. But the model can be conveniently used to analyse laboratory test results and study the deformation properties of asphalt concrete. This approach is used in this thesis work to analyse results of triaxial creep and recovery test and to define a measure of resistance to permanent deformation as discussed in chapter 7.

Sousa and Weissman [42] enhanced the non-linear viscoelastic model developed earlier by Sousa et al by adding a rate independent plasticity component based on associative J2-plasticity with both isotropic and kinematic hardening. The material parameters of the model were determined from a series of tests consisting of constant height shear creep, shear frequency sweeps at constant height, uniaxial strain, hydrostatic (volumetric) and repetitive simple shear at constant height. The authors simulated a standard full depth, 380 mm thick pavement using finite element method and material parameters determined from the tests mentioned above and observed a unique relationship between rut depth and maximum permanent shear strain. They argued that the model captures important properties of asphalt mixes including dilatancy, hardening, plasticity and rate dependency. However, the testing effort required to determine the large number of parameters makes the model difficult to apply for practical purposes. Further, it has been indicated, from simulation of laboratory tests, that the agreement between model predictions and measured values were not so good. Particularly the plasticity model, while simple, failed to capture some of the mix properties [26].

Santagata and Virgili [61] analysed the elasto-viscoplastic behaviour of bituminous mixes and proposed a generalized rheological model. The rheological model represented the various components of the mechanical response of bituminous mixtures,i.e., the instantaneous elastic, time dependent elastic, instantaneous plastic, and time dependent plastic strain components. Figure 4.9 shows schematic of the proposed rheological model.

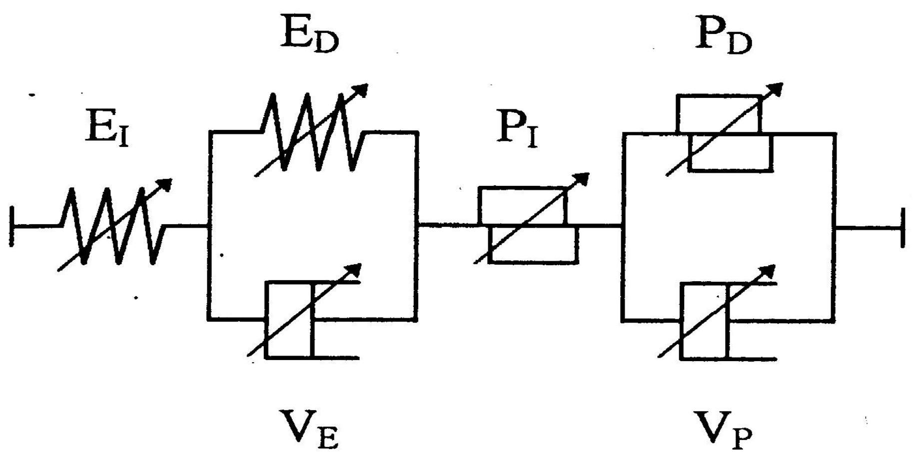


Figure 4.9 Rheological model[61]

In Figure 4.9, E_I is associated with the reversible and instantaneous response, E_D and V_E define the reversible time dependent component, P_I is linked to the irreversible instantaneous response, and P_D and V_P with time dependent irreversible one.

The model components are then mathematically represented by hereditary functions of time as shown below.

$$d_r = d_e + d_{ef}\left[\left(1 + \frac{t}{t_{ef}}\right)^{b_e} - 1\right] \tag{4.42}$$

$$d_i = d_p + d_{pf}\left[\left(1 + \frac{t_p}{t_{pf}}\right)^{b_p} - 1\right] \tag{4.43}$$

Where d_r represents the reversible deformation, d_i represents the irreversible deformation, t represents time, and d_e, d_{ef}, b_e, d_p, d_{pf}, t_{pf}, and b_p are material parameters to be determined from laboratory tests. The authors determined the eight material parameters from cyclic creep tests using non-linear regression and reported a very good agreement of the model prediction with experimental data. They further argued that the viscoplastic component of the strain is so significant that neglecting it would result in serous prediction error. However, the evolution of strain was expressed only as a function of time, i.e., it is not related to stress, which makes it difficult to apply the model in advanced analysis methods such as the finite element method. The temperature dependence of the material parameters were also not indicated.

Ramsamooj et al [54] developed a model for prediction of rutting in asphalt concrete which included elastic, viscoelastic and plastic components. The interaction between rutting and cracking was also treated.The plastic component of the deformation was modelled using the stress-dilatancy theory.The model utilizes multiyield surfaces and isotropic hardening. To understand the behaviour of asphalt concrete under cyclic loading a strain hardening law was developed, which enables calculation of the change in plastic modulus after each cycle. The hardening behaviour of asphalt concrete is now recognised to be of primary importance since it gives an indication of the resistance to rutting under repetitive loading. The viscoelastic component was calculated using linear viscoelasticity theory. The contribution of fatigue cracking to deformation was modelled using concepts of fracture mechanics. Several tests are required to determine the model parameters and study the behaviour in tension and compression, which include triaxial compression test, uniaxial tension test, hydrostatic compression test, fracture toughness test, indirect tensile strength test, creep test and cyclic load triaxial compression tests. The authors reported a good agreement between model prediction and test results. Figures 4.10 and 4.11 show some of the results from this study. It can be seen from this figures that the model tend over predict rutting for larger number of load repetitions. The many tests required to determine the model parameters also make it difficult to apply for practical purposes.

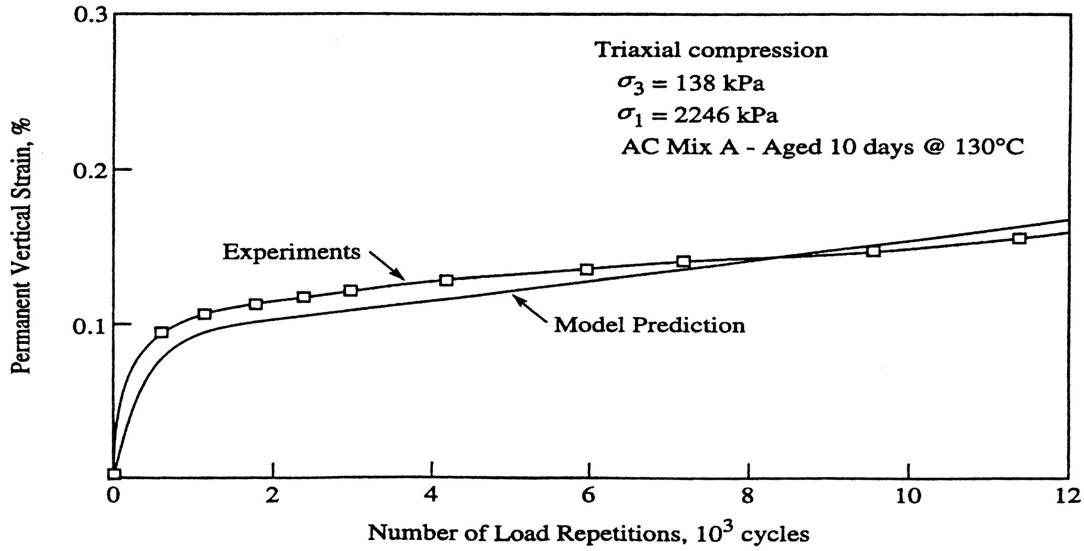


Figure 4.10 Permanent vertical strain versus Number of load repetitions[54]

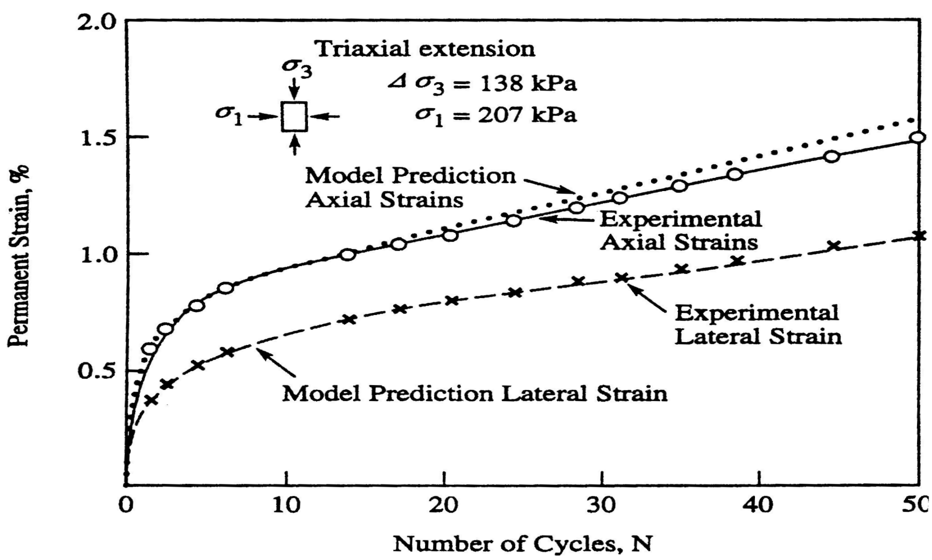


Figure 4.11 Model prediction versus experimental data for permanent strain versus number of cycles of load[54]

The work of Al-Khoury et al [60] represent one of the most advanced approaches to model the response of asphalt concrete pavements. The model is based on Perzyna's formulation of visco-plasticity mentioned earlier, in which the total strain is considered as the sum of elastic strain and viscoplastic strain. In analogy with the classical theory of incremental plasticity, the authors postulated a flow rule and a viscous flow surface. The viscous flow surface is considered to be the geometric locus of the states of stress corresponding to the same level of viscous flow and in the I_1-$J_2^{1/2}$ stress invariant space, it may be represented by a closed or open curve (Figure 4.12). A flow surface developed by Desai and co-workers [59] was employed to mathematically represent the viscous flow surface.The hardening and arrest of viscous flow observed in the primary creep phase of incremental creep test was modelled as the expansion of the viscous flow surface until it encompasses the applied states of stress (Figure 4.12).

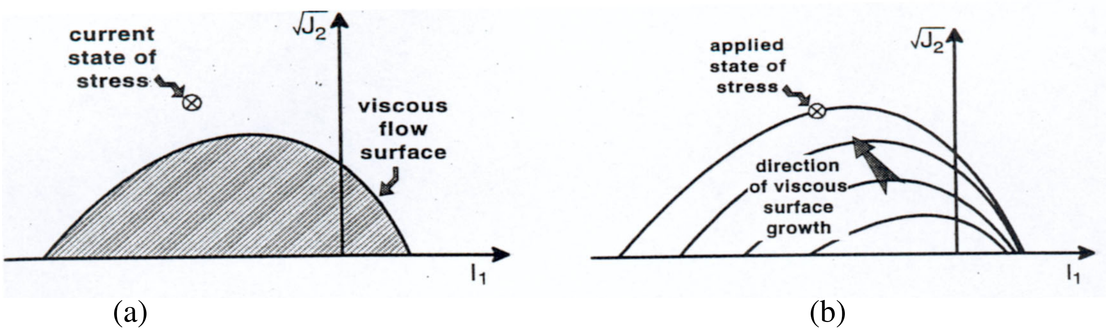


Figure 4.12 (a) A schematic of viscous flow surface (b) Viscous flow surface hardening[60]

For states of stress higher than a critical limit the viscous flow will not cease and the initiation of a secondary creep phase may be observed. This behaviour was represented by a *secondary creep initiation surface,* which was postulated to define the locus of all states of stress corresponding to the onset of secondary creep under multiaxial conditions. The hardening behaviour (the expansion of viscous flow surface) is assumed to occur only up to the size of the secondary creep initiation surface. If the applied state of stress is outside the secondary creep initiation surface, then an overstress is available to cause viscous flow. The authors also defined an ultimate strength and a response degradation functions to model damage under repeated loads. The model parameters were determined from laboratory tests, which included uniaxial tension and compression tests and shear tests. The model was then implemented in finite element program to simulate damage development in top layer under circular uniformly distributed load and also to simulate the response of a pavement to falling weight deflectometer. However, the tests used in calculation of the material parameters are monotonic and incremental creep tests and therefore do not simulate the repeated loading conditions to which pavements are normally subjected to in the field. The ability of the model to predict the performance of actual pavements has yet to be demonstrated.

There is an important shortcoming of most plasticity models, which form a major component of elasto-viscoplastic models, with regard to their ability to describe the behaviour of asphalt concrete materials. These models incorporate some kind of yield surface (or function) and assume that, for states of stress within the yield surface, no plastic (permanent) strain will develop. This implies that under repeated loading, plastic strain will develop only in the first cycle, provided the stress exceeds the current yield surface, and no more plastic strains will develop under subsequent loading cycles. This does not hold true for asphalt concrete and many geologic materials as plastic strains continue to accumulate under repeated application of the same stress level.

In an attempt to solve this problem some more sophisticated models involving multi yield surfaces and nested surfaces were developed. One of the concepts developed to tackle this problem is the bounding surface plasticity. The bounding surface plasticity concept have previously been applied to model deformation in granular pavement materials. A modified cyclic hardening model based on this concept is used to model permanent deformation in this thesis work and is described in chapter 7.

4.6 Micromechanical Approach for Modelling the Behaviour of Asphalt Concrete

The previous sections of this chapter reviewed phenomenological approaches for modelling the behaviour of asphalt concrete. In the phenomenological approach asphalt concrete is considered to be a continuum and the stress-strain relations are derived based on the principles of continuum mechanics. However, it is well known that asphalt concrete is a heterogeneous composite material. The distinct properties of aggregates and asphalt binder and their interface make the asphalt concrete a composite material with complicated stress-strain behaviour. Individual movement of particles has been observed during loading of asphalt concrete. There are normal and shear forces between the aggregates and displacements (translation and rotation) of the aggregates. For this reason some researchers have questioned the validity of the continuum mechanics approach to represent the behaviour of asphalt concrete and have argued in favour of micromechanical approach, which considers the properties of aggregates, the binder and the aggregate-binder interface separately. The aim of micromechanical approach is to find macro-level state variables from micro- variables such as contact forces, grain displacements and local geometrical characteristics.

The micromechanical approach, some times referred to as geometric modelling approach, is based on discrete element method (DEM). DEM was originally devised to model the behaviour of dry granular assemblies but later extended to modelling saturated sand and concrete [62]. The method has been applied to a number of problems such as failure analysis of materials, mechanical behaviour of granular media, mechanical modelling of jointed and fractured systems, etc. Simulation programs based on this method typically involve calculation of the movement of individual grains in cycles [63]. First the resulting forces and moments, which initially may be from gravity or external forces, on each grain are determined. In the next step the movement of grains, i.e., translation and rotation during small time increment is calculated using Newton's second law. Then at each contact point between two grains, new forces are determined from force- displacement laws, the sum of the forces on each particle is calculated and the next cycle is started.

Attempts to model asphalt concrete behaviour based on micromechanical simulation include the works of Chang and Meegoda[62], Attoh-Okine [64], Uddin et al [65], and Rothenburg et al [66]. Chang and Meegoda developed a simulation program named ASBAL by modifying program TRUBAL, which was developed earlier to model the behaviour of granular assemblies. The aim of the modification was basically to include the asphalt binder. Two types of contacts, aggregate-asphalt-aggregate and aggregate-aggregate contacts, were considered in the model. Figure 4.13 shows the micromechanical system used.

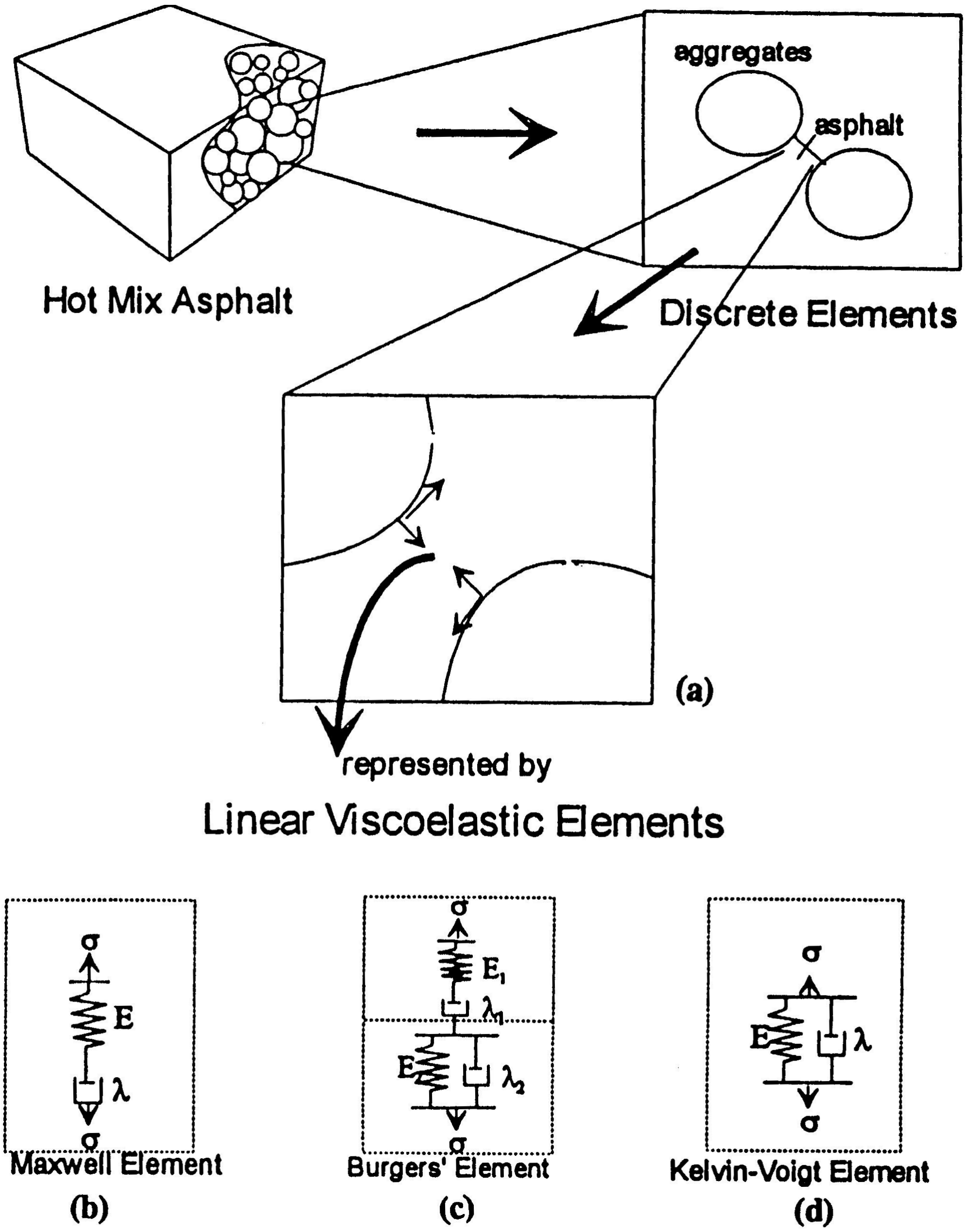


Figure 4.13 Mechanical system of Hot Mix Asphalt[62]

The asphalt binder was considered to be linearly viscoelastic. Burger's elements were used to model the aggregate-asphalt-aggregate contacts and part of aggregate contacts. The Mohr-Coulomb failure criterion was utilized to account for separation of asphalt coated particles due to rotation. An assembly of 512 particles was simulated and compared to a carefully conducted physical test with X-ray tomography results. The authors reported that the ASBAL model accurately predicted residual stresses of the laboratory sample after compaction, the initial modulus, stress levels through out the test, and number of contacts with in the matrix.

Attoh-Okine [64] introduced some modification in the geometric representation of the aggregate particles proposed by others in order to describe the aggregate-binder-aggregate contacts. However no simulation result was reported based on the proposed modification. Uddin et al [65] Used a micromechanical method of 'cells' to estimate the stiffness of asphalt mix and predict its response under traffic loads. The method of 'cells' was developed to determine the overall viscoelastic properties of composite materials, given the material properties of the individual constituents. This method has been applied to particulate composites as well as resin matrices and metal matrix materials. Uddin et al used a program called ASPHALT, which is based on micromechanical analysis, to estimate the creep compliance and creep modulus of an asphalt mixture and reported that the measured modulus of the mix was within 15 percent of the model prediction.

Rothenburg et al[66] studied pavement rutting problem using micromechanical modelling approach. They argued that rutting of asphalt concrete can be modelled using discrete element techniques that are able to simulate the interactions of individual bitumen coated aggregate particles. The particles were treated as elastic elements and the binder as linearly viscoelastic material. Creep displacement of the particles was modelled as a viscous flow of the binder, whose rate is affected by binder viscosity, film thickness, contact stress and other parameters. The binder within voids was considered as compressible Newtonian fluid. The authors performed simulation by solving Newton's equation of motion for individual particles and concluded that the result illustrated the effect of cohesive and frictional contact on creep behaviour and that rutting occurs when the number of frictional contacts is below a certain minimum.

The discrete element method (DEM) appears to be a novel tool to reconstruct the material using a very intuitive approach in three dimensions. However, research on the application of this method to model asphalt concrete behaviour seems to be just beginning. Simulation of real asphalt mixtures with various particle sizes and properties has yet to be done to demonstrate the applicability and advantage of the method.

4.7 Other Models and Permanent Deformation Equations

Several predictive equations and models have been proposed to characterize the permanent deformation behaviour of asphalt concrete. These predictive equations fall in to three general categories [26]: (1) empirical regression equations, (2) typical plastic strain laws, and (3) functional equations directly based on laboratory test results. Most of the equations are connected to a certain method of pavement analysis, such as elastic multilayer theory or finite element method and all of them involve some laboratory test to determine material properties.

The most widely used predictive equation for the development of permanent deformation has been the power law described as:

$$\varepsilon_p = CN^B \tag{4.44}$$

Where:

ε_p	= permanent strain
N	= number of load repetitions
C, B	= material constants

In some multilayer programs such as VESYS the constant C is considered to be a function of resilient strain. In the Strategic Highway Research Program's permanent deformation model C represents the permanent deformation after the first cycle, which is determined from an elasto-plastic formulation. Researchers of the Strategic Highway Research Program (SHRP) indicated that the constant B (S in SHRP's notation) is independent of the state of stress and depends mainly on the material type and condition such as density, moisture content, and internal structure. The SHRP established a relationship between the S and a parameter of creep compliance in shear (named m- value). The m- value was supposed to be a fundamental parameter for prediction of rutting of mixtures. However, using a controlled laboratory experiment intended for evaluation of SuperPave performance prediction models, Zhang [67] found that m-value was not reliable for prediction of rutting.

Another commonly used equation for development of permanent deformation is that of Lytton and Tseng [50], expressed as follows:

$$\varepsilon_p = \varepsilon_0 e^{\left(\frac{\rho}{N}\right)^{\beta}} \tag{4.45}$$

Where ε_0, ρ, β are material parameters. This equation represents a logarithmically work hardening material and the exponent β is a logarithmic rate of work hardening.

A number of such predictive equations are have been developed by various authors over the years. Reference [26] contains an excellent summary of these equations, which is reproduced in Table 4.1 below. As already mentioned most of these equations are the results of regression studies or curve fitting to laboratory test results and hence, their application is limited. Further more most of the equations are not in a form suitable for implementation in pavement structural analysis methods such as finite element method.

In summary, the review discussed in this chapter indicates that, although several attempts have been made and are being made to develop a model for permanent deformation of asphalt concrete, a comprehensive model that takes into account all aspects of the material behaviour has yet to be developed. The ability of many of the available models to predict performance of actual pavements under field loading conditions needs to be evaluated.

Table 4.1 Summarized overview of models and permanent deformation equations used by several authors [26]

Author	Pavement analysis	Permanent deformation equation	Variables	Laboratory test	Observations
Meyer, Haas (1977)	FEPAVE II FE layer strain method	$\varepsilon_p = f(\sigma_1, \sigma_3, T, V, N) \pm E$	ε_p = Axial permanent deformation σ_1 = Vertical stress σ_3 = Lateral stress T = Temperature V = Air voids N = Number of load applications E = error of estimate	Repeated load triaxial test	Measured values for rut depth on test road sections. Good agreement b/n measured and predicted.
Van de Loo (1976)	BISAR elastic layer theory	$\varepsilon_p = c\sigma N^a$	ε_p = axial permanent deformation c = Constant σ = Axial stress level (103.5 Kpa) N = No. of load applications a = Constant	Axial creep test	Basis of SHELL method. Generally overestimates rut depth
Kenis (1977)	VESYS Probabilistic linear visco elastic solution	$\varepsilon_p(N) = e\mu N^{-\alpha}$	$\varepsilon_p(N)$ = Permanent strain per pulse α = 1-S S = Slope of line on a log-log plot of permanent strain versus N e = Peak haversine load strain for a load pulse of duration d = 0.1 sec μ = IS/e I = Intercept	Uniaxial repeated load tests	Basis of the VESYS approach

Table 4.1 Summarized overview of models and permanent deformation equations used by several authors [26]

Author	Pavement analysis	Permanent deformation equation	Variables	Laboratory test	Observations
Franken (1977)		$\varepsilon_p(t) = At^B + C(e^{Dt} - 1)$ (High stress) $\varepsilon_p(t) = At^B$ (low stress)	**$\varepsilon_p(t)$ = Permanent strain** **A,B,C,D = Parameters** $A = 115(\sigma_1 - \sigma_3)\|E^*\|$ $B = 0.182 + 0.294(\sigma_{VM} - \sigma_{VL})$ **σ_{VM} = Maximum stress** **σ_{VL} = Plastic failure treshold** **t = Time** **σ_1 = Vertical sress** **σ_3 = Lateral stress** **E* = Modulus**	**Triaxial dynamic test**	**Method used to determine rutting propensity in mixes**
Verstraeten, Romain, Veverka, (1982	**ORN093 Elastic layer theory, layer strain theory**	$\varepsilon_p(t) = A\left[\frac{t}{10^3}\right]^B$ $= \frac{C(\sigma_1 - \sigma_3)}{\|E^*\|\left[\frac{t}{10^3}\right]^B}$	**$\varepsilon_p(t)$ = Permanent strain at time t (sec)** **A = a coefficient depending on the mix composition and on the experimental conditions (stress, frequency, temperature; it characterizes the susceptibility of the mix to rutting** **B = a coefficient varying between 0.14 and 0.37** **C = $f[V_b/(V_b+V_v)]$** **E* = Modulus of the mix** **σ_1 = Amplitude of vertical stress** **σ_3 = lateral stress** **V_b = volume of bitumen** **V_v = volume of voids**	**Triaxial dynamic tests**	**Acceptable correlation with rut depth measure in 16 in-service roads**
Huscheck (1977)	**BISAR, Elastic layer theory, layer strain theory**	$e_{irr} = c\sigma t^A$ $e_{irr}(\Delta t_1, T, t) = \frac{\sigma \Delta t_1}{[\eta(T, t)]}$ $\eta(T, t) = \frac{t^{1-A}}{cA}$	**e_{irr} = Permanent deformation** **c = Constant** **A = Consolidation characteristic** **σ = Stress level** **η = Viscosity** **T = Temperature** **Δt_1 = Time of loading**	**Uniaxial creep tests Cyclic load creep test**	**Asphalt mix is represented by a Maxwell element: spring and dashpot in series**
Thrower (1977)	**Viscoelastic theory separative method**	$\dot{e}_{ij} = \frac{\sigma_{ij}}{2\eta} \quad (i \neq j)$ $\dot{e}_{ij} = \frac{\sigma_m}{3\chi} + \frac{(9\sigma_{ij} - \sigma_m)}{18\eta}$	**$\dot{e}_{ij}$ = rate of deformation** **σ_{ij} = stae of stress** **σ_m = isotropic mean stress** **χ = Coefficient of volume viscosity** **η = coefficient of shear velocity**		

Table 4.1 Summarized overview of models and permanent deformation equations used by several authors [26]

Author	Pavement analysis	Permanent deformation equation	Variables	Laboratory test	Observations
Battiato et al (1977)	MOREL Viscoelastic theory Two layer viscoelastic incompressible system	$J(t) = J_1 t^{\alpha}$ $u_{ik}^{perm} = \frac{1}{\eta_s} g_{ik}(y, z)$	J(t) = Creep compliance function t = time J_1 = Shear creep parameter α = Slope of line on a log-log between J(t) and time u_{ik}^{perm} = Permanent deformation η_s = shear viscosity of Maxwell element in series $g_{ik}(y,z)$ = Tensor function	Uniaxial creep tests	Asphalt mix represented by a Maxwell model
Mahboub, Little (1988)		$\frac{\varepsilon_{vp}}{N} = a\sigma^b$	ε_{vp}/N = accumulated viscoplastic deformation per cycle σ = peak cyclic stress a, b = regression parameters	Uniaxial creep tests	
Tseng Lytton (1986)		$\varepsilon_a = e_0 \exp\left[-\left[\frac{\rho}{N}\right]^b\right]$	ε_a = permanent strain N = load cycles e_0, ρ, b = regression parameters	repeated load testing	
Lai Anderson (1973)		$\varepsilon_{vp} = a(\sigma)t^b$	ε_{vp} = viscoplastic strain t = time $a(\sigma) = b_1\sigma + b_2\sigma^2$ σ = creep stress b, b_1, b_2 = regression constants	Uniaxial creep	
Celard (1977)	ERDT/ ESSO Three layer elastic system	$\ln(\dot{\varepsilon}) = A + B\ln(\sigma_{vm}) + C\sigma_H + DT$	$\dot{\varepsilon}$ = rate of permanent deformation σ_{vm} = compressive vertical stress σ_H = compressive horizontal stress A, B,C, D = coefficients T = temperature	Dynamic creep tests	Developed iso-creep curves
Khedr (1986)	OSU model	$\frac{\varepsilon_p}{N} = A_a N^{-m}$	ε_p = Permanent strain N = number of load cycles A_a = material properties function of resilient modulus and applied stress m = material parameter	Multi step dynamic test	
Uzan (1982)		$\varepsilon_p(N) = \varepsilon_r \mu N^{-\alpha}$	$\varepsilon_p(N)$ = Permanent strain for N-th repetition ε_r = resilent strain N = number of repetitions α, m = characteristics of materials based on intercept and and slope coefficients	repeated load testing	

Table 4.1 Summarized overview of models and permanent deformation equations used by several authors [26]

Author	Pavement analysis	Permanent deformation equation	Variables	Laboratory test	Observations
Leahy (1989)		**Statistically derived predictive models for permanent strain, ε_p** $\varepsilon_p = f(T, \sigma_d, V, N, \eta_{as}, P_{Was})$	ε_p = **plastic strain** **T = temperature** σ_d = **deviator stress** **V = Volume of air** η_{as} = **asphalt viscosity** P_{was} = **effective asphalt content**	**repeated load and creep axial testing**	**Determined effect of mix variables on both ε_p and ε_r**

CHAPTER 5
TESTING FOR PERMANENT DEFORMATION CHARACTERIZATION OF ASPHALT CONCRETE

Planning any laboratory testing program involves definition of the objectives of the testing and development of testing procedures needed to obtain the data to fulfil the defined objectives. This study aims to investigate the effect of the composition of asphalt concrete mixture on its deformation behaviour and to model and predict the resistance to permanent deformation of the mixture. In order to accomplish this objective it is necessary to obtain data on permanent deformation response of asphalt concrete specimens with varying proportions of the component materials under realistic loading conditions.

The deformation of asphalt concrete mixture is a complex process. The binder is time and temperature sensitive material. The composite nature of the asphalt concrete introduces non-linear and stress sensitive characteristics. Thus deformation of asphalt concrete depends on temperature, rate of loading and the state of stress. Hence, it is necessary to test the material under conditions of stress, loading rate and temperature that best simulates the field conditions. The deformation is comprised of four components; namely, elastic component (recoverable time independent), viscoelastic component (recoverable time dependent), plastic component (irrecoverable time independent), and viscoplastic component (irrecoverable time dependent). In order to be able to select parameters that can be used as a measure of resistance to permanent deformation of asphalt concrete, it is felt necessary to decompose the total deformation in to these components and investigate the development of each of the components with increasing number of load applications. Thus a test method that would allow the decomposition of the total strain into its components has to be adopted. This chapter provides a review of the available testing methods with their pros and cons as a background for the test method selection and describes the specimen preparation and testing procedure adopted in this study.

5.1 Test Methods

Various test methods and procedures have been developed and used by researchers over several decades to characterize the permanent deformation behaviour of asphalt concrete. These test methods may in general be classified in to five types.

- Uniaxial stress tests- unconfined cylindrical specimens in creep, repeated or cyclic loading.
- Triaxial stress tests- confined cylindrical specimens in creep, repeated or dynamic loading.
- Diametrical tests- cylindrical specimens in creep or repeated loading
- Shear stress tests -cylindrical specimens in shear creep or repeated loading
- Wheel track tests- slab specimens or actual pavement cross sections.

5.1.1 Uniaxial and Triaxial Creep Tests

Creep test mostly involves the application of static load over a specified period of time and measurement of the resulting strain. This is considered to be the simplest way to investigate the permanent deformation characteristics of bituminous mixtures and is the most widely used test method for determining material properties because of its simplicity and the fact that many laboratories have the necessary equipment and expertise. Researchers at the Shell Laboratory in Amsterdam conducted extensive studies using the unconfined creep test as the basis for predicting rut depth in asphalt concrete[68,69]. It was reported that the creep test must be performed at relatively low stress levels (with in the linear range of the material) to obtain good comparisons between rut depths observed in test tracks and those calculated using creep test data. The need to use stress levels with in the linear range has been attributed to the fact that the loading time in the field is small compared to the loading time in the creep tests.

Strain, measured as a function of the loading time at a fixed test temperature, is the usual out put of the creep test. Results of the creep test, when expressed as relative deformation (measured change in height divided by the original height), are found to be independent of the shape of the specimen and of the ratio of height to diameter, provided the specimen's ends are parallel, flat and well lubricated[26].

The relevance of this test to the repeated load situation which actually occurs in the road is, however, questionable. Monismith and Tayebali [70] compared the response of three mixes containing conventional and modified binders under both creep and repeated loading. For creep loading at 37 ^{o}C and a confining pressure of 207KPa, difference among the mixes were not discernable. Differences were observed, however, in the result of repeated load testing suggesting that the repeated loading test may be more appropriate than the creep test to evaluate the permanent deformation characteristics of asphalt mixes. The results are shown in Figure 5.1. SHRP's results also indicated that more deformation occurs in repeated loading than in creep loading for the same materials and other test conditions as shown in Figure 5.2. The static creep tests do not capture the elastic rebound that normally occurs during unloading and the time dependence of the material, i.e., the effect of frequency of loading.

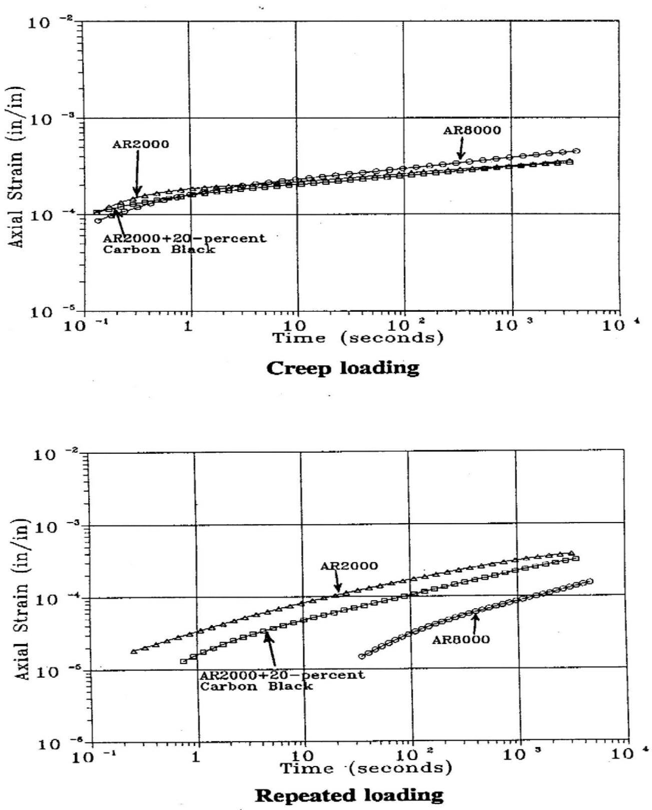


Figure 5.1 Comparison of three mixes in triaxial creep and repeated loading at 37^{O}C, 207Kpa confining stress[26]

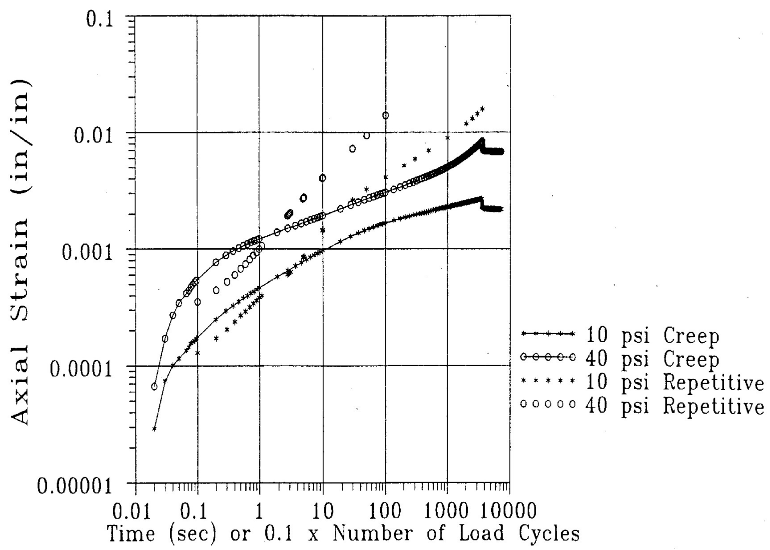


Figure 5.2 Effect of mode of loading on accumulation of strain at 40°C, unconfined conditions[26]

5.1.2 Uniaxial and Triaxial Repeated Load Tests

Repeated load test have been used to characterize permanent deformation response under more realistic conditions than those of the creep test. A variety of loading systems have been used to measure the response of asphalt concrete mixes to repeated loading. These loading systems range from relatively simple mechanical or pneumatic systems to more complex electro-hydraulic systems. The more sophisticated modern systems are typically capable of applying repeated axial and lateral stress pulses of any desired shape in phase with one another, either in tension or compression. They are also capable of incorporating rest periods between stress pulses. To enable testing at a specified temperature, these systems are also fitted with temperature control mechanisms. Figure 5.3 illustrates uniaxial and triaxial repeated load tests. Linear Variable Differential Transducers (LVDTs) are used to measure vertical and horizontal deformations, from which, permanent strain, resilient modulus, and poisson's ratio, as a function of the number of load repetition, can be calculated.

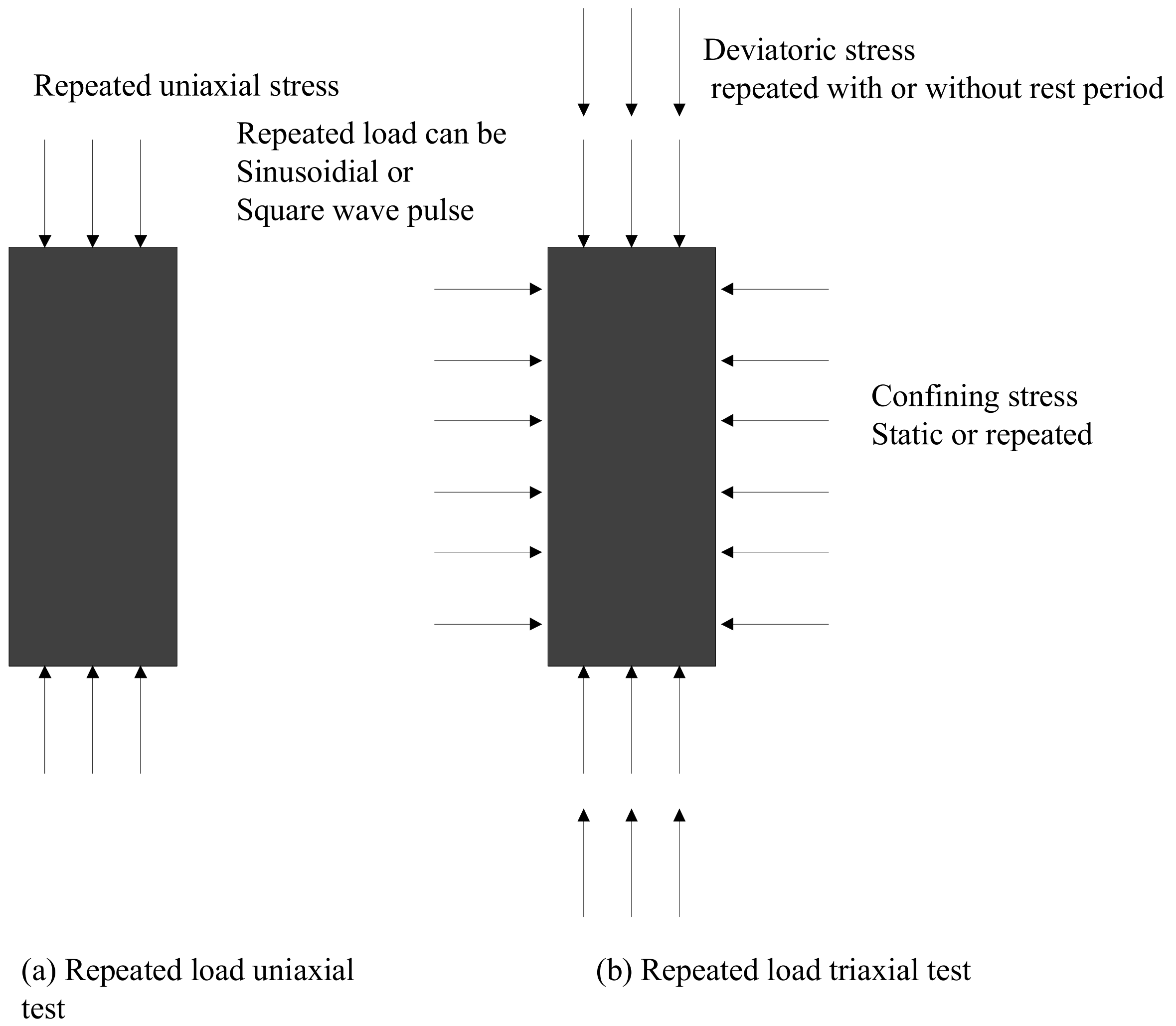


Figure 5.3 Schematic representation of repeated load uniaxial and triaxial tests

It has been argued that the permanent strain which gradually accumulates under repeated loading, is essentially a creep phenomenon, i.e., it is the loading time rather than the number of load applications which controls the permanent strain [71]. However, the pulse shape and duration was found to greatly influence the measurements[26], thus there is a need to duplicate as closely as possible conditions existing in the actual pavement. Repeated load tests are usually carried out with confining stresses. Several studies have demonstrated that the confining stress has a significant effect on the measured permanent deformation[26,71]. Also tests conducted with different confining stresses in this thesis work provided evidence that this is the case. Most repeated loading test are conducted with static confining pressure but similar effects have been reported when cyclic confining pressure is applied in phase with the vertical stress. It was also found that rest period between load cycles does not affect the basic permanent strain against time relationship where the time refers to the time when the material is actually being loaded.

In triaxial stress tests a wide range of stress states can be created by varying axial and confining pressures. Some of the states of stress include shear components and most of the states of stress that are encountered within pavements can be duplicated. However, the conventional triaxial test on cylindrical specimens is not truly triaxial as the two principal stresses are equal.

Some attempts have been made to study the behaviour of asphalt concrete using truly triaxial equipment.These equipment are capable of applying normal stresses along three mutually perpendicular axes independently. Agostinacchio et al [73] conducted experimental investigation on asphalt wearing course material using equipment capable of applying loads along three mutually perpendicular axes to cubical speciemens.The authors developed a yield function for the asphalt material from the data obtained by applying various stress increments along the three axes. Merzlikin [74] described a device for cyclic triaxial test of cubic asphalt concrete specimens, which is similar to the one used by Agostinaccho et al. These types of triaxial tests may reproduce the stress conditions in real pavements more closely but they are also more complicated and not yet standardized. Furthermore, in all types of axial or triaxial tests, care must be exercised in aligning and lubricating the ends to obtain uniform states of stress.

5.1.3 Diametrical Tests

Diametrical test are primarily used for measurement of the stiffness of asphalt concrete specimens. This method employs an indirect tension device, which produces tensile stress along the vertical diameter of the specimen. Thus the measured resistance to load is largely a function of the asphalt binder and the aggregate has less influence as compared to other tests such as the triaxial test. Accordingly the indirect tension test may be better suited for repeated load testing associated with modulus measurement than for the long time periods associated with permanent deformation measurements. Figure 5.4 illustrates stress distribution in indirect tension test. Major problems that make diametrical testing unsuitable for measurement of permanent deformation behaviour of asphalt concrete specimens include:

1. The state of stress is not uniform and it strongly depends on the shape of the specimen. At high loads or temperatures, permanent deformation produces significant changes in the specimen shape, which in turn significantly affects the state of stress and the result of measurements.
2. It has been recognised that shear stresses contribute significantly to permanent deformation and they are known to cause non linear behaviour. Because of the non uniform field of shear

stress that result in diametrical testing, deformation measurements can not be related to specific stress level.

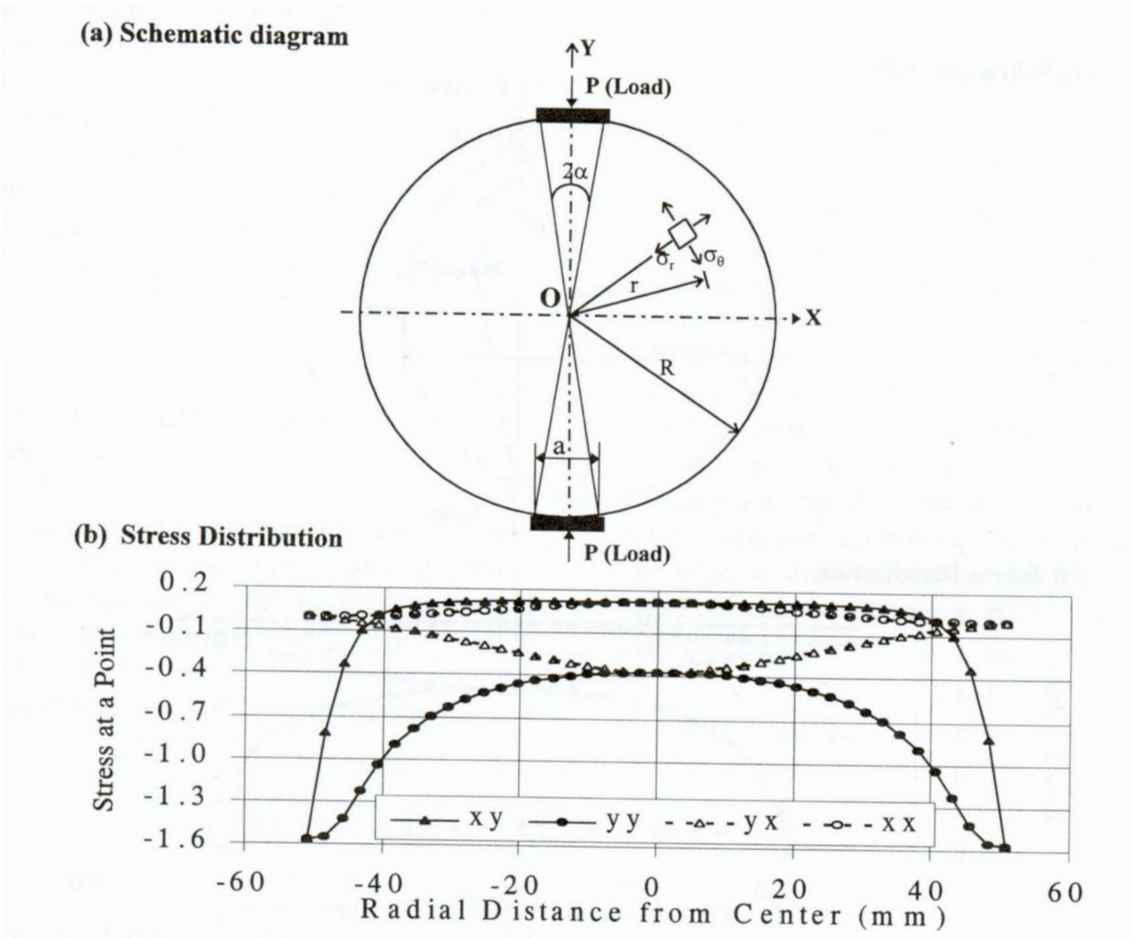


Figure 5.4 Stress distribution in indirect tension test based on Hondros solution[75]

5.1.4 Shear Stress Tests

In most triaxial test equipment, principal stresses are fixed in one direction, and only an interchange of principal stress directions can take place. This is considered to be one of the limitations of conventional test methods for asphalt concrete. Rotation of the principal stress axes can only be achieved in equipment in which shear stresses can be applied to the specimen surfaces. A laboratory simulation of principal stress rotation involves subjecting hollow cylindrical specimens to axial load, torque about central axis, and to internal and external radial pressures. Due to the symmetry of the hollow cylindrical specimen, normal and shear stresses are uniformly applied. Figure 5.5 schematically illustrates the states of stress in this test.

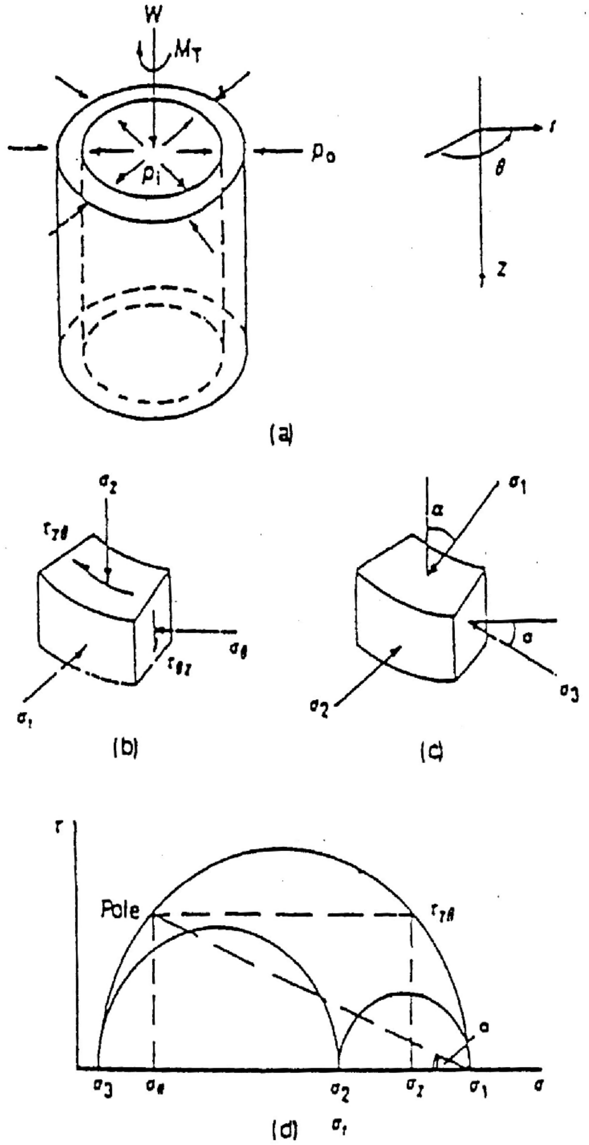


Figure 5.5 Idealized stress conditions in hollow cylinder test: (a) loading; (b) stresses on wall element; (c) principal stresses on wall element; (d) Mohr circle representation of principal stresses[26]

Crockford [72] used this test to study the effect of principal stress rotation on measured permanent deformation response of asphalt concrete and reported a significant difference between results of tests performed with and without principal stress rotation. The equipment required to conduct this kind of tests is too complex to be used for routine applications but it is considered to be useful as a research tool.

Simple shear tests are frequently used in geotechnical engineering. They approximate field conditions that are characterized by a pure shear stress state. Simple shear test is the simplest test that permits controlled rotation of the principal axes of stress and strain.This test has not been widely used for measuring asphalt concrete properties but it may be suitable for investigating the rutting propensity of asphalt concrete as rutting is thought to be predominantly caused by plastic shear flow. An example of the use of simple shear test was reported by Monismith and Tayebali [70] who used it to compare the creep response of cored specimens obtained from field pavements with the response of specimens compacted with kneading compactor.

A more sophisticated shear testing device was developed by SHRP's researchers. The device, generally referred to as Superpave Shear Tester (SST), is capable of performing the following shear tests on asphalt concrete specimens:

- simple shear test at constant height,
- repeated shear test at constant stress ratio,
- shear frequency sweep test at constant height, and
- repeated shear test at constant height

In the simple shear test at constant height, a shear stress is applied while maintaining the specimen at constant height and the resulting shear strain is measured. The shear frequency sweep test at constant height is a strain controlled test, in which a horizontal shear strain is applied at various frequencies. The test results can be used to obtain the values of the complex shear modulus and phase angle as a function of frequency. Repeated shear stress tests involve, as the name indicates, repeated application of shear stress for specified duration and measurement of the resulting shear strain and they can be conducted either in constant stress ratio or constant height mode.

The SST is also capable of conducting uniaxial compression test and volumetric tests. The former involves the measurement of axial strain resulting from application of axial stress while lateral strain is kept constant by application of confining pressure and the later involves measurement of both axial and lateral strains resulting form the application of hydrostatic pressure. The SST results are primarily used for prediction of permanent deformation performance of asphalt mixtures. However, the repeatability of the SST, particularly the uniaxial compression test and the volumetric tests are found to be not so good[76].

5.1.5 Wheel-Tracking Tests

Laboratory wheel- tracking tests have been used to evaluate the rutting resistance of asphalt aggregate mixtures in many European countries. The wheel-tracking testers typically measure the rut created by repeated passage of a wheel over prismatic asphalt concrete samples. The laboratory simulation of the rutting phenomenon must approach actual pavement stress conditions, for the results to have practical importance in mix design applications. The specimen can be compacted in the laboratory or may be cored from an actual pavement. Rutting is measured by relative percentage reduction in thickness of the specimen in the wheel path.Fig 4.4 shows a wheel tracking equipment used by the Laboartoires des ponts et Chausees(LCPC) of France for practical mix design.

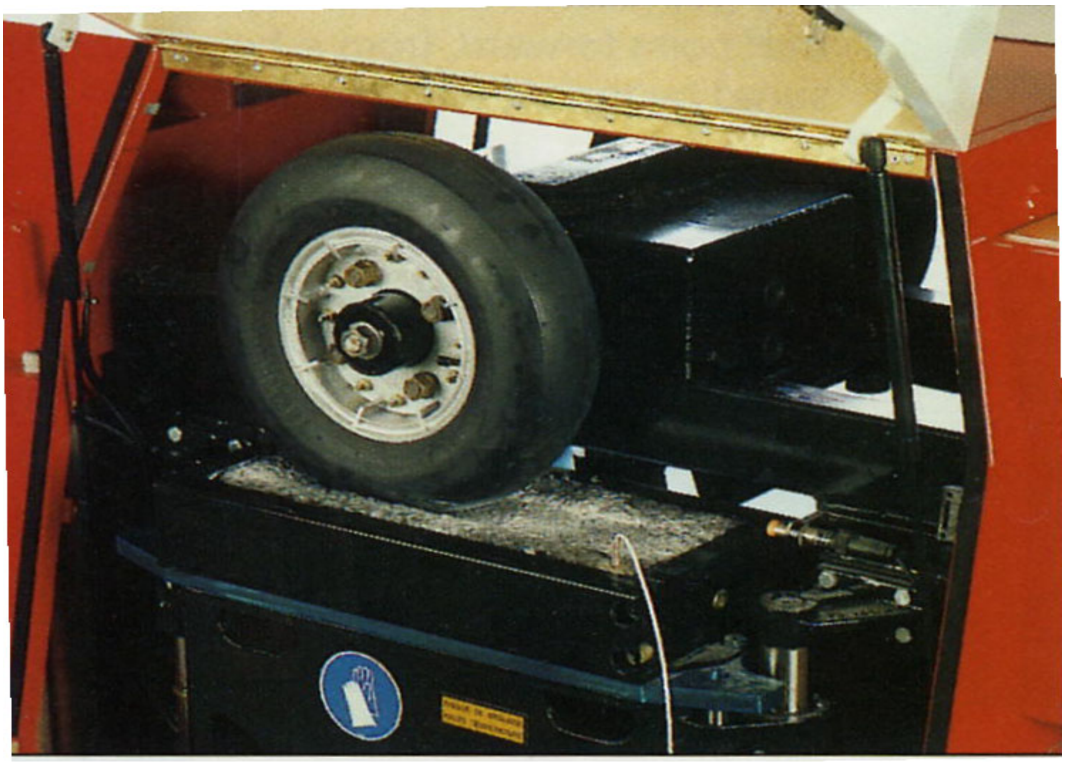


Figure 5.6 Wheel tracking test equipment[26]

The disadvantage of the wheel-tracking type of tests is that they are essentially empirical strength tests in the sense that their out put results only in accept or reject decision based on the experience of a particular agency and the correlation of the tests to real pavement conditions. That is they do not measure a degree of performance and do not allow economic comparisons of alternative materials. Even though the out puts of the test can be backed by performance data from real pavements, the experience is only applicable to the materials and environmental conditions tested. Further more, the correlation of wheel tracking test results to field performance appears to be poor as indicated by a major study which involved evaluation of several types of wheel tracking devices by the US federal highway administration[77]

5.2 Selection of Test Method

The previous sections of this chapter reviewed the various test methods used to characterize asphalt concrete mixtures. Several factors have to be considered in selecting a test method, which include the following.

- The ability of the test method to reproduce the *in situ* stress conditions as closely as possible.
- The ability of the testing method to fulfil the defined objectives of the study, i.e., the out puts of the test should enable an analysis of the effect of variables (factors) considered in the study on the selected parameters of the study. For this particular study, for instance, the out puts of the test should show the effect of the selected factors, i.e., binder content, void content, and temperature on the permanent deformation resistance of asphalt concrete, as indicated by their effect on parameters selected as a measure of resistance.
- The test method should be repeatable and reproducible.
- The test method should be as simple as possible.

Understanding the stress conditions to which pavement materials are subjected is necessary to be able to judge how closely can a certain test method reproduces the in situ stress conditions.Figure 5.7 shows a typical pavement element and the stresses acting on it. The stresses change with time as the wheel passes over and the variations of vertical, horizontal, and shear stresses are shown in Figure 5.8.

The contact stress distribution between tire and pavement was generally assumed to be uniformly distributed vertical pressure over a circular area. This is analogous to modelling the tire as structure less balloon in contact with smooth surface. In reality the contact stress distribution is more complex due to the rigidity of the rubber and the internal structure of the tire. A method for measurement of contact stress between the tire and pavement was developed by CSIR in South Africa using what is called the Vehicle-Road Surface Pressure Transducer Array system as reported by De Beer et al[78]. This system involves measurement of forces acting on the pavement surface in three dimensions using an array of pins, one row of which is instrumented. Measurement are taken as the wheel rolls across the pins. The contact stress distribution was found to be highly non-uniform in vertical direction and significant horizontal components also exist. The three dimensional stress distribution produces significant tensile stress at the edge of the tire. De Beer et al [78] compared the magnitude of the strain energy of distortion, octahedral shear stress, and the bulk stress within the asphalt layer resulting from the application of the conventional uniformly distributed contact stress and the more realistic contact stress distribution found from measurement using finite element analysis. They reported a significant difference.

Weissman [79] investigated the influence of tire-pavement contact stress distribution on the development of distress mechanisms in pavements based on simulation study using a layered-elastic software program. He compared stresses resulting form application of uniformly distributed pressure on circular area and those resulting from application of vertical contact stress distribution reported by De Beer et al [78]. He concluded that the axisymmetric state resulting from assumption of uniformly distributed pressure on circular area leads to an underestimation of stresses that develop in pavements. In particular he stated that, where ruts are concerned, the localized stresses lead to larger flows and hence to increased ruts and the application of uniform pressure on circular area as an approximation of the load leads to different predictions of accumulation of permanent deformations.

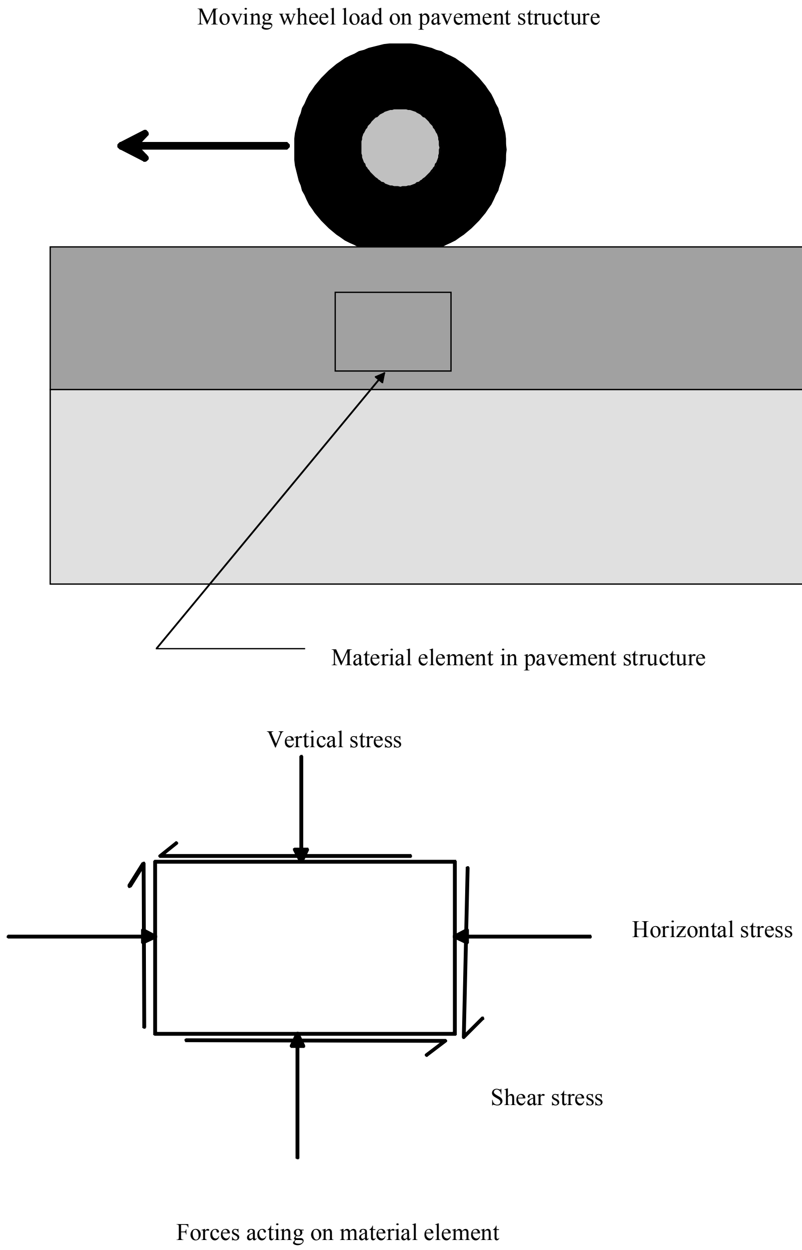


Figure 5.7 Stresses acting on pavement element

The foregoing discussion indicates that none of the commonly used test methods can reproduce the in situ stress conditions accurately. It is also clear that several tests are needed for proper characterisation and constitutive modelling of asphalt concrete as the out puts of a single test are usually not enough. However, the repeated load triaxial test appears to come closer to the in situ condition than other commonly used test methods.The triaxial test has been used to characterize soils and other pavement materials for decades and many laboratories have the required expertise to conduct the test. The results of repeated load triaxial test were also found to correlate well with wheel tracking tests and field performance. Further, the repeated load triaxial test has been found to be appropriate for studying the effects of asphalt concrete composition, such as air voids, on its deformation properties as reported by Bouldin et al[80], thus meeting the objective of this study.

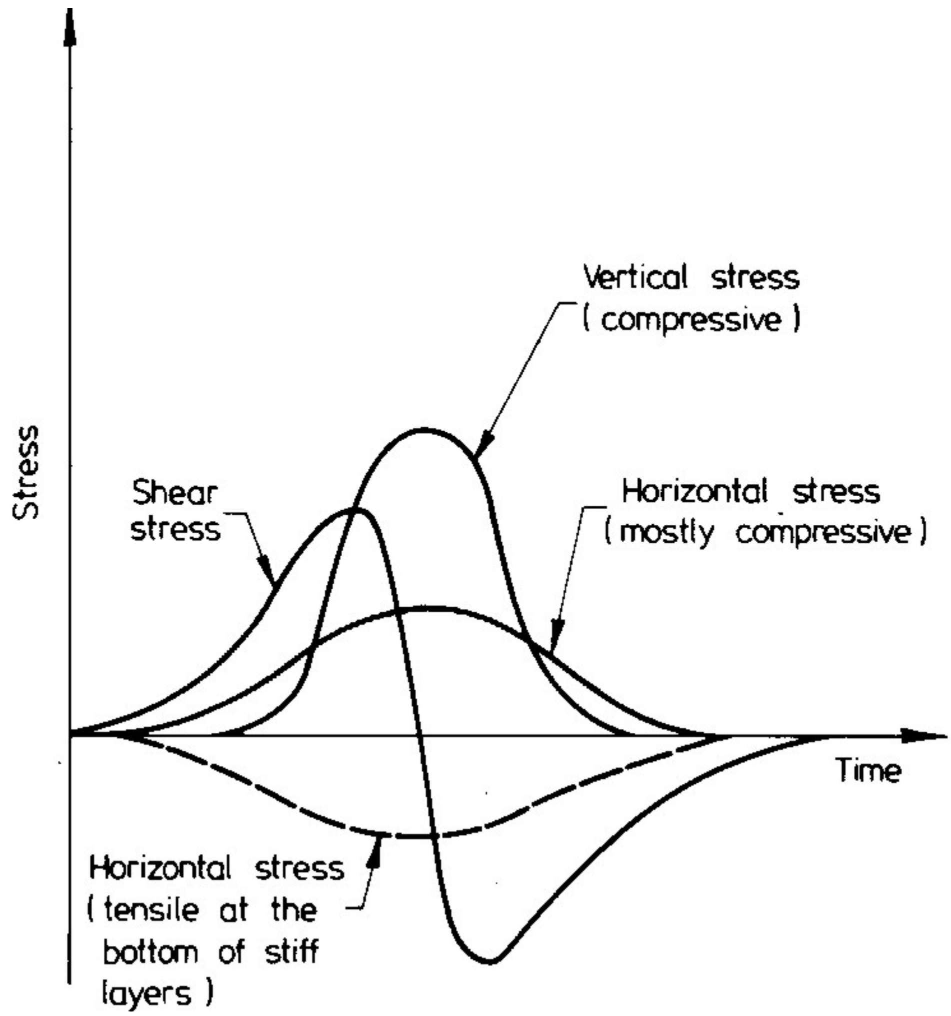


Figure 5.8 In situ stresses caused by moving loads[71]

Consideration of the factors outlined in the preceding paragraph has led to the choice of the repeated load triaxial test to study the permanent deformation characteristics of asphalt concrete mixtures in this study. A simpler version of the repeated load triaxial test, which involves the application of cyclic deviator stress under constant confining stress was adopted. The deviatoric stress is applied in the form of a continuous sinsuoidal wave. Figure 5.9 illustrates stresses and strains in repeated load triaxial test.

A variation of the repeated load triaxial test, repeated triaxial creep and recovery test, is selected to study the various components of deformation. In this test the deviatoric stress is applied in the form a block (square) wave. The results of this test allow the decomposition of strain into elastic, plastic, viscoelastic and viscoplastic components as described in detail in chapter 7. Figure 5.10 illustrates the form of stress and the resulting strain in a triaxial creep and recovery test.

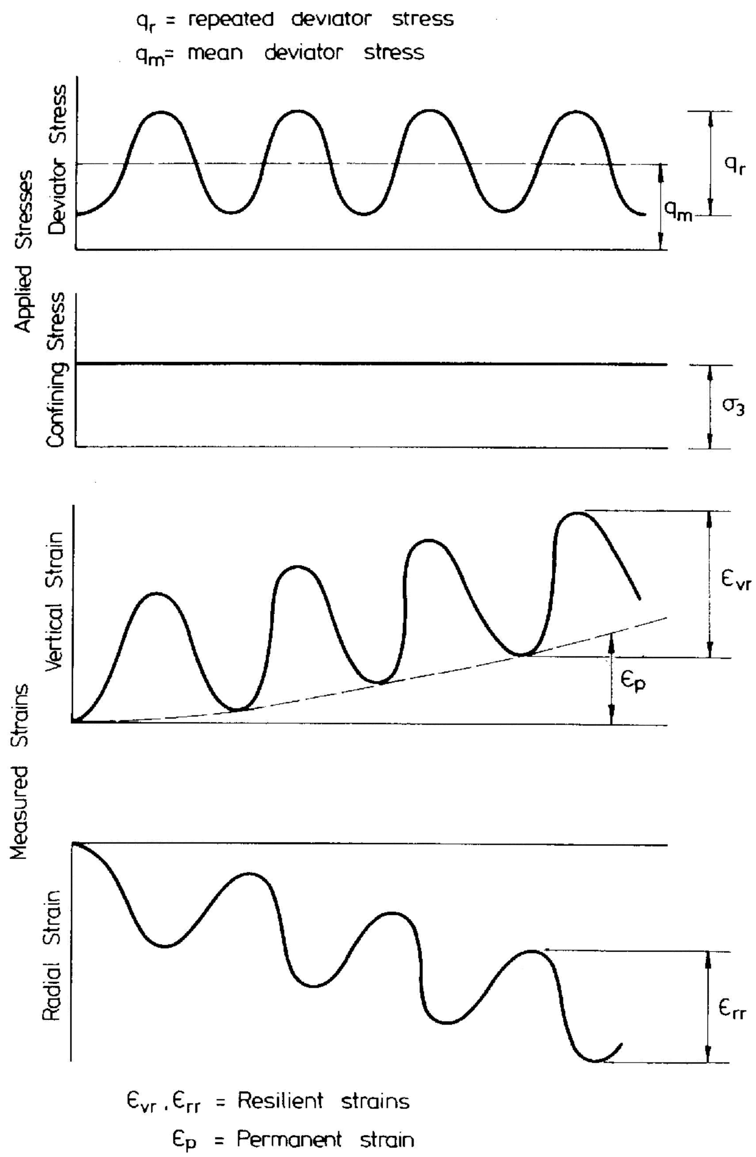


Figure 5.9 Stresses and strains in repeated load triaxial tests[71]

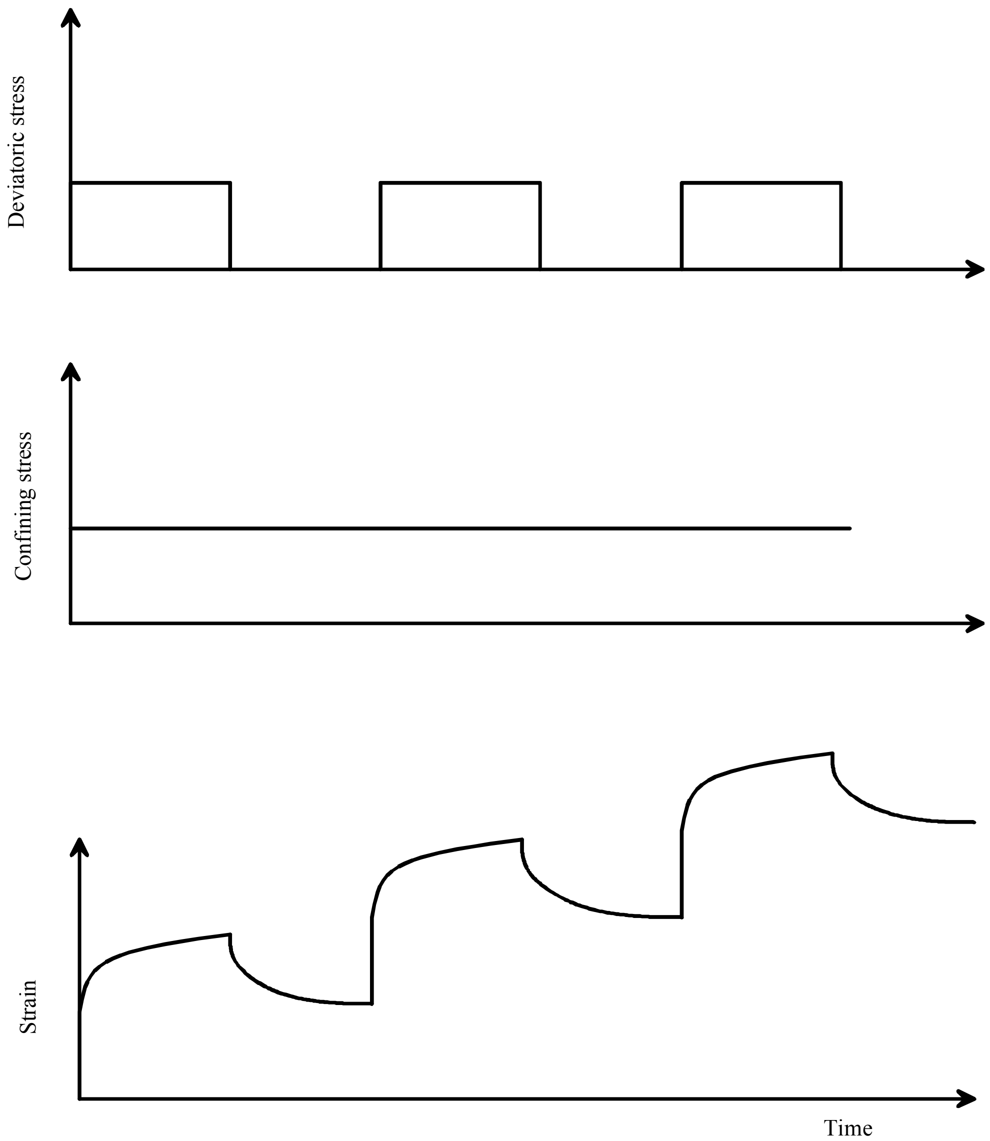


Figure 5.10 Stresses and strains in triaxial creep and recovery test

5.3 Materials

The material used in this study consists of an asphalt mixture, which was designed using Marshal method with optimum binder content of 4.7%. To study the effect of binder content on permanent deformation of the mixture, three levels of binder content, 4.0%, 4.7% (optimum), and 5.4% were employed. The mixtures were compacted to three levels of void content using California Kneading compactor.The aggregate consists of 51% crushed rock, 44% natural gravel and sand, and 5% lime stone mineral filler. Table 5.2 gives the properties of the aggregate material. Aggregate gradation curve was produced by combining several fractions using trial-and-error procedure. The gradation curve was based on gradation requirements for Ab 11 mixture, according to Norwegian specification. Ab 11 mixture is an asphalt concrete mixture with nominal maximum size of 11.2 mm commonly used in Norway. The gradation was slightly modified by mainly reducing the proportions of material less than 2 mm in size. This was found necessary because the Ab 11gradation produced a very dense material and would not allow compaction of the specimens to the required three levels of void content. The gradation curve also passes below the restricted zone on the gradation chart proposed by the SHRP for asphalt concrete with nominal maximum size of 12.5mm, which is equivalent to Ab 11. Figure 5.11shows the gradation curve.

Table 5.2Aggregate properties

Type	Specific gravity	Los Angeles Abrasion (LAA) Value (%)	Crushing strength (S value,%)	Flakiness Index (According to Norwegian standards)
Crushed rock	3.05	13.5	39.8	1.35
Natural gravel	2.74		36.1	1.27

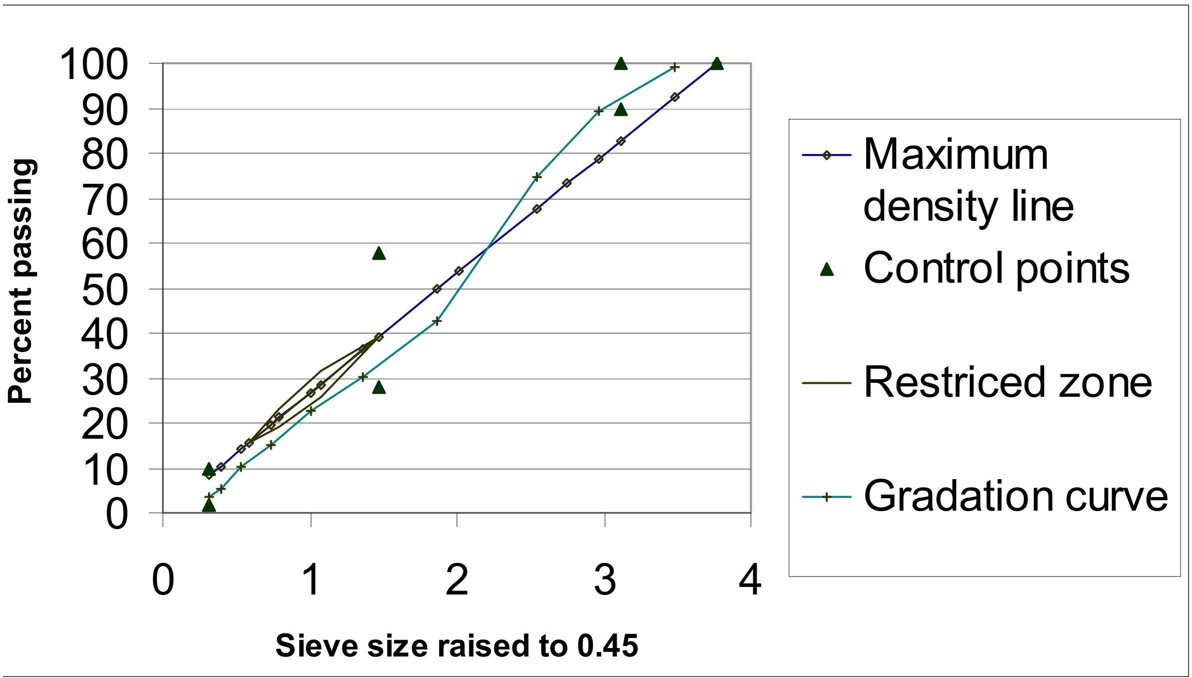


Figure 5.11 Aggregate gradation curve

The binder used in this study was a penetration grade B85 unmodified bitumen. The properties of the binder are given in Table 5.3.

Table 5.3Binder properties

Property	Value	Unit
Penetration at 25°C	75.2	1/10 mm
Dynamic viscosity at 60 °C	230	Pa.s
Dynamic viscosity at 135°C	364	mm^2/s
Softening point	45.9	°C

5.4 Specimen Preparation

5.4.1 Mixing

Specimens were produced from material mixed in the laboratory. About 4 kg of material was used to produce specimens 100mm in diameter and about 200mm in hight.Both the binder and aggregates were heated in an oven for about 2 hours to bring them the mixing temperature of 145 °C. After mixing the material was apportioned in to five portions of about 800 gms and conditioned at the compaction temperature for about one and half hours, the compaction temperature varied between 110 and 140°C.

5.4.2 Compaction

One of the key issues in specimen preparation is compaction. Compaction has a profound influence on performance, both in the laboratory and in the field. Compaction transforms the mix from its very loose state in to a more coherent mass permitting it to carry loads. The efficiency of any compactive effort depends on the internal resistance of asphalt concrete, which in turn depends on the aggregate interlock, friction resistance and viscous resistance of the binder. In principle, a selected method of compaction should reproduce the field compaction as closely as possible. Further, the compaction should produce specimens of uniform density.But commonly available compactors such as the Marshal compactor do not fulfil these requirements. As a result attempts have been made in the last several decades to develop other compactors, which are supposed to reproduce field compaction and produce relatively uniform densities.

Historically there have been three compaction methods that have been used for specimen preparation for both asphalt mixture design and other laboratory tests. These are the impact compaction, kneading compaction and gyratory compaction. Impact compaction is the oldest method of laboratory compaction and it includes the Proctor hammer, which was adopted in early mixture design procedures such as the Hubbard and Field method, and the Marshal Hammer, which is being used in Marshal mix design method. It has been pointed out by many researchers that impact compaction does not simulate field compaction[81,82].

Kneading compaction was developed in connection with the Hveem mixture design methods. Kneading compaction applies force through a roughly triangular shaped foot that covers only a portion of the specimen's face, as a result of which kneading action is produced. This is more realistic than the impact compaction in that it attempts to simulate both the compresive and shearing action produced by field compactors. Kneading compaction has been used by several states in the western USA, but it has not been commonly used in the rest of the world.

Gyratory compaction was developed in the 1930's in Texas, USA[81]. It was further improved by the US Army Cops of Engineers in the 1950's and 1960's. The gyratory compactor has also been used by the Central Laboratory for Bridges and Roads (LCPC) of France since 1970's. More recently the SHRP adopted the gyratory compactor in its volumetric mixture design procedure and it is now being introduced in many other countries.The process of gyratory compaction involves applying a constant uniaxial compressive force while shear action (kneading action) is applied. The shear action is obtained by applying the compressive force at an offset generated by tilting the mould. The angle at which the axis of the mould is inclined to a vertical plane during compaction multiplied by two is known as the angle of gyration and it generally varies from 1.00 to 6.00 degrees for various compactors.

It has been recognized that different compaction techniques produce asphalt concrete specimens with different particle orientation and thus differing physical properties. When evaluating asphalt concrete mixtures in the laboratory, it is desirable to produce test specimens that duplicate, as nearly as possible, the compacted mixture as it exists in an actual pavement layer. The question is, therefore, which of the available compaction techniques are best suited to achieve this goal. Several researcher have conducted compaction studies and have compared different kinds of compaction techniques in an attempt to answer this question. These include Button et al[82], Consuegra et al [83], and Sousa et al[76].

Button et al[82] compared four compaction devices, which are Exxon rolling wheel, Texas gyratory, rotating base Marshal hammer and Elf linear kneading compactors, in order to determine which of these four compactors closely simulate actual field compaction. The authors obtained field cores from five pavement sites and also compacted materials, which are identical to those obtained from the field, in the laboratory to the same range of void contents as in the field cores.The specimens from different compaction devices were subjected to indirect tension at 25^{o}C, resilient modulus at 0 and 25^{o}C, Marshal stability, Hveem stability, and uniaxial repetitive compressive creep followed by compression to failure tests. The results of these tests on samples compacted in the laboratory using the four compaction devices were statistically compared to those obtained from the same tests conducted on field cores. Specimens representing the selected mixtures in this study was also analysed using microscopy and imaging techniques at the National Road Laboratory in Denmark. The main conclusion from the study were:

1.The gyratory method most often produced specimens similar to pavement cores. Elf and Exxon compactors had the same probability of producing specimens similar to pavement cores and the Marshal rotating base compactor had the least probability of producing specimens similar to pavement cores. However, the differences among these compactors were not statistically significant (at $\alpha = 0.05$).
2. The Exxon rolling wheel compactor exhibited much more difficulty in controlling air voids in the finished specimens while the gyratory compactor was found to be much more convenient, faster and cheaper for producing specimens of specific air void contents.
3. Generally, the pavement cores and the Exxon rolling wheel compactor exhibited better homogeneity of air void distribution than the gyratory compactor.

The work of Consuegra et al[83] involved comparative evaluation of laboratory compaction devices based on their ability to produce mixtures with engineering properties similar to those produced in the field. The engineering properties considered were, resilient modulus, indirect tensile strength, strain at failure, and tensile creep data. Five compaction devices, namely, Texas gyratory shear compactor, California kneading compactor, Marshal impact compactor, mobile steel wheel simulator, and Arizona vibratory kneading compactor were evaluated. The conclusion from this study was that:

1. the Texas gyratory compactor demonstrated the ability to produce mixtures with engineering properties nearest to those from the field cores,
2. the California kneading compactor and the mobile steel wheel simulator ranked second and third, respectively, but with very little difference between the two, and
3. the Marshal impact hammer and the Arizona vibratory kneading compactor were ranked as least effective in terms of their ability to produce mixtures with engineering properties similar to those from the field cores.

However, the compaction devices considered in this study were not evaluated on the basis of the uniformity of void distribution in the compacted samples. The gyratory compactor was reported to be less effective in producing specimens similar to field cores in terms of the uniformity of void distribution in other studies.

The study by Sousa et al[84] evaluated three compaction devices: Texas gyratory, kneading and rolling wheel compactors. These devices were evaluated based on the extent to which method of laboratory compaction affects fundamental mixture properties (permanent deformation and fatigue) related to pavement performance. The main points of the conclusion drawn from the study were:

1. Samples compacted with Texas gyratory compactor appear to be more sensitive to asphalt type (binder grade) than samples by the kneading compactor
2. Samples prepared using the kneading compaction device were more resistant to permanent deformation and were more sensitive to aggregate angularity and surface texture.
3. Specimens prepared using rolling wheel compactor were ranked between specimens prepared using kneading and gyratory methods in terms of their resistance to permanent deformation but they were stiffer under dynamic loading and more fatigue resistant than either gyratory or kneading specimens.

Based on these findings, the authors stated that the compaction method had a profound impact on fundamental mixture properties and concluded that among the methods investigated, rolling wheel appeared to best duplicate field compacted mixtures. A shortcoming of this study is, however, that it was not correlated to field results.

While research reports cited above tend to imply that the gyratory shear compactor is the best compaction device in terms of producing specimens with average properties similar to those from field cores, several other authors including Voskuilen[85], Neubauer[86], Butcher[87] pointed to the problem of non-uniform density distribution in gyratory compacted specimens. Density distribution (void distribution) in gyratory compacted specimens was analysed using techniques such as nuclear density measurement methods and image analysis[85]. It emerged that the gyratory compactor compacts from the centre outwards and it produces a specimen with density at the centre significantly higher than that at the ends and the periphery of the specimen. Figure 5.12 illustrates the variation of density with respect to height of a gyratory compacted specimens.

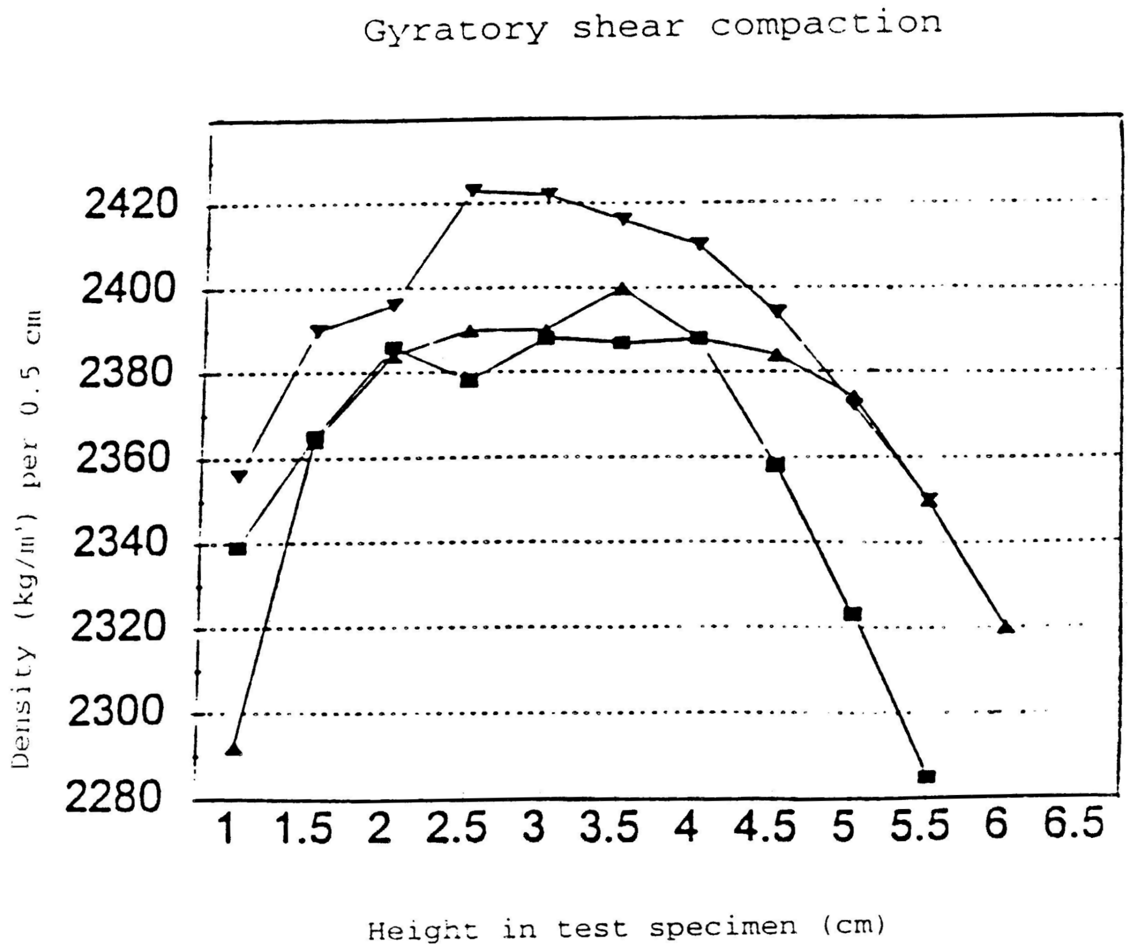


Figure 5.12 Density distribution in specimen made by gyratory compactor[85]

It has also been reported that even if specimens compacted in the laboratory have the same density (voids) as those from field, their mechanical properties can still be different. This difference might be due to the non uniform distribution of density (voids). Therefore the significance of this problem should not be underestimated.

For this particular study the choice of compactor was made based on the discussion in the preceding paragraphs. Only California kneading and gyratory shear compactors were available and thus the choice was limited to these two compaction devices. California kneading compactor was chosen primarily because of concern regarding density distribution in gyratory compacted specimens.

Specimens 100 mm in diameter and about 200 mm in height were compacted in five layers. Several trial compactions were made to determine the number of tamps and compaction pressure required to produce specimen with a uniform density. The specimens were then sliced into five and their densities measured and coefficient of variation calculated. A compaction procedure which produced specimen with the coefficient of variation in density of the slices of 0.5% or less was then selected. After compaction, the specimens were subjected to static compression load of about 56KN for 30 seconds following ASTM (ASTM 1561-92) recommendations.The specimens are extruded from the mould and were allowed to cool to room temperature before they were trimmed and their densities were measured. Specimen densities were measured using the saturated surface dry (SSD) procedure. The specimens were trimmed at both ends (about 10 mm from each end) to produce the final test specimens. Overall specimens with void levels of the targeted void levels +/- 0.8 were obtained. Cutting the ends was found to further improve the uniformity of density since it is usually the ends that show relatively higher difference in density from the mean. Figure 5.13 show a picture of the California kneading compactor used in this study.

Figure 5.13 California Kneading Compactor

5.5 Testing Procedure

Two types of tests were conducted in this study; cyclic load triaxial and confined creep and recovery tests. For both tests, specimens with height of 180 mm and diameter of 100mm, thus with height to diameter ratio of 1.8, were used. Both tests were conducted using electro-hydaulic triaxial testing apparatus schematically shown in Figure 5.14.

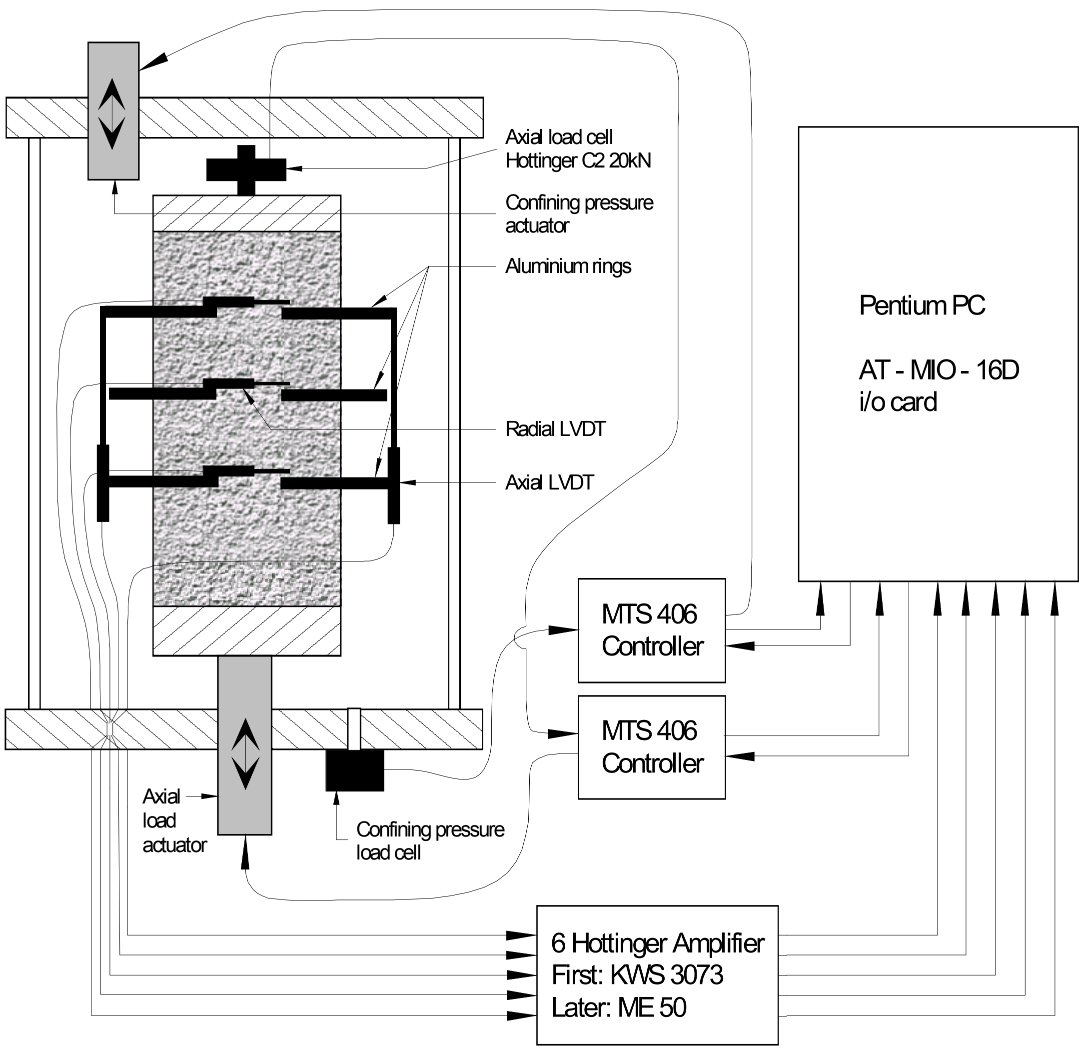


Figure 5.14 Schematic diagram of triaxial testing apparatus

Linear Variable Differential Transducers (LVDTs) were used to measure the axial and radial deformations. Two LVDTs were used to measure the axial deformation in the middle 100mm of the specimen. The radial deformation was measured using three LVDTs. The LVDTs have a measuring range of about 8 milimeters.Six small brass plates (15mmX15mm) with threaded hole at the centre were glued to the specimen. The plates were curved so that they can be securely glued to the curved surface of the specimen. Three circular rings were attached to the plates using small screws. The LVDTs, both axial and radial, were then mounted on the rings. The specimens were covered by plastic membrane prior to mounting the rings. Circular rubber rings and steel bands were placed on the bottom and top plates to secure the membrane to the plates and thus prevent ingress of water. Figure 5.15 shows a picture of a test specimen with rings and LVDTs mounted.

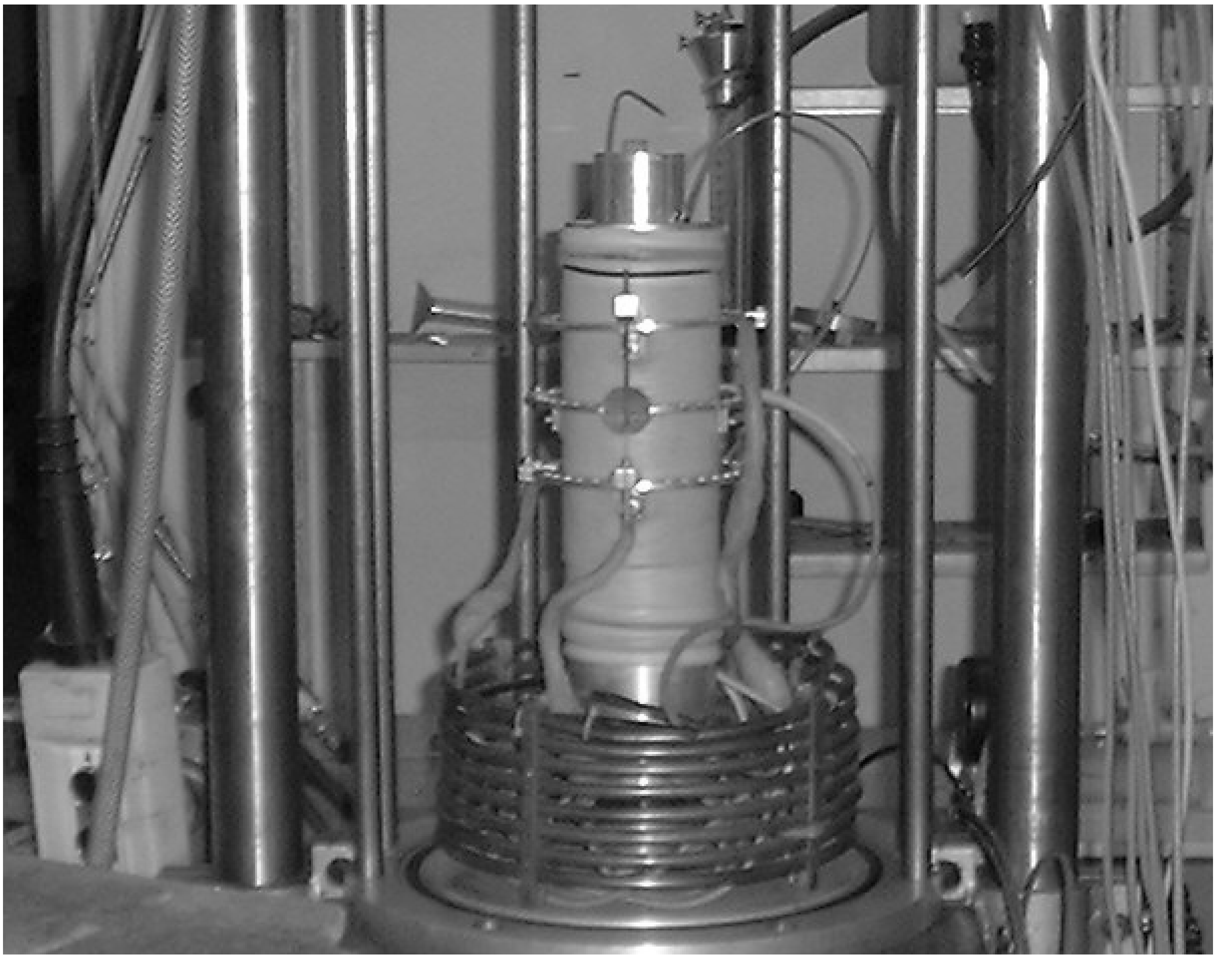


Figure 5.15 Test specimen

Friction and the associated restraining effect at the ends of the specimen is a major cause for concern in this types of testing because it can produce non-uniform stress distribution. To reduce the friction, two layers of smooth teflon paper were placed between the specimen ends and the bottom and top loading plates. Silicon oil lubricant was applied between the layers of teflon paper and also between the teflon paper and the loading plates.

After attaching the rings, the specimen was mounted on the testing rig and LVDTs were connected. When instrumentation was completed, a glass chamber was mounted on the rig and filled with water, which was pre-heated to the test temperature. Specimens were conditioned at the test temperature for two hours prior to testing. A circulation pump was employed to maintain uniformity of temperature with in the chamber. The temperature inside the chamber was monitored to make sure that the specimen was conditioned at the test temperature.Figure 5.16 shows a picture of the specimen in the testing apparatus.

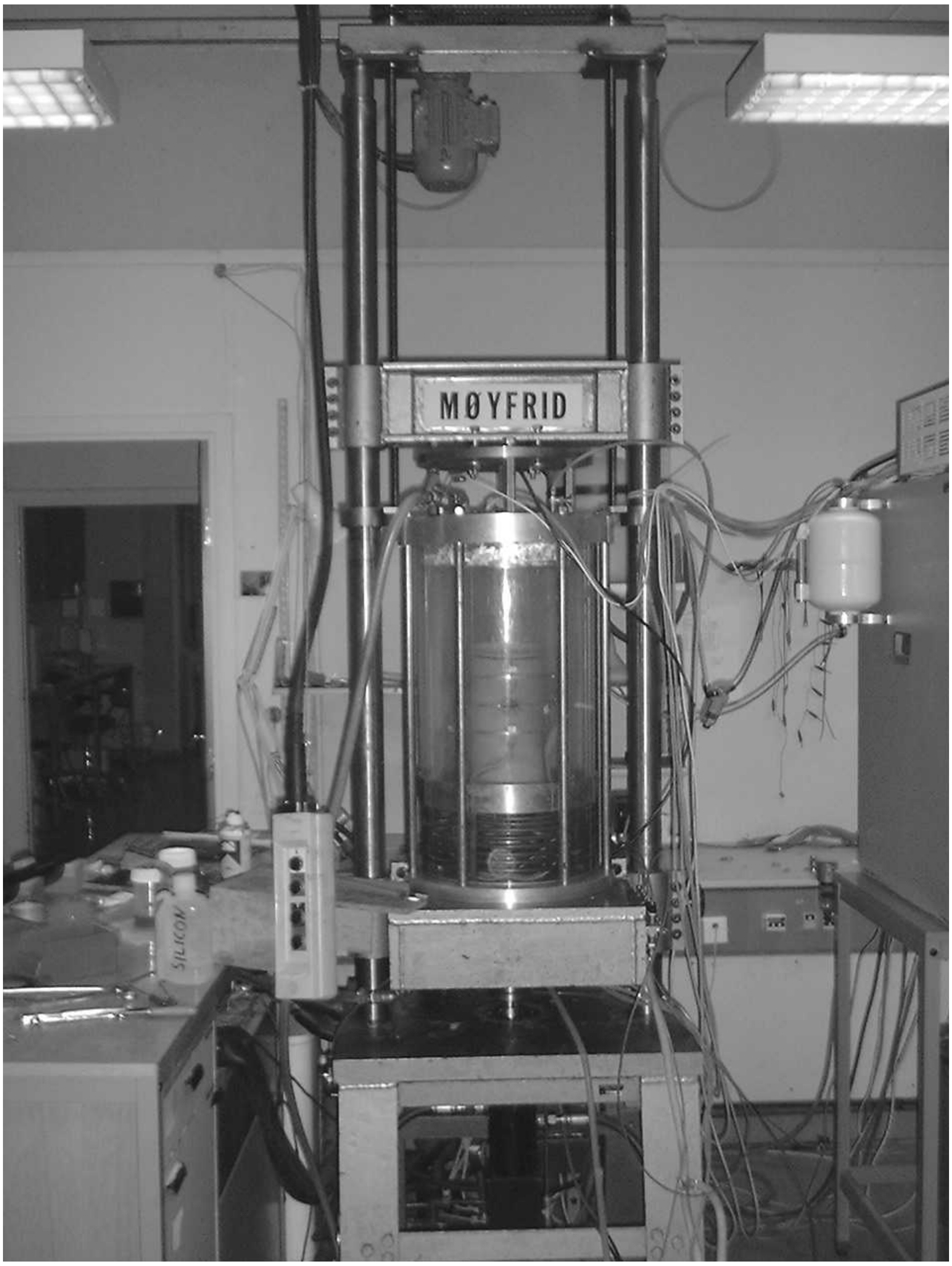


Figure 5.16 Specimen in triaxial testing apparatus

The cyclic load triaxial testing program consisted of application of a haversine load with constant confining pressure of 150 KPa. The amplitude of the haversine load pulse varied from 750 to 1500 Kpa, in steps of 250 KPa. In each step 100,000 load repetitions were applied except in the first step where 150,000 repetitions were applied at frequency of 10Hz. But only few specimens went through all steps, because for most of the specimens the deformation was so big after the first or second steps, that it went out the measuring range of the LVDTs. Some specimens were tested with confining pressure of 75KPa to study the effect of confining pressure on the development of permanent deformation. Two parallel specimens were tested for each combination of void content, binder content, and temperature. The tests were conducted at two temperature levels; 25 and 50 ^{o}C.

The creep and recovery test was conducted at 50^{o}C and at three stress levels of 450, 750, and 1000 KPa. The confining pressure was held at 150KPa. The loading program involved the application of a square load pulse for 10 seconds followed by a rest period (un-loading time) of 10 seconds. During testing strain was measured and recorded at a rate of 30 data points per second at the beginning and end of the loading period and at a rate of 4 data points per second in between loading and unloading. Thus, detailed data on deformation development during loading and recovery during unloading was obtained. The test speciemns had various levels of binder content and void content similar to those in the cyclic load triaxial testing program.

CHAPTER 6: ANALYSIS AND DISCUSSION OF TEST RESULTS

Several specimens were tested following the procedure described in the previous chapter. As mentioned before one of the objectives of the testing was to investigate the effect of changing volumetric composition on permanent deformation properties of asphalt concrete mixtures. Testing was conducted at two temperatures of 25 and 50^{o}C. The loading involved application of a cyclic deviatoric stress varying from 750 kPa to 1500 kPa at a frequency of 10 Hz and a constant confining stress of 150 kPa for most of the specimens. In this chapter the effect of changing binder content and void content on the measured permanent deformation at these two temperature levels will be presented. The effect of changing the loading conditions will also be presented to a limited extent. Analysis is limited to the accumulated axial permanent strain in the first stage of the loading, which involved the application 150000 load repetitions. Results are first presented in graphical form followed by an evaluation of the conventional permanent deformation parameters for their sensitivity to changes in volumetric composition. Three levels of binder content and void content are considered. The targeted void levels were 3%, 5%, and 8%. As mentioned in chapter 5, the void content of the specimens were within +/- 0.8% of the target void contents. However, the target void contents are used in presentation of the results for convenience.

6.1 Effect of Volumetric Composition on Permanent Deformation Properties of Asphalt Concrete Mixtures.

Volumetric composition of asphalt concrete mixtures is usually specified in terms of binder content and void content.The objective of any mixture design, for a given aggregate and binder types, is to find the optimum levels of binder content and void content, which would produce a mixture with satisfactory performance with respect to major distress mechanisms such as rutting and fatigue cracking. It is thus necessary to find out the effect of varying binder content and void content on performance of the mixture, in this particular case on its performance with regard to permanent deformation or rutting.

6.1.1 Effect of Binder Content

The binder is a viscoelastic material. Thus the time and temperature dependence of the deformation of asphalt concrete is due to the binder. Results of tests in this study indicate that, the binder content has a significant influence on permanent deformation behaviour of asphalt concrete mixtures. Three levels of binder contents were considered, i.e., 4.0%, 4.7% (optimum according to Marshal method) and 5.4%. Specimens with these three levels of binder content were compacted to the same level of void content and were tested at temperatures of 25 and 50^{o}C under the same loading conditions. Figures 6.1 and 6.2 show graphs of accumulated axial permanent deformation as a function of number of load repetitions for specimens with the three levels of binder content. These figures clearly show that the higher the binder content the higher is the accumulated permanent axial deformation. However, a low binder content does not necessary lead to rut resistant mixture, as mixtures with low binder content can be susceptible to shear deformation. This issue will be discussed in chapter 7.

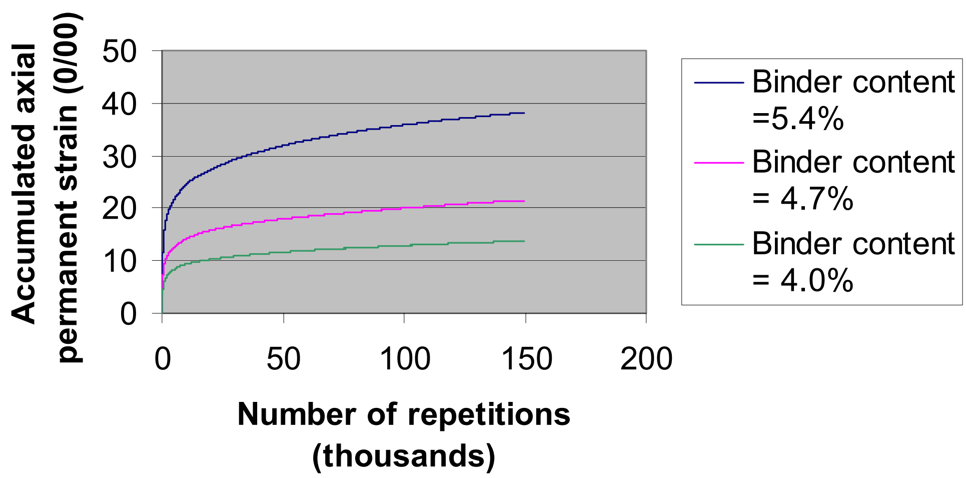


Figure 6.1 Effect of binder content on the accumulated axial strain at 50^{o}C (void content = 5%)

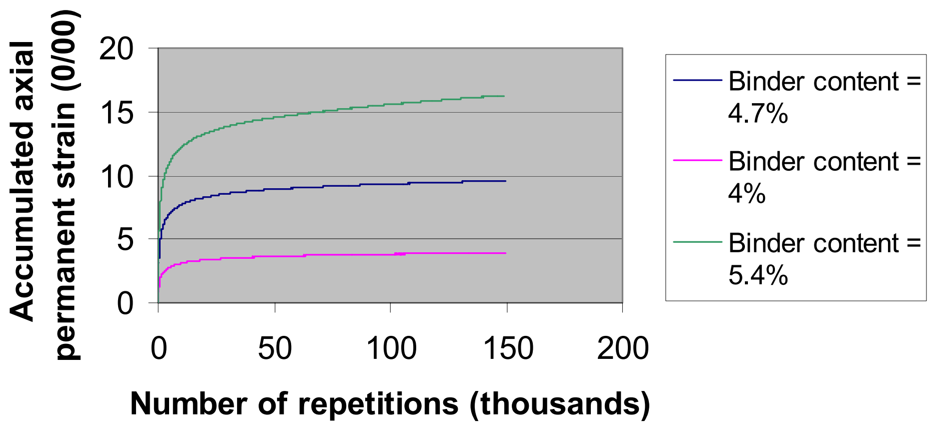


Figure 6.2 Effect of binder content on the accumulated axial strain at 25^{o}C (void content = 5%)

Figure 6.3 shows the variation of accumulated axial permanent strain after 150 000 load repetitions with binder content and temperature. It can be seen from the figure that the accumulated axial permanent strain for specimens with binder content of 5.4% is nearly double that of specimens with optimum (4.7%) binder content at both temperature levels. The effect of temperature appears to be more pronounced at lower binder content. The accumulated axial permanent strain for specimens with binder content of 4% at 50 ^{o}C is 3.5 times that of similar specimens at 25 ^{o}C while the accumulated deformation at 50 ^{o}C is 2.2 and 2.3 times that at 25 ^{o}C for specimens with binder content of 4.7% and 5.4% respectively. This might be because the thinner binder film at low binder content may yield relatively easily at higher temperatures as compared to the thicker binder film resulting from use of higher binder contents.

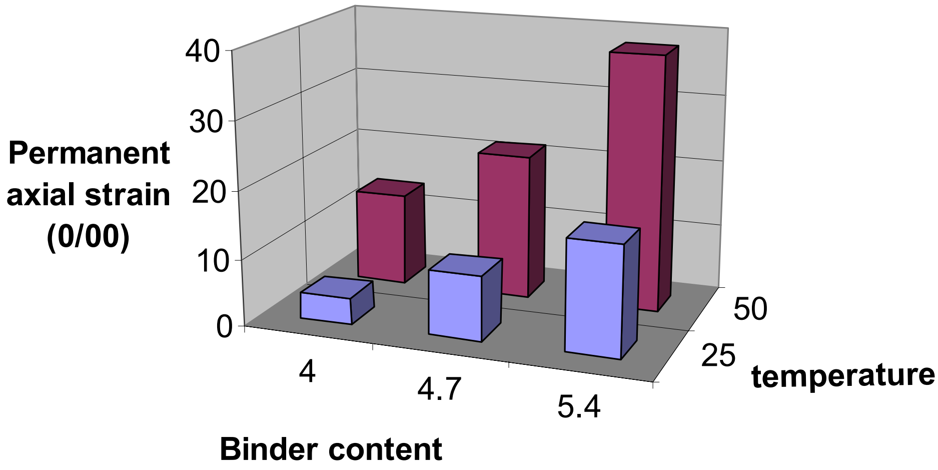


Figure 6.3 Variation of accumulated axial strain after 150000 repetitions with binder content and temperature (void content = 5%)

6.1.2 Effect of Void Content

The void content is one of the most important volumetric properties affecting the performance of asphalt concrete mixtures. The void content in asphalt concrete mixtures depends on the degree of compaction and available voids in the mixture, which in turn depends on aggregate gradation and binder content. There should be sufficient voids in the mixture to allow for additional compaction under traffic. A mixture with too high void content can undergo large permanent deformation due to compaction while the one with low void content can become unstable and rut due to shear deformation. Figures 6.4 and 6.5 show the accumulated axial permanent deformation for specimens with varying levels of void contents subjected to the same loading conditions at temperatures of 25 and 50^{o}C. Figure 6.6 shows the accumulated axial strain after 150000 load repetitions. It can be seen from the figures that the higher the void content the higher is the accumulated axial permanent strain, for a given number of load repetitions. This may give the impression that by densely compacting asphalt concrete mixture to a low void level, one can reduce or eliminate the problem of rutting. However, as will be shown in chapter 7, mixtures with low void levels may fail due to shear deformation, which involves dilatancy and debonding. This also indicates one dimensional analysis may lead to misleading results with regard to rutting because it fails to take into account the shear deformation which manifests itself as large radial deformation relative to axial deformation.

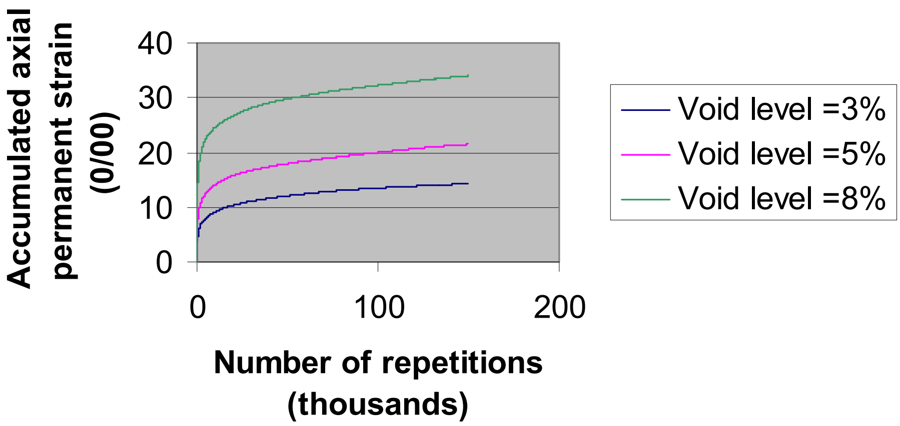


Figure 6.4 Accumulated axial permanent strain for specimens with different void levels at 50^{o}C (binder content = 4.7%).

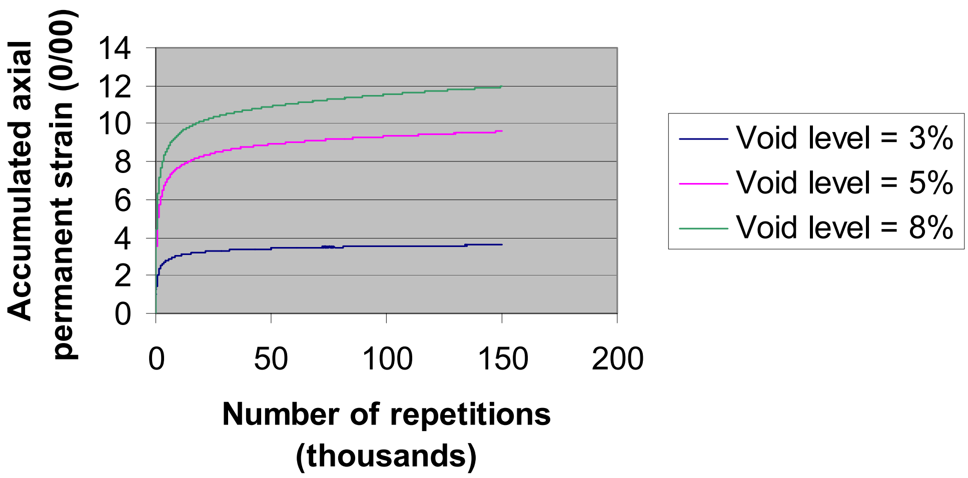


Figure 6.5 Accumulated axial permanent strain for specimens with different void levels at 25^{o}C (binder content =4.7%).

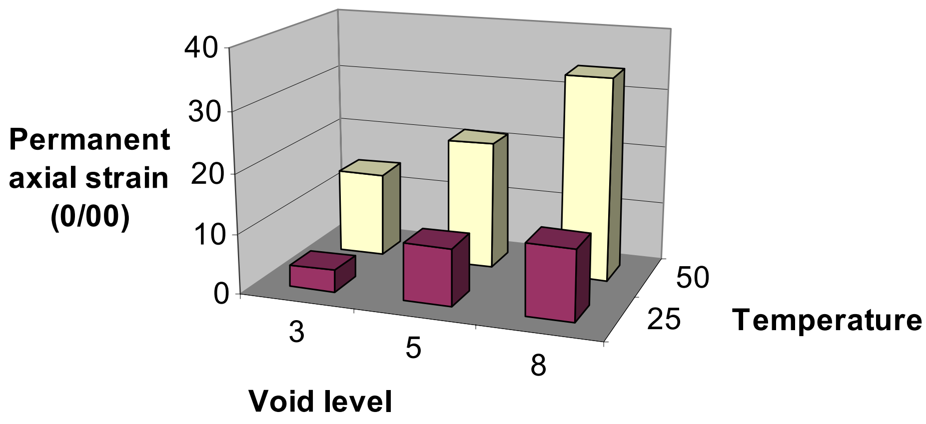


Figure 6.6 Accumulated axial strain after 150 000 repetitions for specimens with varying levels of void content (binder content = 4.7%)

6.1.3 Combined effect of binder content and void content

As mentioned before, the available void content depends, to some extent, on the binder content. Thus, it might be appropriate to look at the combined effect both on permanent deformation of asphalt mixtures. Figure 6.7 shows the accumulated axial permanent strain after 150 000 load repetitions for specimens with various levels of binder content and void content. It can be seen from the figure that the magnitude of change in the accumulated permanent deformation in response to changes in binder content is different for the various void levels. Comparing the changes in accumulated deformation resulting from increasing the binder content from 4.7% to 5.4%, it can be observed that the most pronounced change occurs at the low void level of 3%. Indeed, it was observed that the specimens with void level of 3% and binder content of 5.4% became quite unstable showing dilatant behaviour relatively early during loading. Thus, the combination of high binder content and low void level can result in a mixture with poor performance with regard to rutting. Specimens with binder content of 5.4% and void level of 8% experienced relatively large deformation, mainly involving compaction.

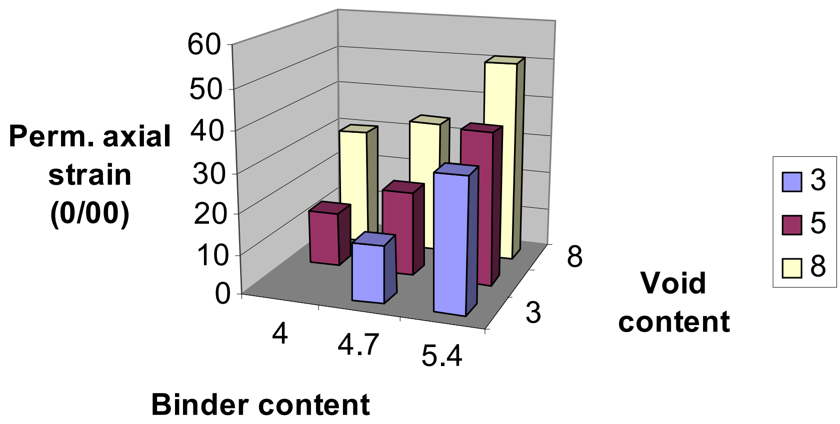


Figure 6.7 Accumulated axial strain after 150000 repetitions at 50^{o}C.

6.2 Effect of loading conditions on permanent deformation

Permanent deformation (or rutting) in asphalt pavements is a load-associated type of distress and hence the conditions of loading have great influence on the magnitude and rate of accumulation of permanent deformation. In the field asphalt pavements are subjected to complex loading with varying magnitude and frequency. In the laboratory, test must be conducted under loading conditions, which simulate the field loading conditions as closely as possible. The types of stress to which an asphalt pavement layer will be subjected and the necessity of testing at an appropriate level of those stresses have been discussed in chapter 5.

In this study most of the tests were conducted under similar loading conditions but limited tests were conducted with different loading conditions to have insight into the effect of loading. This was done by testing specimens with the same levels of binder content and void content at two levels of deviatoric stress and confining stress. Specimens with a void level of 5% and binder content of 4% were tested at two levels of deviatoric stress. Figure 6.8 shows the accumulated axial permanent deformation for specimens tested under deviatoric stress levels of 1000 and 750 kPa. It can be seen that the effect of the change in loading is very significant. The effect of this change is even more apparent when comparing the volumetric strains resulting from the two loading conditions. The accumulated volumetric strains are plotted in figure 6.9. It can be observed that the specimens, which were tested under deviatoric stress of 1000 kPa underwent dialation early in the loading process. As discussed in chapter dialation can result in debonding and deterioration of the pavements.

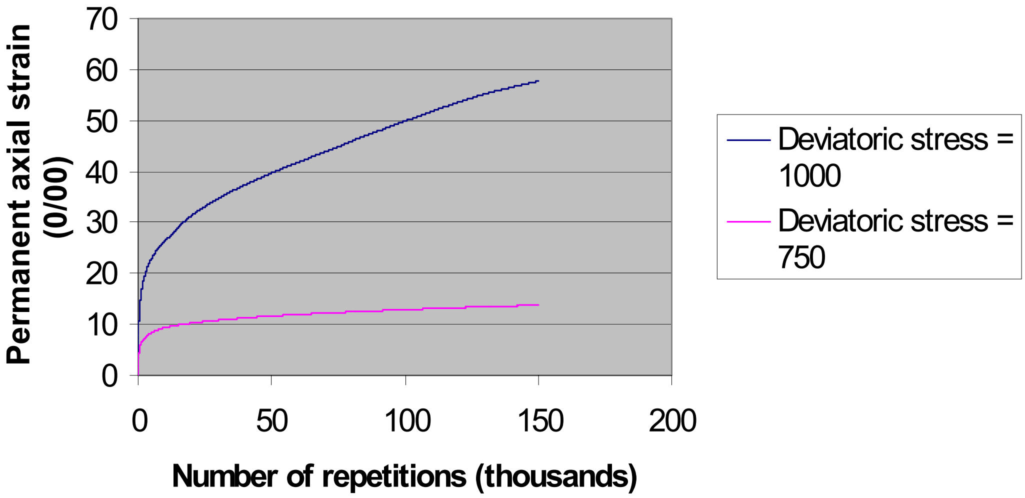


Figure 6.8 Permanent axial strain under different deviatoric stresses at 50^{o}C

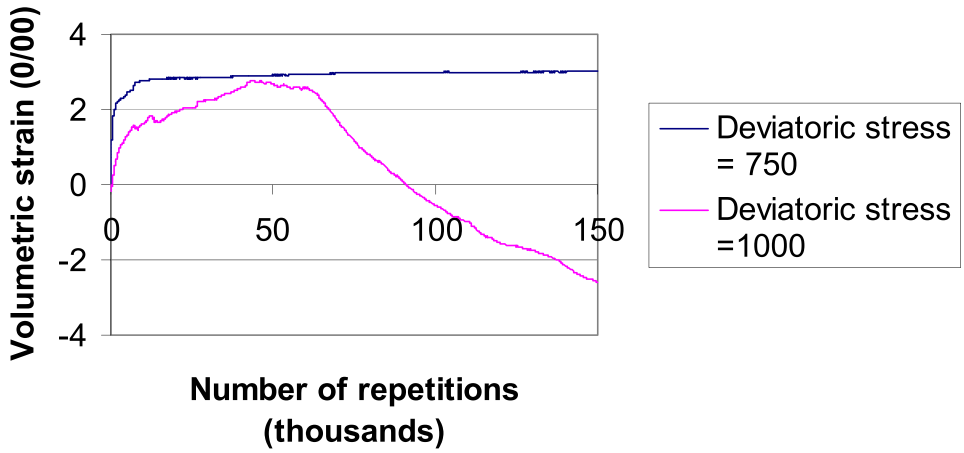


Figure 6.9 Volumetric strain under different deviatoric stresses

The confining pressure was also found to significantly influence the development of permanent deformation. Similar specimens were tested with a confining pressure of 75 kPa and 150 kPa at 50^{o}C. The specimens had void level of 5% and were made with binder content of 4.7%. Figure 6.10 shows the accumulated axial permanent deformation for these specimens. The effect of the change in confining pressure is very significant as can be seen from the plot. It is thus necessary to test laboratory specimens at an appropriate confining stress for characterization and prediction of their performance with respect to permanent deformation. It is difficult to measure the magnitude of confining or side pressure in the field. As a result, levels of confining stress varying from zero (no confinement) to several hundred kilopasicals have been used and reported in the literature. Given the influence that the confining stress has on the deformation, it is necessary to find ways and means of measuring or estimating the in situ side pressure for use in laboratory testing.

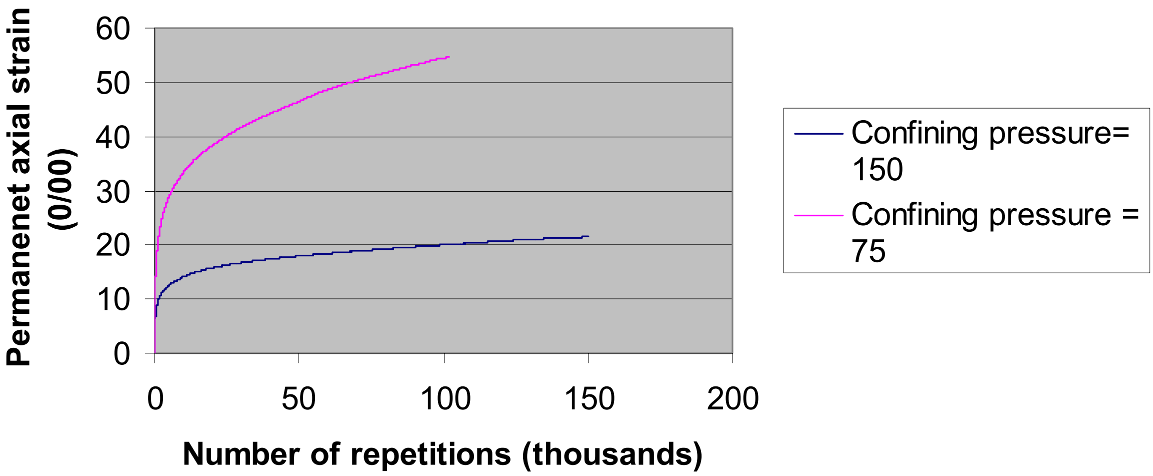


Figure 6.10 Permanent axial strain under different confining stresses

6.3 Measures for the Rutting Resistance of Asphalt Mixtures

One of the objectives of this study was to find or define a simple measure of resistance to permanent deformation or rutting of asphalt concrete mixtures. Various measures have been used and reported in the literature. Recent research indicated that the correlation of these measures with field performance is poor [77,88]. In this section attempt will be made to calculate some of the commonly used measures of performance for the specimens tested to see if these measures can be used to compare mixtures made from the same materials but with varying proportions of the components.

6.3.1 Creep Rate (Rutting Rate)

The rate of accumulation of permanent deformation, i.e., the accumulated permanent deformation per cycle of load application is often referred to as creep rate or rutting rate. This parameter has been used to evaluate asphalt mixtures for their susceptibility to rutting based on cyclic load triaxial test or the wheel tracking test. The use of creep rate as a measure of resistance to rutting has also been suggested in the draft European standard for cyclic triaxial compression test on asphalt mixtures. The creep rate is often calculated in the secondary creep range (the straight line portion) of the creep curve. However, this has proved to be difficult in many cases because generally there is no part in the creep curve with really constant slope. In addition some specimens can fail (enter the tertiary creep range) without showing any distinct secondary creep range and others undergo large deformation apparently in the primary creep range. The creep rate can be calculated by a least square linear fit of the linear part of the creep curve, if any linear part is present, but the result depends highly on the selected interval used for curve fitting. In this study data was recorded at relatively close intervals of about 30 cycles of load applications and the creep rate was calculated by assuming the curve with in each interval as straight line and taking the slope of the line. In the quasi straight part of the curve this method can be considered as fairly accurate. The calculated creep rate was then plotted against the number of load application. For purpose of completeness the creep rate in the primary creep range is also estimated in the same way but the values are obviously a crude approximation. The appropriate method for calculation of the slope of such a curved line is to fit a mathematical model to it and take its derivative. Such an approach will be discussed in the next section. Figure 6.11 shows the creep rate for specimens with different levels of binder content plotted against the number of load applications. As can be seen from the figure the creep rate is practically the same for the three specimens after few hundred cycles of load applications. However, the accumulated permanent deformation for the three specimens is significantly different as can be seen from figures 6.1 and 6.2. This indicates that the difference in total permanent deformation among these specimens occurs during the first few cycles of loading and the creep rate in the straight line portion of the creep curve is practically the same. Therefore, the creep rate does not differentiate the specimens and can not be used to compare these mixtures. The same is found to be true for specimens with varying levels of void content. Hence, it is reasonable to conclude that the creep rate is not an appropriate indicator of resistance to rutting and can not be used to evaluate and compare mixtures made form the same materials but with varying volumetric composition. Other studies have also indicated that the creep rate might not be a good indicator of rutting resistance [77].

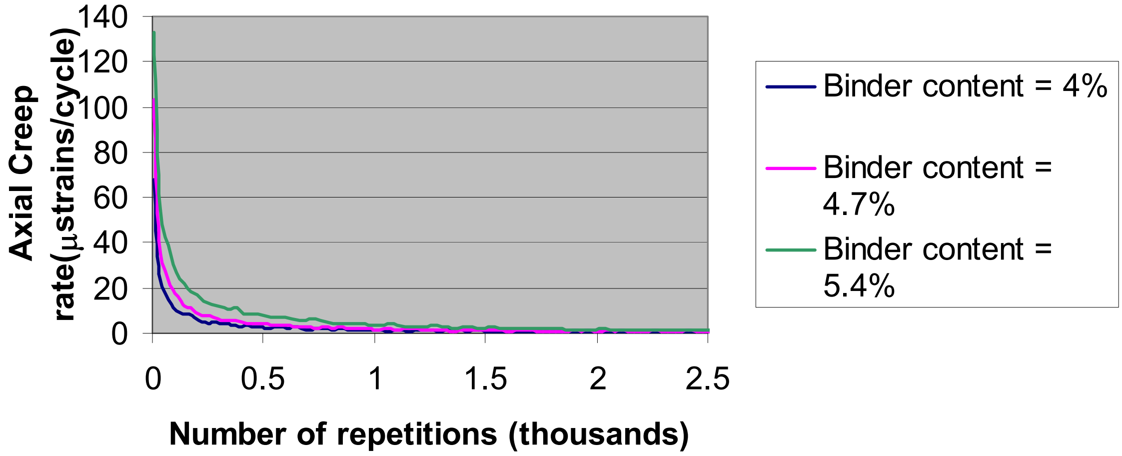


Figure 6.11 Axial creep rate for specimens with different levels of binder content

6.3.2 The Slope and Intercept of the Power Model

The power model is often fitted to the accumulated permanent deformation curve. It is probably the most commonly used permanent deformation equation. The power models plots as straight line on log-log scale.It has also been thought that the slope and intercept of this model when plotted on log scale may be used as indicators of rutting resistance. The power model may be expressed as:

$$\varepsilon_p = aN^b \tag{6.1}$$

Where ε_p is the accumulated permanent strain and a and b are regression constants. On a log-log scale the intercept a represents the permanent strain at N = 1, whereas the slope b represents the rate of change in permanent strain as a function of change in loading cycles (log(N)). Another form of the power model used to characterize the plastic strain per load repetition (ε_{pn}) can be expressed as:

$$\frac{\partial \varepsilon_p}{\partial N} = \varepsilon_{pn} = \frac{\partial}{\partial N}(aN^b) \tag{6.2}$$

$$\varepsilon_{pn} = abN^{(b-1)} \tag{6.3}$$

The resilient strain(ε_r) is assumed to be independent of load repetition. The ratio of plastic to resilient strain can thus be defined as:

$$\frac{\varepsilon_{pn}}{\varepsilon_r} = \left(\frac{ab}{\varepsilon_r}\right) N^{b-1} \qquad 6.4$$

Letting $\mu = (ab/\varepsilon_r)$ and $\alpha = 1\text{-}b$ one obtains:

$$\frac{\varepsilon_{pn}}{\varepsilon_r} = \mu N^{-\alpha} \qquad 6.5$$

where μ is a permanent deformation parameter representing the constant of proportionality between permanent strain and resilient strain (i.e. plastic strain at N= 1) and α is a permanent deformation parameter indicating the rate of decrease in incremental permanent deformation as the number of load applications increases. Thus the two sets of parameters a & b and μ & α are closely related. The parameters of the power model a and b were calculated using the method of linear regression, i.e., by fitting straight line to the logε_p-logN plot. Table 6.1 gives the values for specimens with varying levels of binder content. The specimens had a similar void levels of about 5% and were tested at 50^{o}C. Similarly, Table 6.2 provides the calculated values of a and b for specimens with varying levels of void content. These specimens had binder content of 4.7% and were also tested at 50^{o}C.

Table 6.1Power model parameters for specimens with different binder contents

Binder content (%)	4.0	4.7	5.4
a	2.0305	2.9806	4.5185
b	0.1617	0.1663	0.1806

Table 6.2Power model parameters for specimens with different void levels

Void level (%)	3	5	8
a	1.6485	2.9806	6.9103
b	0.1836	0.1663	0.135

It can be observed from the tables 6.1 and 6.2 that the values of the slope parameter b varies within narrow range. For specimens with different binder contents it varied between 0.1617 and 0.1806 whereas for specimens with different void levels it varied between 0.135 and 0.1836. The slope parameter appeared to increase with an increase in binder content while the reverse is observed for increase in void level, i.e., the slope parameter decreased with increase in void level. Since the slope parameter varied within a narrow range, it is difficult to make comparisons and draw conclusions on trend it indicates.The deformation rate was calculated using equation 6.3 and is plotted in figure 6.12. It can be seen that the deformation rate increases with increase in void level, contrary to the trend shown by the parameter b, indicating that the parameter by itself does not provide a measure of the rate of deformation. Thus, it seems to be reasonable to conclude that the slope parameter may not be a good measure of resistance to permanent deformation and in particular it might not be an appropriate measure of resistance to be used to compare asphalt mixtures made from the same materials, as is the case in this study.

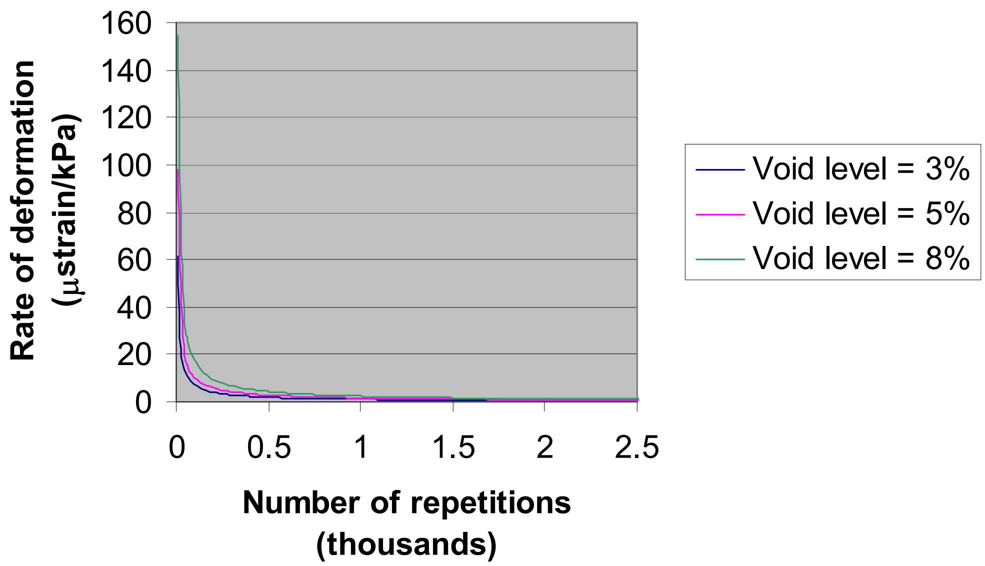


Figure 6.12 Computed rate of deformation

The intercept parameter of the power model appears to be sensitive to changes in both binder content and void level. As can be seen from tables 6.1 and 6.2, this parameter increased with increase in binder content and void level, following the trend shown by the accumulated permanent deformation. Thus a relatively high intercept value indicates a higher accumulated permanent deformation for any given number of load repetitions. As mentioned before the intercept parameter represents the permanent deformation in the first cycle of loading. Hence, it might be argued that for mixtures made from the same materials but with varying proportions, the difference in accumulated permanent deformation occurs in the first few cycles and the rate of accumulation of the permanent deformation is not significantly different. As will be discussed in chapter 7, a smaller axial permanent deformation does not necessarily indicate a rut resistant mixture because of the existence of shear deformation. Thus even if the intercept parameter of the power model is sensitive to changes in binder content and void level, it might not be a good indicator of the resistance to rutting.

6.3.3 Parameters of the Logarithmic Work Hardening Model

Subjecting asphalt concrete to repeated loading produces permanent deformation that increases at a decreasing rate. This response can also be represented by the following equation:

$$\varepsilon_p = \varepsilon_0 e^{-\left(\frac{\rho}{N}\right)^{\beta}} \qquad 6.6$$

where ε_p is the accumulated permanent strain and ε_0, ρ, and β are material parameters. Equation 6.6 is an equation of a logarithmically work hardening material and the parameter β is considered to be the logarithmic rate of work hardening. Work hardening refers to a hypothesis which assumes that the hardening behaviour of materials depends only on the plastic work, and is independent of the strain path, implying that the resistance to further yielding depends only on the total plastic work that has been done on the material. Equation 6.6 has been used as a rut prediction model in mechanistic - empirical approach based on finite element method and was reported to have been applied to both asphalt concrete and granular pavement materials[89].

Taking logarithm of both sides of equation 6.6 results in:

$$\ln\varepsilon_p = \ln\varepsilon_0 - \left(\frac{\rho}{N}\right)^{\beta} \qquad 6.7$$

The plot of $\ln\varepsilon_p$ versus N on log-log scale results in straight line, the slope of which is the parameter β. The parameters, ε_o, ρ, and β were calculated using regression technique. Tables 6.3 and 6.4 give the values of the parameters for specimens with varying levels of void content and binder content.

Table 6.3Variation of Logarithmic work hardening parameters with binder content

Binder content	4.0	4.7	5.4
ε_0	0.02775	0.04137	0.07129
ρ	17975.16	14598.63	13916.55
β	0.15197	0.15894	0.1880

Table 6.4Variation of logarithmic work hardening parameters with void level

Void level	3.0	5.0	8.0
ε_0	0.02515	0.04137	0.06643
ρ	8645.42	14598.63	10018.86
β	0.18125	0.15894	0.13668

As can be seen from table 6.3, the values of the parameters seem to be sensitive to binder content. The parameter ε_0 increased with increasing binder content following the trend indicated by the accumulated axial permanent deformation. The reverse is true for ρ, i.e., it decreased with increasing binder content. The parameter β showed an increasing tendency with increasing binder content, though the range of variation is small.

The variation of ε_0 with void level is similar to that with binder content as can be observed from table 6.4, i.e., it increased with increase in void level. The variation of ρ with void level did not show any clear tendency while β decreased with increasing void level.

The range of values of ε_0, ρ and β parameters obtained in this study generally fall with in the range reported by Tseng and Lytton [89] for asphalt concrete materials. It is not so easy to give some physical meaning to the parameters ε_0 and ρ. β has been termed the logarithmic rate of work hardening by Tseng and Lytton. But the range of variation of β is generally very small and the trend it shows is not clear. Further the observed variation of β with binder content does not agree with observations made earlier regarding the effect of binder content on permanent deformation. It therefore appears that these parameters may not be good indicators of rutting resistance and are not pursued further.

6.4 The Stiffness of Asphalt Mixtures and its Relation to Permanent Deformation

Traditionally, it was believed that the stiffer the mix, the better its performance against permanent deformation. Empirical mix design methods have relied on this belief and have been employed to produce high modulus mixes. Early empirical equations for calculation of rutting in flexible pavements used the stiffness modulus (or the resilient strain) as one of the major variables. An example of such equations was the one proposed by Van de Loo and was incorporated into Shell Pavement Design Manual expressed as follows:

$$R_D = C_m h_1 \left(\frac{\sigma_{av}}{S_{mix}}\right) \qquad 6.8$$

in which R_D is the rut depth, C_m is a correction factor for dynamic effects, h_1 is the thickness of asphalt layer, σ_{av} is the average vertical stress in the asphalt layer, and S_{mix} is the stiffness modulus of the mix.

Various types of material properties are used for representing the stiffness characteristics of asphalt concrete, including creep compliance, relaxation modulus, complex modulus (dynamic modulus and phase angle), resilient modulus, etc. The stiffness measured in the laboratory is commonly used as an input to structural analysis models to predict pavement response under load. Measures of stiffness such as the relaxation modulus and complex modulus are considered to be properties of a linear visco-elastic material. However, under field loading conditions, asphalt concrete material may undergo substantial plastic deformation and displays a non-linear elasto- viscoplastic response.

In this study, the resilient strain was measured during testing and the corresponding resilient moduli of the specimens were computed. The resilient modulus, M_r, is defined as:

$$M_r = \frac{\sigma_d}{\varepsilon_r} \qquad 6.9$$

in which σ_d is the deviatoric stress, ε_r is the resilient strain. Pavement materials, including asphalt concrete tend to 'shake down' through strain hardening under repetitive loading. Thus, after several repetitions of loading subsequent deformation becomes predominantly recoverable. The resilient modulus is the ratio of the repeated stress to corresponding recoverable or resilient strain during such loading, i.e., it is the elastic stiffness of the material after many load repetitions have been applied. The resilient modulus of asphalt mixtures is thought to be more appropriate for use in multi-layer elastic programs than other moduli.

Figure 6.13 shows a plot of resilient moduli against number of load repetitions for specimens with varying levels of binder content and tested at temperature of 25^{o}C. As can be seen from the figure, the lower the binder content the higher the modulus. Also, as has been shown earlier, the lower the binder content, the lower is the accumulated permanent deformation. However, this trend could not be reproduced for tests conducted at 50^{o}C, as can be seen from figure 6.14. The specimens with binder content of 5.4% appear to have more or less similar stiffness values with the ones with binder content of 4.7%, which is contrary to the trend shown by the accumulated axial deformation. Such behaviour has also been observed by Tayebali et al [90]. A possible explanation for this effect could be that when specimen undergoes large plastic deformation, the resilient (elastic or recoverable) strain appears to be smaller, because the specimen does not fully recover during the unloading period. Since the resilient modulus is calculated as a ratio of the deviatoric stress to the resilient strain, specimens which experience large plastic strain may exhibit apparently higher stiffness values.

The variation of resilient modulus with void level also did not follow the trend shown by the accumulated permanent deformation. Thus, the resilient modulus, while appropriate for elastic structural analysis, might not be an appropriate measure of resistance to permanent deformation.The resilient moduli for specimens with low void level or binder content are relatively higher. However, as will be shown in chapter 7, these specimens could be relatively more susceptible to shear deformation, which is one of the major causes of rutting in flexible pavements, further weakening the relation of resilient modulus to rutting.

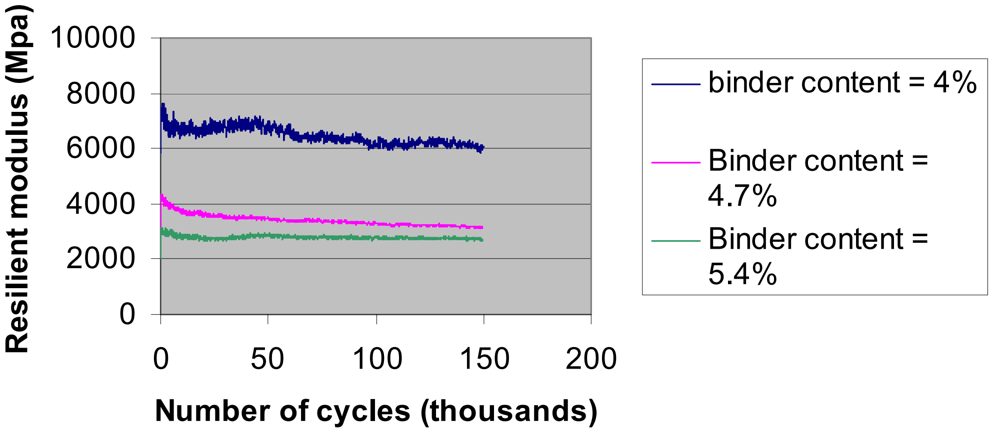


Figure 6.13 Resilient moduli of specimens with varying binder contents at 25 ^{o}C

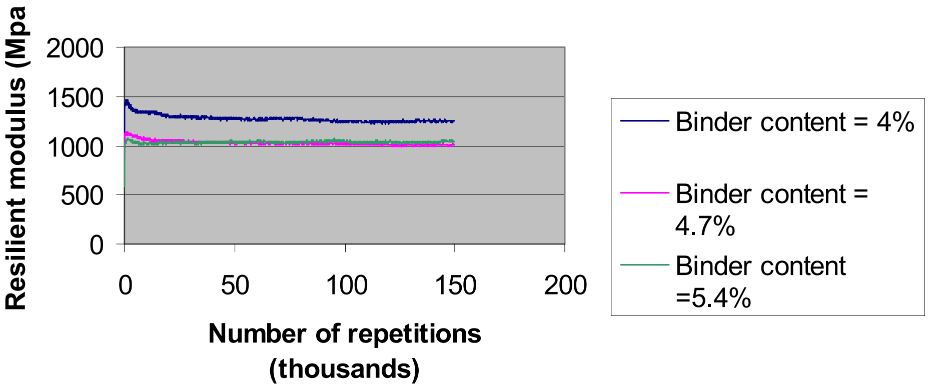


Figure 6.14 Resilient moduli of specimens with varying binder content at 50^{o}C

6.5 Summary

In this chapter results of cyclic load triaxial test on asphalt concrete specimens with varying levels of binder content and void content were presented and discussed. It has been observed that both the binder content and void content significantly affect the magnitude of the accumulated permanent deformation. Also temperature and loading conditions were found to substantially influence the permanent deformation response of asphalt concrete mixtures.

Parameters that are based on commonly used permanent deformation models and were used as a measure of resistance to permanent deformation were calculated for various specimens. The ability of these parameters to differentiate between the various specimens was assessed. It was shown that most of these parameters are either insensitive to changes in volumetric composition or do not show any consistent trend. Further, the parameters were based on one dimensional analysis, i.e., axial permanent strain. But a major cause of rutting in asphalt pavements is shear deformation, which manifests its self as a lateral movement of materials under load. Thus, the use of parameters that are based on axial deformation alone fails to capture all aspects of rutting or permanent deformation and might not be appropriate. Measure of resistance that takes the lateral deformation into account is proposed in the next chapter based on creep and recovery tests. Also methods of evaluation of mixtures for their shear susceptibility is discussed in the next chapter.

CHAPTER 7: MODELLING THE PERMANENT DEFORMATION PROPERTIES OF ASPHALT CONCRETE MIXTURES.

Asphalt concrete mixtures are very complex to characterize as their properties vary with composition, temperature, and the level and frequency of load as discussed in previous chapters. At low temperatures, low load levels and high frequencies, asphalt concrete materials could be modelled as linearly viscoelastic materials with hereditary characteristics. But at high temperatures, slow loading rates, and high loads, the behaviour of asphalt concrete materials tend to be non-linear elasto-viscoplastic.

At present no comprehensive model capable of describing the complex behaviour of asphalt concrete exists. Several empirical and theoretical models are however proposed. These models have been reviewed in chapter 4. In this chapter an attempt will be made to first describe the mechanisms of asphalt concrete deformation and then model the deformation behaviour using two approaches; the bounding surface plasticity and elasto-viscoplastic approaches.

7.1 Mechanisms of Asphalt Concrete Deformation

Asphalt concrete is a composite material. The bulk of the material is constituted of aggregates and mineral fillers varying in size from tens of micrometers to several millimetres. The aggregates are bound together by bituminous binders. Asphalt concrete also contains air voids. The composition of the material has a significant effect on its deformation behaviour as indicated by test results discussed in chapter 6.

The binder is a viscoelastic material with properties similar to polymers. The time and temperature dependent properties of asphalt concrete are thus imparted to it by the binder. On the other hand the aggregates, mostly naturally occurring crushed rock or gravel, have negligible time and temperature dependent properties and impart mechanical strength to the mixture. The response of asphalt concrete to an external stimulus, be it thermal or mechanical, is thus made up of the responses of the two entirely different constituents and their interface. Thus at low temperatures, low load and high frequencies, we observe a behaviour like that of the binder and at the other extreme, i.e., at high temperature, high load and low frequency of loading, the properties are close to those of aggregates.

Ideally, one needs to consider separately the contribution of each of these components and their interface to the response of the material in order to realistically model the deformation and damage behaviour of asphalt concrete materials. Such approach has been widely used in modelling the behaviour of composite materials, both particulate and fibber reinforced, but a comprehensive model based on this approach has yet to be developed for asphalt concrete. Thus most of the modelling effort made so far are either empirical or are based on the continuum mechanics approach.

In response to creep loading, both static and cyclic, asphalt concrete materials develop permanent deformation which accumulates with time or number of load repetitions. This accumulated permanent deformation is the cause of rutting in asphalt pavements. The plot of the accumulated strain against time is often called creep curve. While the term creep often imply time dependent deformation, the accumulated permanent deformation in the case of asphalt concrete contains both time independent and time dependent components as discussed in the proceeding chapter. The creep curve is thought to have three distinguishable regimes; the primary creep, secondary creep, and tertiary creep regimes.

For other materials such as metals, the mechanisms of deformation in each of the creep regimes have been described in terms of changes in the internal crystalline structure. For asphalt concrete however, no such description of the mechanisms of creep exists. For instance, the primary creep regime is said to involve compaction as a result of particle rearrangement. But it can be argued that major particle rearrangement in this type of composite material involves substantial debonding at the binder- aggregate interface, which is unlikely to occur in the primary creep range.

In this study, the deformation of asphalt concrete is assumed to be made up of compaction and shear deformation. Given that pavements are laid in the field with void levels of as high as 8%, the compaction appears to account for a significant portion of the deformation as evidenced by test results presented in the previous chapter. It is postulated that during compaction, the matrix is pushed into the air voids filling the spaces between aggregates and resulting in reduction of the air void content. This process may also involve a bulk viscoelastic deformation of the binding matrix but major particle rearrangement is unlikely. During this stage the material hardens, with decreasing rate of deformation which eventually approaches zero unless the stress level is high enough to push the deformation into the secondary creep range.As the compaction process ceases gradually the volumetric strain becomes more or less elastic.

During the secondary creep the matrix-aggregate interface will be subjected to sustained shear straining with a tendency of debonding and slippage at the interface. In this stage, the permanent strain increases at a constant rate. Further loading will result in material degradation involving de-bonding of the binder matrix from the aggregates and development of microcracks. The material degradation manifests it self in the form large aggregate rearrangement and plastic dilatancy. This stage of deformation was referred to as tertiary creep though, as pointed out by Robert Lytton in his millennium lecture at TRB's 79th annual meeting, it is not a creep at all but softening of the material as a result of development of microcracks.

Thus, during a creep process, the micro-structure of asphalt concrete material changes continually. A realistic material model should, therefore, take into account the evolution of the microstructure during the loading process. Accordingly, it might be of interest to look into the process of void reduction in asphalt concrete during creep loading.

Asphalt concrete mixtures are designed to have sufficient air voids in total compacted mix to allow for additional compaction under traffic loading. If the voids in the total mix are low, rutting due to shear flow is imminent. It has been pointed out that reduction of air void levels to below 3% will result in excessive plastic shear deformation leading to rutting. Thus air void reduction can be looked up on as a damage parameter with regard to rutting. So far, however, no significant effort has been made to model the reduction in air void and consequently the compaction of asphalt concrete under load.

From thermodynamical view point, the ability of a material to absorb the deformation work may be used as a measure of the change in the state of the material, i.e., in its structure. Ideally elastic material does not absorb any deformation work and all its deformations are, therefore, reversible. Ideally viscous and ideally plastic materials absorb all their deformation work and undergo, consequently, only irreversible plastic strain, i.e., they are dissipative media (energy is dissipated in the form of heat). The degree of energy dissipation, therefore, indicates the degree of change in the structure of the material. In the following sections, the two mechanisms of deformation, i.e., densification and shear deformation are discussed.

7.1.1 Densification

Asphalt concrete mixtures under go compaction and densification under traffic loading. This densification or volumetric hardening of the material improves the resistance of the material to permanent deformation. The extent of densification or the degree of energy dissipated in densification can thus be used as a measure of resistance of asphalt concrete mixtures to permanent deformation or rutting. Such a measure has been proposed by Ramsamooji et al[13] based on stress-dilatancy theory. According to the stress-dilatancy theory, the ratio of permanent volumetric strain and vertical strain in triaxial compression test on cohesive/frictional materials is given by:

$$\frac{d\varepsilon_v^p}{d\varepsilon_1^p} = 1 - \frac{\sigma_1}{K\sigma_3 + 2c_f\sqrt{K}} \qquad 7.1$$

Where, σ_1 and σ_3 = major and minor principal stresses, respectively,

$K = \tan^2(45 + \phi_f/2)$

ϕ_f = the equivalent angle of friction between particles, modified to include simaltaneous deviations of individual particle directions from the mean direction. Its value varies from ϕ_μ, the angle of interparticle friction, to ϕ_{cv}, the critical state angle of friction.

Experimental studies indicated that the value of c_f tends to zero and that the value of K was strongly dependent on pore fluid [13]. In the stress-dilatancy theory, K was defined as:

$$K = \textit{Incremental Work in / Incremental Work out} \qquad 7.2$$

The ‘incremental work in’ refers to the work done by external forces, while the ‘incremental work out’ refers to the work done by internal forces of resistance, i.e., work done by frictional forces generated at points of contact. A relatively high value of K indicates that the incremental work out is relatively small and the external work input is absorbed in compacting or densifying the material rather than in producing sliding motions or tendencies at aggregate- aggregate contacts.

The plastic potential function for compression of granular materials in plain strain is given by:

$$g = \frac{\sigma_1^K}{\sigma_3} \qquad 7.3$$

For triaxial compression the plastic potential is deduced to be:

$$g = \frac{\sigma_3^{\frac{1}{(2K)}}}{\sigma_1} \qquad 7.4$$

In which σ_1 = vertical stress and σ_3 = horizontal stress. Based on this plastic potential, the ratio of volumetric and vertical strains can be expressed as:

$$\frac{d\varepsilon_v^p}{d\varepsilon_1^p} = 1 - \frac{\sigma_1}{K\sigma_3} \qquad 7.5$$

According to stress-dilatancy theory, the ratio of the plastic horizontal and vertical strains for triaxial compression is:

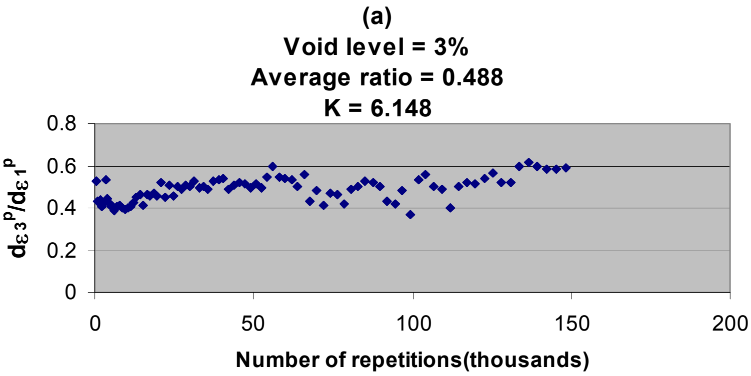
$$\frac{d\varepsilon_3^p}{d\varepsilon_1^p} = -\frac{\sigma_1}{2K\sigma_3} \qquad 7.6$$

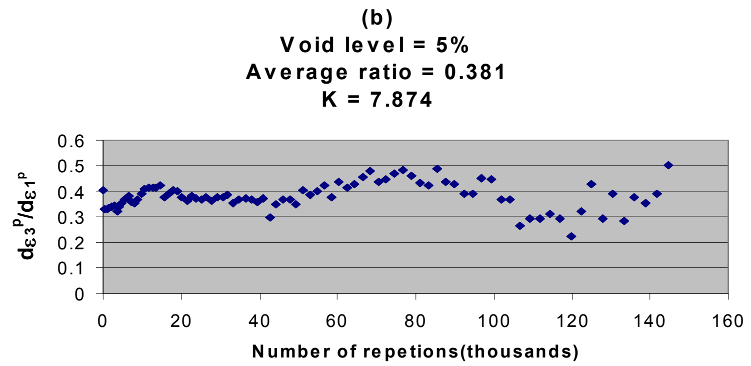
A small value of this ratio corresponds to higher value of K. Thus the higher the value of K, the higher the resistance to shear deformation. In other words, higher value of K indicates that higher proportion of the input energy is dissipated in plastic volumetric hardening or densification of the material. Shear deformation refers to a deformation with little or no volume change. In other words it is a deformation which results in distortion or change in shape. Shear deformation involves sliding or tendency for sliding at contact points, which may lead to debonding, plastic dilatancy and failure.

Figure 7.1 shows the ratio of incremental radial and vertical plastic strains plotted against the number of repetitions for specimens with three levels of void content, form tests conducted in this study. The ratio is computed by dividing the radial strain increment by vertical strain increment for every 500 repetitions.The specimens were tested at 50 ^{o}C under a sinusoidal deviatoric stress of 750 kPa with frequency of 10 Hz and confining pressure of 150 kPa. The specimens had a binder content of 4.7% (optimum binder content). It can be seen from the figure that, the ratio of incremental radial and vertical plastic strains is constant, though the data has some dispersion. This is found to be the case for all specimens undergoing compactive deformation and it agrees with results obtained by Brown and Snaith[92]. Using the average values of the ratio of radial and vertical plastic strains and equation 7.6, the values of K were determined to be 6.148, 7.874, and 8.174 for specimens with 3%, 5%, and 8% void contents, respectively.

As mentioned earlier, higher value of K indicates that higher proportion of the input energy is dissipated in densifying the material. In this regard, the K value has ranked the specimens correctly according to their void levels, i.e., the higher K values correspond to higher void levels, though the difference in value of K between the specimens with 5% and 8% void levels is relatively small. Thus a relatively lower value of K for specimens with low void level indicates that these specimens are relatively more susceptible to shear deformation.

However, a relatively higher K value does not necessarily indicate a lower overall accumulated axial permanent deformation as can be seen by comparing 7.1 and figure 6.4. This indicates the significance of permanent deformation resulting from compaction in the rutting of asphalt concrete pavements. As mentioned before, rutting in asphalt concrete pavement is caused by permanent deformation resulting form both densification and shear deformation. Although it has been argued that the shear deformation represents the mechanism of deformation for the greater part of the life of the pavement, deformation caused by compaction can be a significant portion of the total accumulated deformation.





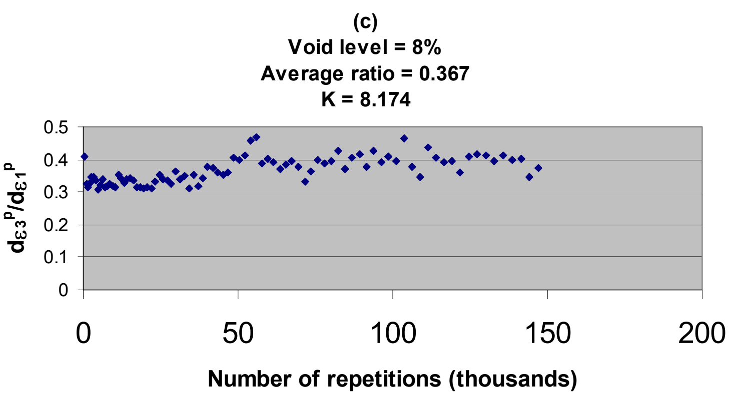


Figure 7.1 (a), (b) and (c) Ratio of incremental radial and vertical plastic strains for different levels of void content

The shear deformation may involve relatively larger lateral displacement which may be associated with debonding and damage of the material.As will be shown shortly the shear deformation can result in relatively high plastic dilatancy. In this regard, a volumetrically hardening (or densifying) material as indicated by its higher K value can be considered to have superior resistance to rutting caused by shear deformation as compared to the material with lower K value.

This analysis shows that for specimens with higher void level, rutting results mostly from compaction, while for specimens with low voids it results from shear deformation. Thus when evaluating mixtures for their rutting resistance it necessary to check both the total deformation and their susceptibility to shear deformation.Thus, mixture design has to aim at striking a balance between minimizing deformation resulting from compaction by making the material dense and minimizing the chance of shear deformation and failure by making it less dense. For the cases where shear deformation is considered to be the primary cause of rutting, the K value can be used as a measure of resistance to rutting of asphalt concrete mixtures in evaluating and ranking mixtures in terms of their resistance against rutting resulting from shear deformation.

Volumetric strains were calculated from the measured radial and axial strains using equation 7.7.

$$\varepsilon_v = \varepsilon_1 + 2\varepsilon_3 \qquad 7.7$$

Where:

ε_v = Volumetric strain

ε_1 = Vertical strain

ε_2 = radial strain

A positive value for volumetric strain indicates compaction or densification, while a negative value indicates dialation or increase in volume. Compressive strains are considered positive in the calculation of volumetric strain. The calculated volumetric strains were plotted against the number of load repetitions and are shown in Figure 7.2 for specimens whose ratio of lateral and vertical strains are given in Figure 7.1. The axial deformation for the specimens with void level of 8% were so large that they went out of the measuring range of the LVDTs in the second loading sequence. It can be seen that, for the first loading sequence in which a major principal stress

(σ_1) of 900 kPa and a minor principal stress (σ_3) of 150 kPa were applied, all the specimens underwent compaction, i.e., positive volumetric strain. The amount of compaction, however, varies with void level with higher void level corresponding to higher compaction as expected. The dense specimen (3% void level) showed very little change in volume indicating that it was undergoing shear deformation.

When the major principal stress was increased to 1150 Kpa in the second loading sequence, the specimens were observed to undergo plastic dialation. The extent of dialation is however more pronounced for the dense specimens. Plastic dilatancy involves major particle rearrangement and sliding at contact points, which results in debonding and material deterioration. Thus the dense specimens are more susceptible to this kind of deformation as compared to the less dense ones. This indicates that even though the accumulated deformation for the dense specimens appear to be lower than the other specimens, the probability of failure due to debonding and deterioration is higher.It can therefore be concluded that the dense material would experience more damage and deterioration compared to other materials with higher level of void content under the same state of stress. This conclusion is in agreement with the observation of many authors who stated that when the void level decreases to below about 3%, asphalt concrete may become unstable as result of shear flow.The forgoing discussion indicates that the K value can be used to evaluate asphalt concrete materials for their resistance to shear deformation.

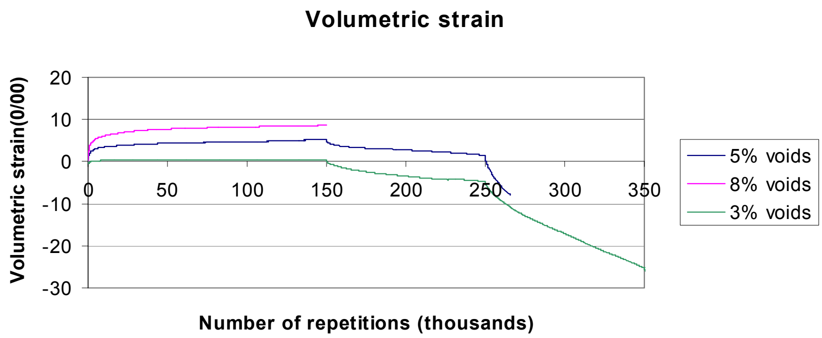


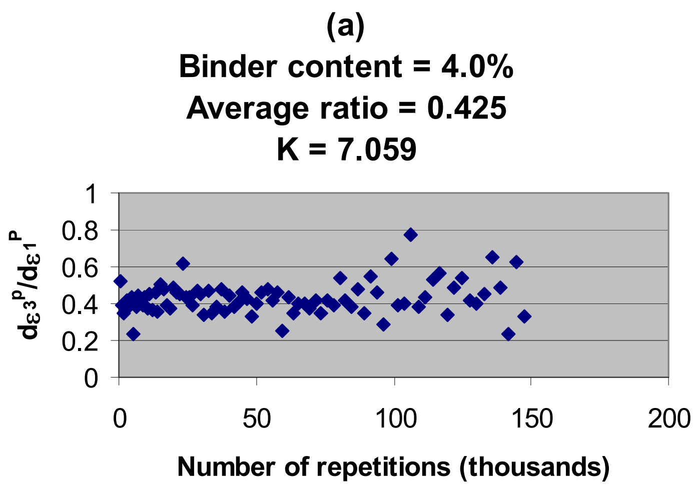
Figure 7.2 Volumetric strain for specimens with varying levels of void content

It has to be pointed out that the ratio of radial and vertical deformation and consequently the K value does not remain constant when the material deteriorates as a result of damage due to, for instance, development of microcracks, debonding and dialation. This has also been observed by

Brown and Snaith[92] who found that, for asphalt concrete specimens tested without confining pressure, the ratio of radial and vertical permanent strains was not constant, which probably was due to development of microcracks and damage.The K value is related to equivalent angle of friction as stated previously. Rowe [91] reported that for triaxial test on dense materials up to peak stress, the equivalent angle of friction equals the interparticle friction and thus remains constant and thereafter increases to the value of the critical state angle of friction.

The ratio of incremental radial and vertical plastic strains and the K values were also calculated for specimens with varying binder contents. Figure 7.3 shows the plot of the ratio of radial and vertical plastic strains against the number of load repetitions for specimens with binder contents of 4% and 5.4%. The specimens had void level of 5% and are tested under a deviatoric stress of 750Kpa and confining stress of 150Kpa at 50 ^{o}C.

As can be seen from Figure 7.3, the ratio of radial and vertical plastic strains is again constant.The calculated K values are 7.059, and 8.197 for the specimens with 4%, and 5.4% binder contents respectively. The K value for specimen with binder content of 4.7% and void level of 5% has already been determined to be 7.874. It was observed, during the fabrication of the specimens, that it was difficult to compact the specimens with 4% binder content to less than a void content of about 5% without excessively crushing the aggregates. Thus, further compaction during loading is not expected for these specimens. Obviously, the degree of compaction and the density that can be achieved depends on the proportion of the binder. Thus the relatively lower K value for the specimen with binder content of 4% indicates that its deformation mostly involves shearing instead of densification. As before the higher the K value the higher is the accumulated vertical permanent deformation indicating that a significant portion of the total rutting could result from compaction.



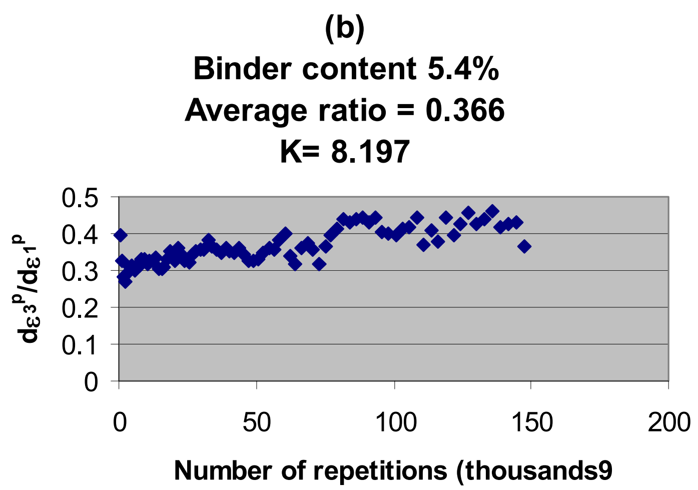


Figure 7.3 (a) and (b) Ratio of radial and vertical plastic strains for specimens with varying binder content

Figure 7.4 shows the variation of volumetric strain with the number of load applications for the same specimens whose ratio of radial and vertical plastic strains is shown in Figure 7.3. It can be seen from the figure that the higher the binder content the higher is the compaction and consequently higher accumulated axial deformation. It has been pointed out that the compaction stage involves the movement of binder matrix in to the void spaces between aggregate particles. Therefore, the observed higher compaction for the specimens with higher binder content is an expected result. The specimens with low binder content underwent small compaction but relatively large dialation.

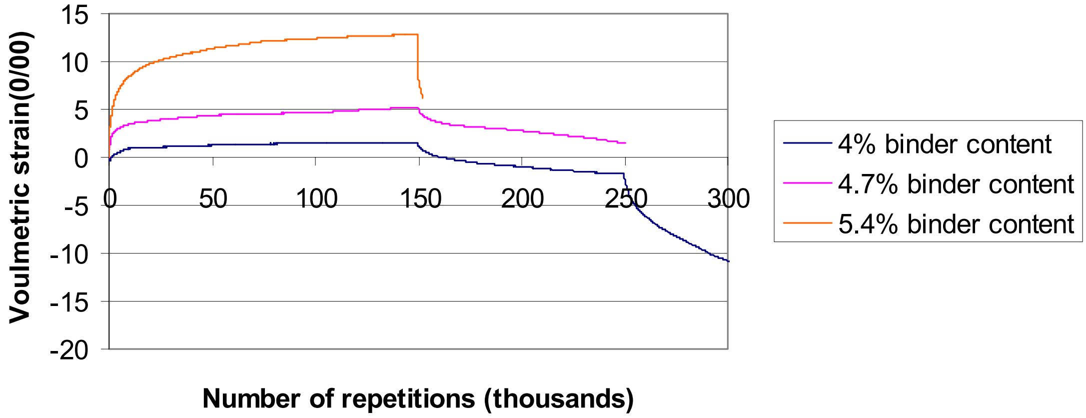


Figure 7.4 Volumetric plastic strain for specimens with varying binder content

Figure 7.5 shows plastic volumetric strain for specimen with binder content of 5.4% compacted to about 3% voids subjected to the same state of stress as the specimens discussed above. It can be seen that this material begun dialating much earlier than the other specimens, indicating a combination of high binder content and low void content may produce a material with least resistance to rutting.

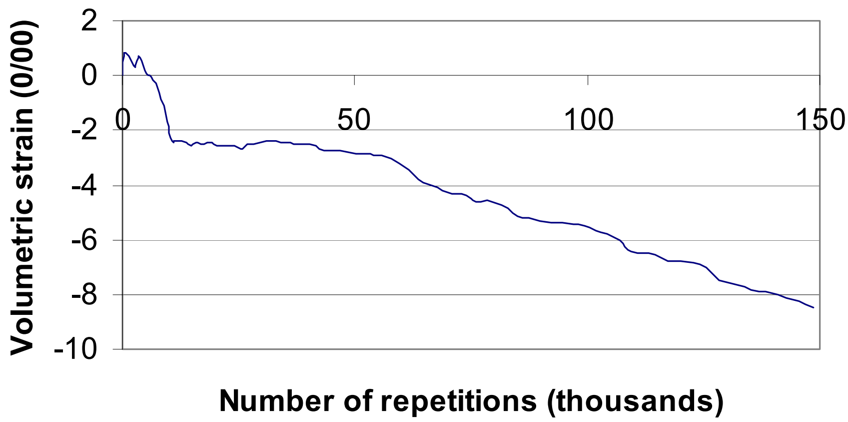


Figure 7.5 Volumetric plastic strain (binder content 5.4%, void content 3%)

7.1.2 Shear Deformation

It has been emphasized that rutting in asphalt concrete results both from densification and shear deformation. In principle shear deformation refers to distortion without volume change. However, in many materials including asphalt concrete, volume change and distortion are coupled. In a triaxial tests, symmetrical shear distortion combined with volume change, shown in Figure 7.6 below, would most probably occur.

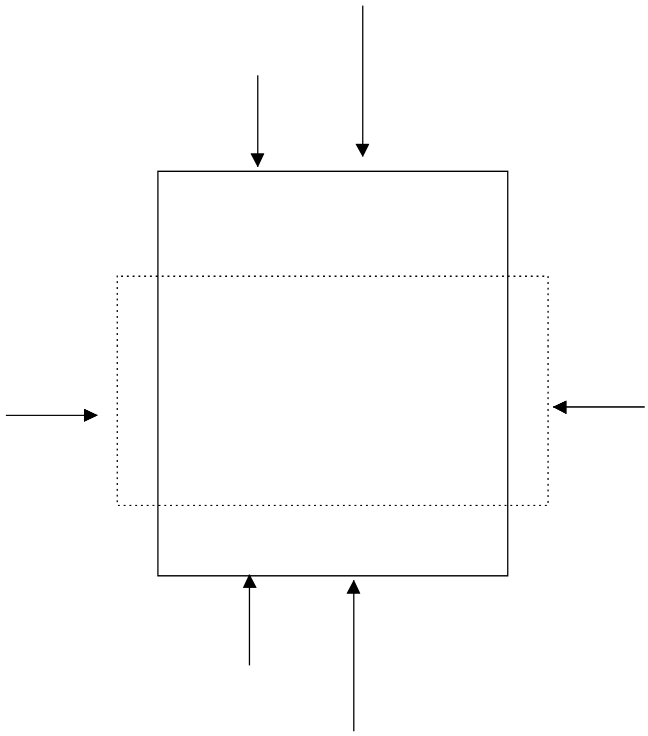


Figure 7.6 Symmetrical shear distortion and volume change combined

Under shearing action, both irrecoverable deviatoric strains and irrecoverable volumetric strains can result. In geotechnical literature, the source of these irrecoverable deformations is considered to be particle rearrangement [96]. The total irrecoverable volumetric strain increment can thus be divided into two parts; irrecoverable volumetric strain due to increment of mean pressure and irrecoverable volumetric strain due to increment of deviatoric stress. The strain energy increment can be expressed as:

$$dw = dw^e + dw_c^p + dw_d^p \qquad 7.8$$

$$dw^e = \sigma_{ij} d\varepsilon_{ij}^e \tag{7.9}$$

$$dw_c^p = p d\varepsilon_{vc}^p \tag{7.10}$$

$$dw_d^p = p d\varepsilon_{vd}^p + s_{ij} de_{ij}^p \tag{7.11}$$

Where:

dw^e	= elastic strain energy increment
$dw^p{}_c$	= strain energy increment dissipated in consolidation
$dw^p{}_d$	= strain energy increment dissipated in particle rearrangement
σ_{ij}	= stress tensor
$d\varepsilon^e{}_{ij}$	= elastic strain tensor
$d\varepsilon^p{}_{vc}$	= plastic volumetric strain due to increment of mean stress
$de^p{}_{vd}$	= irrecoverable volumetric strains due to increment of deviatoric stresses
$d\varepsilon^p{}_{ij}$	= irrecoverable deviatoric strain increment
$3p$	= $\delta_{ij}\sigma_{ij}$
s_{ij}	= σ_{ij} -$\delta_{ij}p$ is the deviatoric stress tensor
δ_{ij}	= the Kronecker delta

Strain energy increment dissipated in particle rearrangement has been used to model the shearing properties of soils and granular materials[96]. Here it was assumed $dw^p{}_d$ is a function of intergranular frictional properties, the mean pressure and a measure of particle rearrangement. The problem in using $dw^p{}_d$ as expressed by equation 7.11 is the difficulty of separating the volumetric strain caused by increase in mean pressure from the volumetric strain caused by the imposed deviatoric stress.

The irrecoverable deviatoric strain has been considered to be the most plausible measure for macroscopic manifestation of particle rearrangement. For asphalt concrete, however, the existence of the binder (or the matrix) changes the deformation process. The most plausible hypothesis for deformation of asphalt concrete is that in the first phase of deformation, the matrix will be displaced and pushed into spaces between aggregates. This phase is characterized by compaction and densification. That this phase is dependent on the proportion of the binder has already been discussed. Further loading will result in weakening of the bond between aggregate particles and initiation of frictional properties similar to that of granular materials. Therefore, the total deviatoric strain, which includes the effect of both matrix and particle displacements, may not be a good measure of particle rearrangement, if the term particle rearrangement is used in the strict sense of aggregate particle rearrangement. But the trajectory of the irrecoverable deviatoric strain may be used as measure of distortion or microstructural change, including both the displacement of the matrix and aggregate particles. The trajectory of irrecoverable deviatoric strain, ζ, can be expressed as:

$$\zeta = \int\sqrt{de_{ij}^{p} de_{ij}^{p}} = \int\sqrt{J_2(de^{p})} \qquad 7.12$$

Where $J_2(de^p)$is the second deviatoric irrecoverable strain invariant.

For the cylindrical state of strain in conventional triaxial testing, the irrecoverable deviatoric strain trajectory is the same as the cumulative deviatoric plastic strain. Figure 7.7 shows the cumulative deviatoric plastic strain for some of the specimens under consideration.

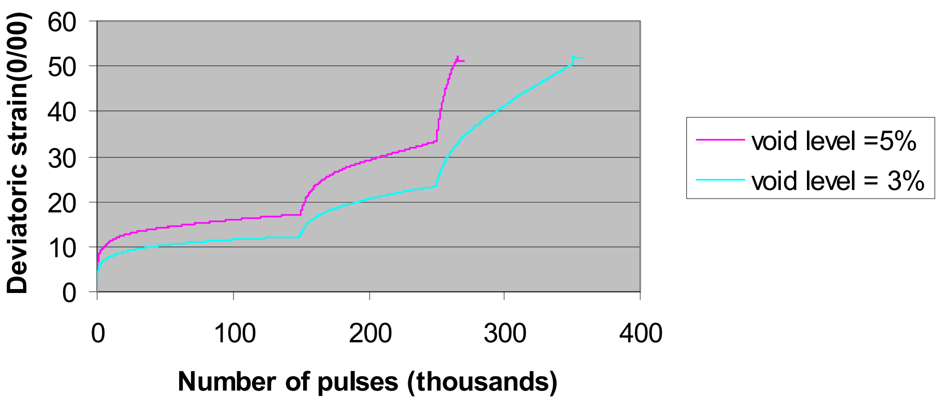


Figure 7.7 Accumulated plastic deviatoric strain (binder content = 4.7% a, temperature = 50^{o}C)

The two curves are similar to plots of axial plastic strain given in chapter 6 and as such do not provide additional information. However it might be of interest to look at this distortion measure in relation to the change in volume. Figure 7.8 shows the ratio of accumulated plastic deviatoric strain to the first invariant of the strain (or the volumetric plastic strain) for the two specimens shown in Figure 7.7.This ratio might be considered as a relative measure of distortion. The curves are nearly infinite (have a very large value) near the phase change point, i.e., when the behaviour changes from contarctancy to dilatancy, because at this point the volumetric strain is in principle zero. The figure shows that the ratio is constant in the first phase of loading for the specimen with 5% voids and it is less than the ratio for specimen with 3% voids. The volumetric strain behaviour of the two specimens is shown in Figure 7.2. It can thus be observed, once again, that the specimen with 3% voids under went more shear distortion than the specimen with 5% voids under the same loading program, though the over all axial or deviatoric strain is higher for the later specimen. This means most of the deformation of the specimen with 5% voids comes from compaction and not from shear deformation in agreement with the earlier conclusion reached based on the K-value.Thus, when shear deformation is considered to be the main cause of rutting, this ratio can also be used as another measure of resistance to rutting of asphalt concrete mixtures. The ratio of deviatoric strain and volumetric strain is incorporated into the rutting resistance index defined in section 7.3 in this chapter, as a measure of distortion.

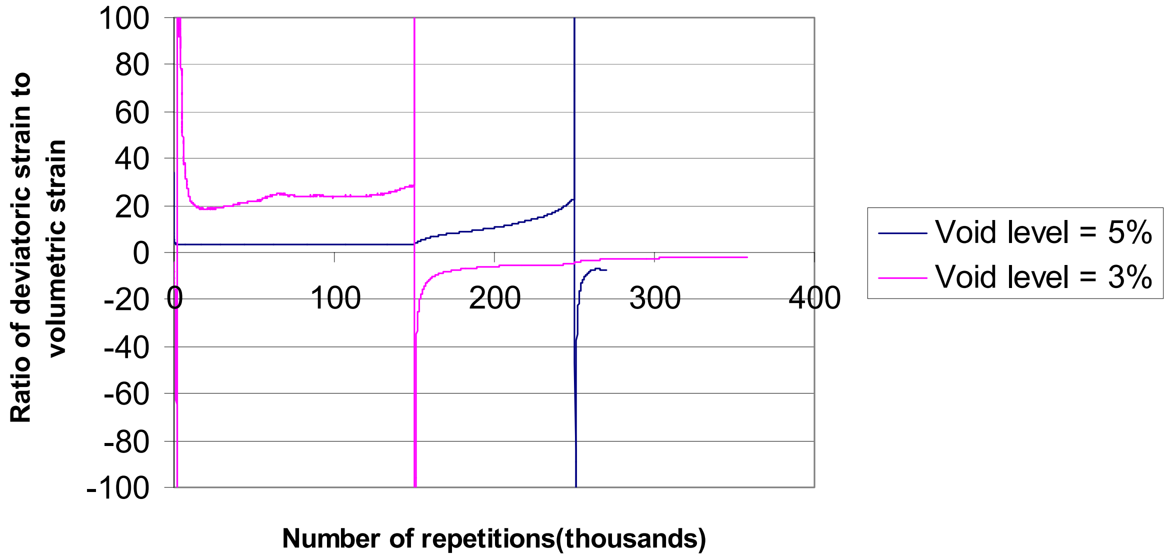


Figure 7.8 Ratio of deviatoric strain to volumetric strain

In the field, shear deformation manifests itself as a displacement of the material to the side of the wheel track and subsequent formation of a hump on each side of the wheel track in the area of rutting. In the laboratory, the shear deformation manifested itself in the form of large increase in the radial strain relative to axial strain and in some cases showed tendency towards formation of slip plane. Therefore it seems reasonable to conclude that the ratio of radial and axial plastic strains can also be used as measure of distortion.

7.2 The Bounding Surface Concept for Modelling Permanent Deformation

Under the action of repeated loading, asphalt concrete develops increased resistance to deformation, i.e, the strain amplitude subsequently decreases with each cycle until it stabilises on some value after large number of cycles. This phenomenon is called cyclic hardening. Understanding the cyclic hardening phenomenon is a key to modelling the accumulation of permanent deformation. There are several hardening models developed for various materials. Most of these models fall into the category of isotropic hardening models, developed to model the increase in resistance to deformation of materials subjected to monotonic loading.

According to isotropic hardening models, loading expands the yield surface to some current size as described by yield function and hardening rule. Unloading results in purely elastic deformation, because the entire unloading stress path is within the current yield surface.Unless the stress state exceeds the current yield surface, subsequent reloading will also result in purely elastic response. Thus, for repeated application of the same stress state, the classical isotropic hardening models produce no additional permanent deformation after the first load cycle. However, experimental results for many frictional materials such as granular materials and asphalt concrete clearly show that permanent deformation continues to accumulate under repeated application of the same stress state.

In an attempt to model the behaviour of soils under complex loading histories involving repeated loading, researchers developed the anisotropic hardening models. These models account for hysteretic behaviour and additional permanent deformation by allowing the yield surface to translate and/or contract on unloading Mroz et al[93]. The anisotropic hardening models are relatively complex involving several parameters. A simpler model was proposed for calculation of permanent deformation in granular pavement materials by Bonaquist and Witzack [94] based on the bounding surface concept originally proposed by Mroz et al.

7.2.1 The Bounding Surface Concept

In the bounding surface approach, two yield surfaces are used to describe the response of materials to repeated loading. The first one is the bounding surface which describes the current stress state to which the material is subjected. The bounding surface depends on material density and reflects the isotropic property of the material. The second yield surface is called the initial surface and describes the past loading history of the material. The initial surface expands during during repeated loading, while the bounding surface remains stationary. For stress states within the initial surface, elastic response is assumed. The size of the initial surface grows at a decreasing rate with the number of cycles of load applications and asymptotically approaches the bounding surface. When the initial surface and the bounding surface coincide the response will be purely elastic. If the material is subjected to a higher stress state a new bounding surface is established and permanent deformations can occur. The size of the bounding surface and the deformation behaviour under each loading applications can be obtained from plasticity models such as Desai's hierarchial single surface model [94]. The change in location of the initial surface can be determined from repeated load tests. Figure 7.9 illustrates the concept of bounding surface.

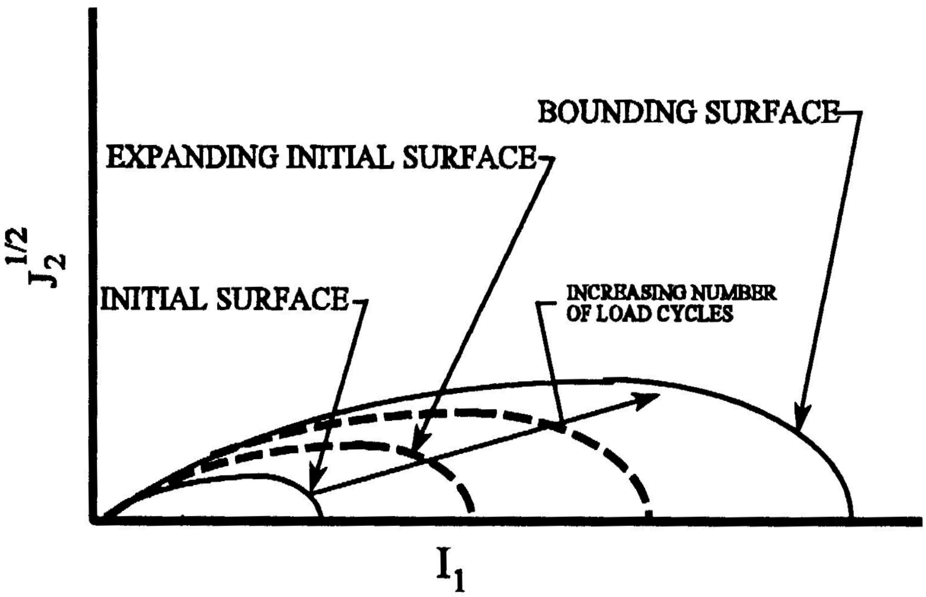


Figure 7.9 The bounding surface concept[94]

7.2.2 Cyclic Hardening Model

During cyclic loading the yield surface that defines the elastic range expands with the number of cycles, which cause cyclic hardening. The location of the initial surface expands quickly when the number of load repetitions are small, then asymptotically approaches the bounding surface after large number of load repetitions. To describe this deformation process, Bonaquist and Witzack [94] proposed the following model:

$$\xi_i = \xi_o + \left(1 - \frac{1}{N^{h_c}}\right)(\xi_b - \xi_o) \qquad 7.13$$

Where:

ξ_i = Plastic strain tajectory corresponding to the i-th intial surface

ξ_b = Plastic strain trajectory corresponding the bounding surface

ξ_o = Plastic strain trajectory coressopnding to the intial or in situ stress

N = number of load cycles

h_c = cyclic hardening parameter

The plastic strain trajectory is given by:

$$\xi = \int (d\varepsilon_{ij}^{p} d\varepsilon_{ij}^{p})^{\frac{1}{2}} \qquad 7.14$$

Where $d\varepsilon_{ij}{}^{p}$ is plastic strain increment tensor.

For the case of a repeated application of the same stress to an initially unstrained material, ξ_o = 0 and the cyclic hardening model of equation 7.13 reduces to:

$$\xi = \xi_b - \xi_i = \frac{\xi_b}{N^{h_c}} \qquad 7.15$$

Where ξ is plastic strain trajectory for load cycle N and other symbols are as defined in equation 7.13. In this study equation 7.15 was slightly modified to better fit the experimental results and expressed as follows:

$$\xi = \xi_b - \xi_i = \frac{\xi_b}{N^{h_c}} + q \tag{7.16}$$

Where q is a parameter and the other symbols are as defined in equation 6.14. The cyclic hardening parameter controls the rate of expansion of the initial yield surface and the magnitude of the permanent deformation for each load cycle. According to equation 7.15, the plastic strain per cycle continues to decrease with number of cycles, but experimental result shows that after large number load applications, the plastic strain per cycle tend to approach a certain constant value. Thus, the parameter q was added to take this fact into account and improve the ability of the model to fit the experimental results.

The accumulated permanent strain trajectory is the sum of plastic strain trajectories on each loading cycle and can be expressed as follows:

$$\sum \xi = \sum \left(\frac{\xi_b}{N^{h_c}} + q \right) \tag{7.17}$$

Equation 7.17 was fitted to the measured repeated load test results and the hardening parameter h_c and the parameter q were computed. Some of the specimens were tested under step loading. The parameters were calculated for each loading step using the corresponding bounding strain trajectory.The bounding strain trajectory was taken as the first cycle strain trajectory. Table 7.1 shows the calculated hardening parameter for specimens with varying binder content and air void contents and subjected to a cyclic deviatoric stress of 750 kPa and a constant confining pressure of 150 kPa.

Table 7.1 Values of the cyclic hardening parameter h_c

		Void Level (%)		
Binder content (%)		3	5	8
	4		1.172	1.047
	4.7	1.224	1.19	1.166
	5.4	0.928	1.128	1.178

It can be seen from the table that the values of the hardening parameter are more or less similar for the various specimens. This is hardly surprising as it has already been observed that the rate of deformation for these specimens approach a similar value. In terms of the accumulated permanent deformation, it is the first cycle (or the first few cycles) permanent strain that makes real difference among the specimens. Thus, it seems that it is the bounding strain trajectory that is sensitive to changes in void level and binder content and not the progression of deformation within the bounding surface. However, it is worth mentioning that the value of cyclic hardening parameter is less than 1 for the specimen with binder content of 5.4 and void level of 3. It has already been mentioned in the previous section that this specimens were observed to be the most unstable during testing showing plastic dilatancy right in the first loading stage. When the hardening parameter was computed for the specimens under consideration in the second and third loading stages, it was found that its values ranged between 0.18 and 0.4. All the specimens were observed to undergo plastic dilatancy and degradation in these stages as discussed in the previous section. Thus, small values of the hardening parameter and particularly those close to zero could be indicative of materials deteriorating and failing. Indeed, Bonaquist and Witzack [94] related the hardening parameter to the stress to strength ratio for granular materials and found that the hardening parameter is almost constant for stress to strength ratio of less than approximately 0.6. Above the stress to strength ratio of 0.6, the parameter (denoted by a_2) decreased dramatically as shown in Figure 7.10.

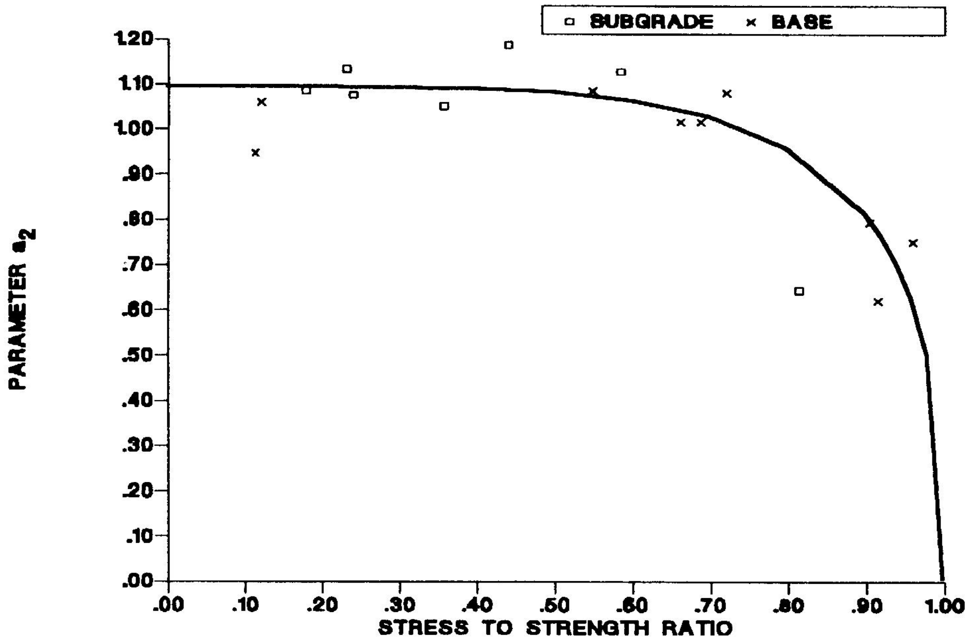


Figure 7.10 Cyclic hardening parameter [94]

It has been pointed out that the bounding strain trajectory under a given stress state can be determined using a general constitutive law. In this study, however, the measured bounding strain trajectory is used to demonstrate the ability of the cyclic hardening model to fit the test data.

It is clear that the effect of time of loading is not considered in the cyclic hardening model. The time dependence of permanent deformation will be considered in the next section where the strain decomposition approach is used. Experimental results in this study has shown that the magnitude of the time dependent component of the permanent strain drops dramatically after few cycles. Furthermore, the tests were conducted at a relatively fast loading rate of 10 Hz, a situation which diminishes the effect of time.

Figures 7.11, 7.12, 7.13 give the measured plastic strain trajectories and the ones computed using the cyclic hardening model. It can be observed that an excellent fit can be obtained by using the modified cyclic hardening model.The model was also fitted to a result of a test conducted at 25^{o}C (figure 7.14). The hardening parameter ranged between 0.85 - 0.96 for specimens tested at 25^{o} C in the first loading step. The relatively lower value of the hardening parameter reflects more gradual hardening due, most probably, to the more pronounced effect of binder viscosity at this temperature. As can be seen from figure 7.14 the model fits the test result very well.

It has, thus, been demonstrated that the cyclic hardening model, which is based on bounding surface concept fits the experimental data quite well. This model provides a rational basis for calculation of permanent deformation resulting from mixed loading. Once a constitutive relationship is established between stress and resulting bounding strain, the cyclic hardening model can be used to calculate the permanent strain resulting from repeated application of the stress.If a higher stress is applied, a corresponding bounding strain is established using the constitutive model and permanent deformation is calculated using the new bounding strain trajectory.

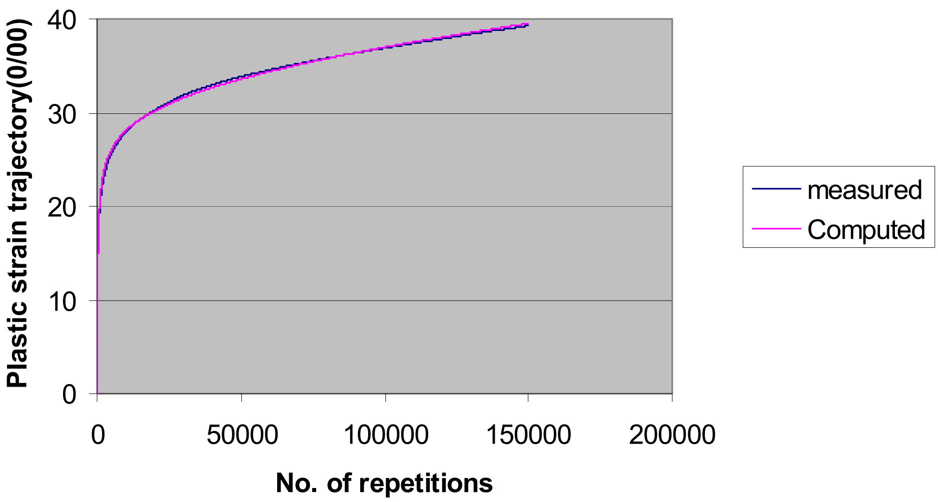


Figure 7.11 Measured and computed plastic strain trajectories (binder content 4.7% and void content 8%)

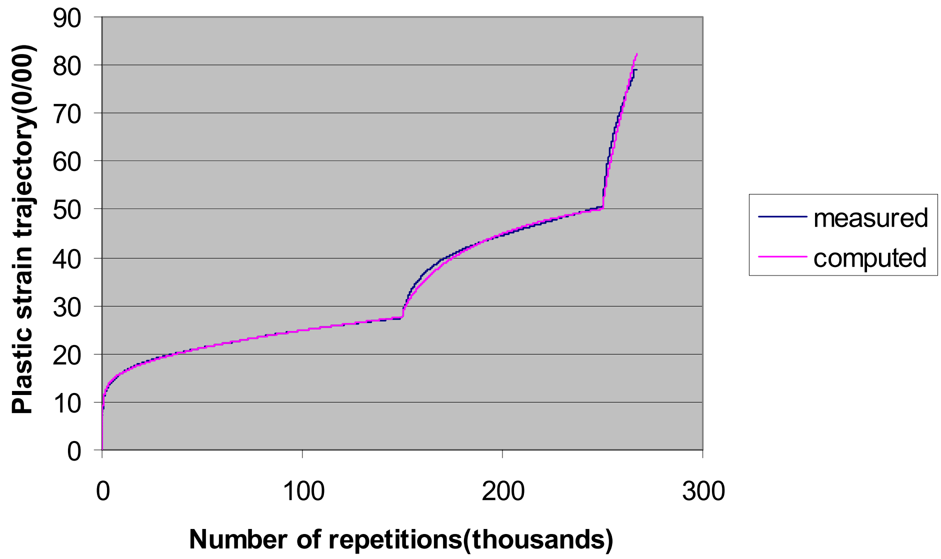


Figure 7.12 Measured and computed plastic strain trajectories (binder content 4.7%, void content 5%)

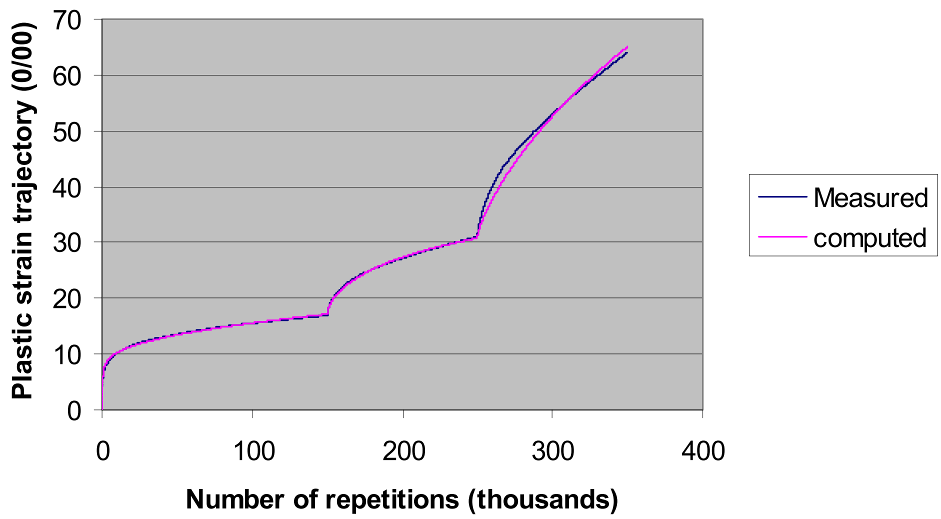


Figure 7.13 Measured and computed plastic strain trajectory (binder content 4.7%, void content 3%)

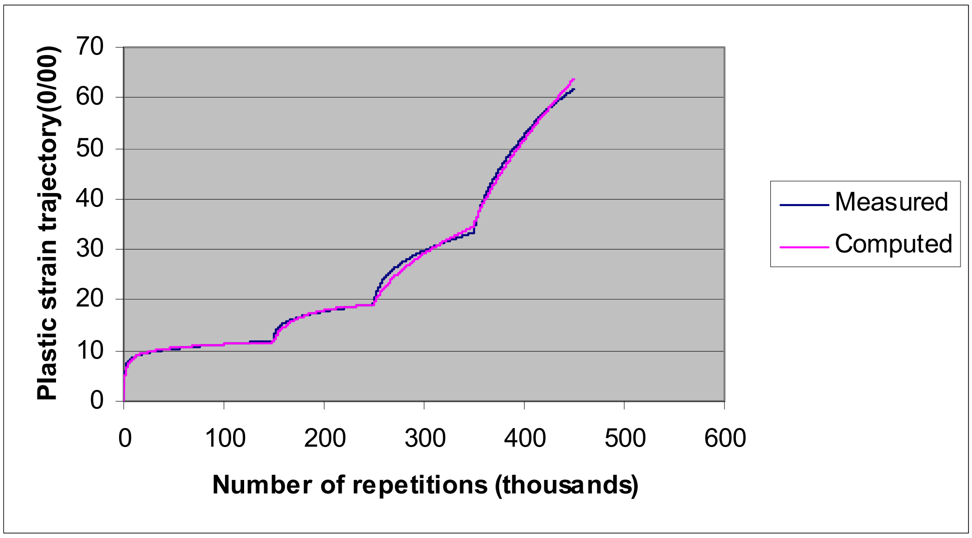


Figure 7.14 Measured and computed plastic strain trajectory at 25^{o}c (binder content 4.7%, Void level 5%)

7.3 Strain Decomposition Approach

Experimental evidence indicates that the total strain resulting from loading of asphalt concrete specimens has recoverable and irrecoverable elements some of which are time-dependent and some time-independent. In general, the total strain can be divided into four components and can be expressed as follows:

$$\varepsilon = \varepsilon_e + \varepsilon_p + \varepsilon_{ve} + \varepsilon_{vp} \qquad 7.18$$

Where:

ε	= total strain
ε_e	= elastic strain, recoverable and time-independent
ε_p	= plastic strain, irrecoverable and time-independent
ε_{ve}	= viscoelastic strain, recoverable and time-dependent
ε_{vp}	= viscoplastic strain, irrecoverable and time-dependent

The strain decomposition approach, developed by Uzan and co-workers[56], involves the resolution of the strain into these four components and modelling the components separately. The procedure used in this study to decompose the strain into its components is the same as that used by Uzan [95]. Results of confined compressive creep and recovery test conducted on asphalt concrete specimens with varying levels of void content and binder content were analysed using the strain decomposition approach. The specimens were tested with deviatoric stress of 750 kPa, in a form of a block pulse, and a constant confining stress of 150 kPa at a temperature of 50°C.The tests were conducted with a cycle time of 20 seconds; 10 seconds loading and 10 seconds unloading times.

Figure 7.15 shows the graph of total strain in a typical creep and recovery test cycle on asphalt concrete specimen and illustrates the four components of strain described by equation 7.18. It can be seen that, up on loading, an instantaneous strain consisting of elastic and plastic strains develops. As the specimen undergoes creep, both the viscoelastic and viscoplastic strains are accumulated. Upon removal of the load, the elastic strain disappears instantaneously and in the recovery period all or part (depending on the length of recovery period) of the viscoelastic strain recovers. At end of the cycle, the residual strain consists of the plastic and viscoplastic components plus the remainder of the viscoelastic strain that has not been recovered.

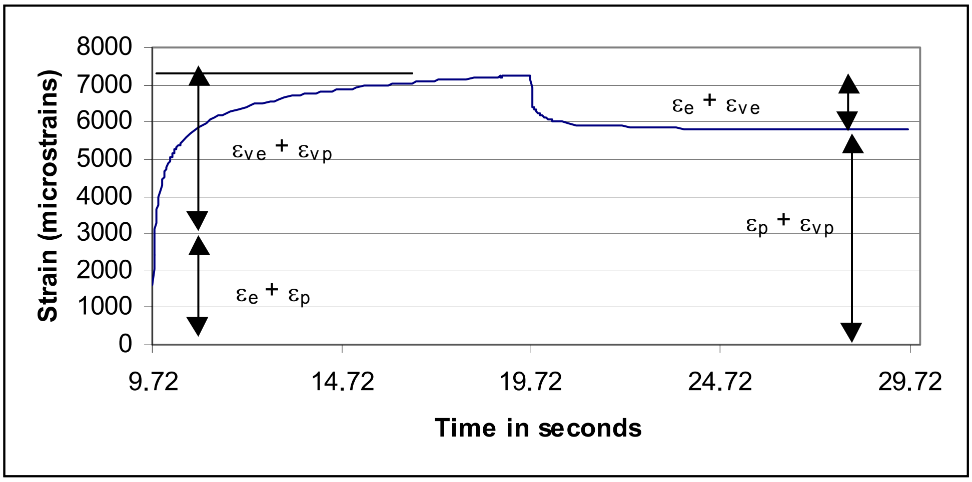


Figure 7.15 Total strain in a typical creep-recovery cycle

7.3.1 Calculation of Strain Components

The elastic strain component is obtained from the recovery curve and it is equal to the instantaneous decrease in the total strain which occurs at the moment the load is removed. The plastic strain component is then calculated by subtracting the elastic component from the instantaneous increase in total strain at the beginning of loading. The viscoelastic component is determined by fitting a power creep model, described in the next section, to the recovery curve. The remaining component, i.e, the viscoplastic strain is calculated from the creep curve by subtracting the sum of elastic, plastic and viscoelastic components from the total strain.

The residual viscoelastic strain can be calculated by using the superposition principle. However, it was found that the value of this residual strain is small and negligible. It was, therefore, decided to consider any remaining viscoelastic strain as part of the viscoplastic strain.

7.3.2 Elasto-Viscoplastic Model

The elastic, plastic, viscoelastic, and viscoplastic strain components computed following the procedure described above can be incorporated into a comprehensive elasto-viscoplastic model. Following Uzan's modelling approach, the creep compliance under step loading can be expressed as:

$$\frac{\varepsilon(t, N)}{\sigma} = D_e + D_p(N) + D_{ve}\frac{t^m}{1 + at^m} + D_{vp}(N)t^n \qquad 7.19$$

Where:

D_e	= elastic compliance (time-independent)
D_p	= plastic compliance (time-independent)
D_{ve}, a, m	= viscoelastic parameters
D_{vp}, n	= viscoplastic parameters, and
σ	= the stress

The plastic compliance, D_p, and viscoplastic compliance D_{vp} are functions of number of load repetitions. Under cyclic loading, the plastic and viscoplastic deformations accumulate and increase with number of cycles N. The increase of the accumulated plastic strains with the number of cycles is usually assumed to follow the power law.

$$D_p(N) = \frac{\varepsilon_p(N)}{\sigma} = \frac{\varepsilon_p(1)}{\sigma}N^{\alpha} \qquad 7.20$$

The viscoplastic strain is also assumed to be a power function of time within each cycle, which is considered to be a generalization of Maxwell's model.

$$\varepsilon_{vp}(t_n, N) = \varepsilon_{vp}(T_L, N-1) + [\varepsilon_{vp}(T_L, N) - \varepsilon_{vp}(T_L, N-1)]\left(\frac{t_n}{T_L}\right)^n \qquad 7.21$$

Uzan and others[56,95] have used the power law model to describe the accumulation of viscoplastic component with number of loading cycles. In this study, however, the magnitude of the viscoplastic component in a cycle was found to decrease sharply with the number of cycles. In fact, after few cycles the viscoplastic component of the creep compliance was found to reduce to a negligible amount. A logarithmic function, described in equation 7.22, was found to better fit the accumulation of the viscoplastic compliance with number of cycles and was, therefore adopted.

$$D_{vp}(N) = D_{vp}(1) + \beta \ln(N) \qquad 7.22$$

Where, in equations 7.20, 7.21 and 7.22:

α, β = material parameters

T_L = duration of loading time during one cycle

t_n = time measured from the beginning of cycle N.

The material parameters, with the exception of α and β, were determined using the data of the first creep and recovery cycle. The parameters α and β were calculated by fitting the models given by equations 7.20 and 7.22 to the accumulated plastic and viscoplastic compliances respectively.

Figures 7.16 and 7.17 show examples of the computed and measured compliances for some cycles. Only axial compliances were considered in this analysis. It can be seen that the ability of the model to fit the test data is reasonably good for the initial cycles but appears to decrease with increase in the number of cycles.This is because the model tends to over estimate the viscoelastic component within a cycle, but as most of this component is recoverable, its effect on permanent strain is minimal.

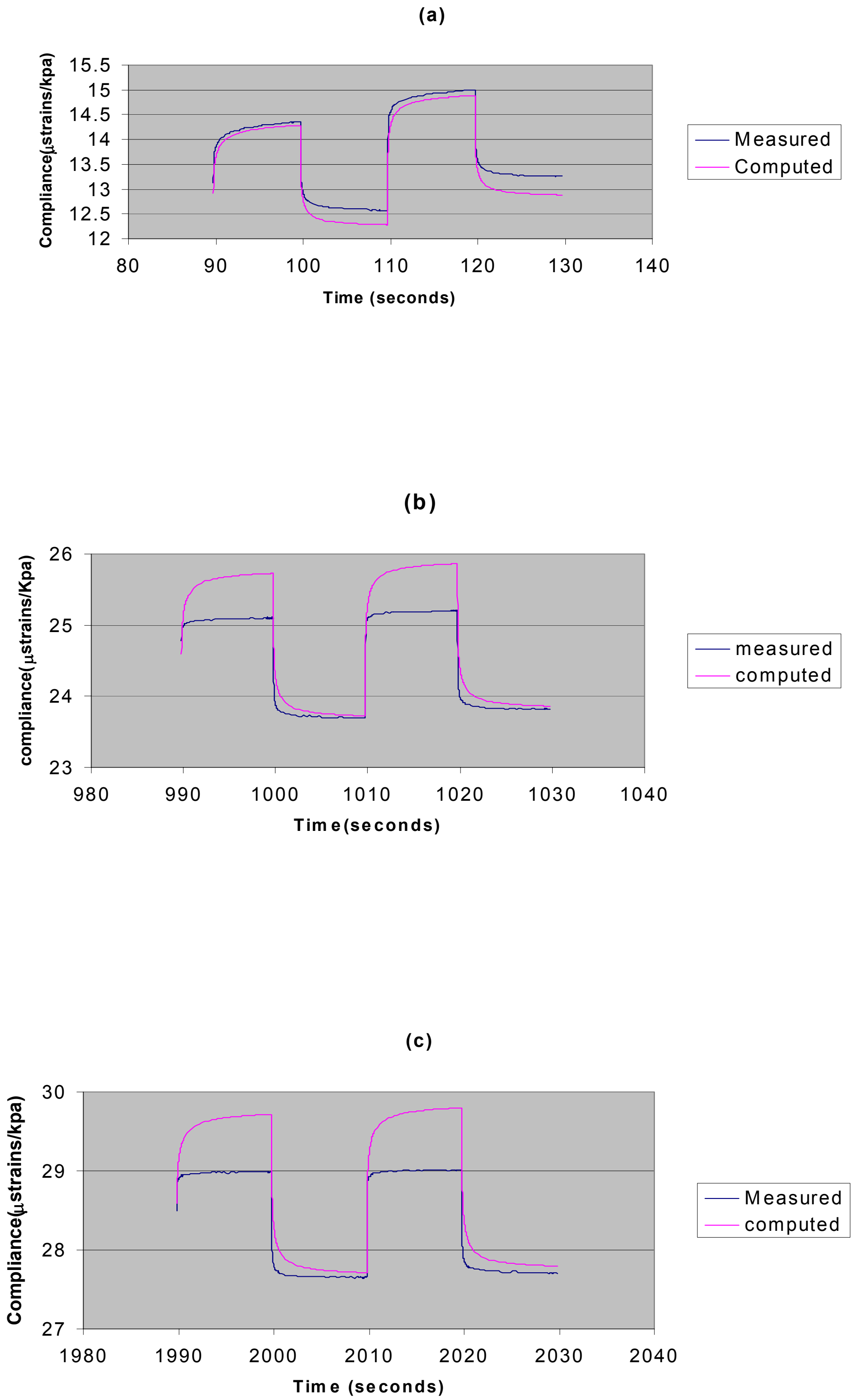
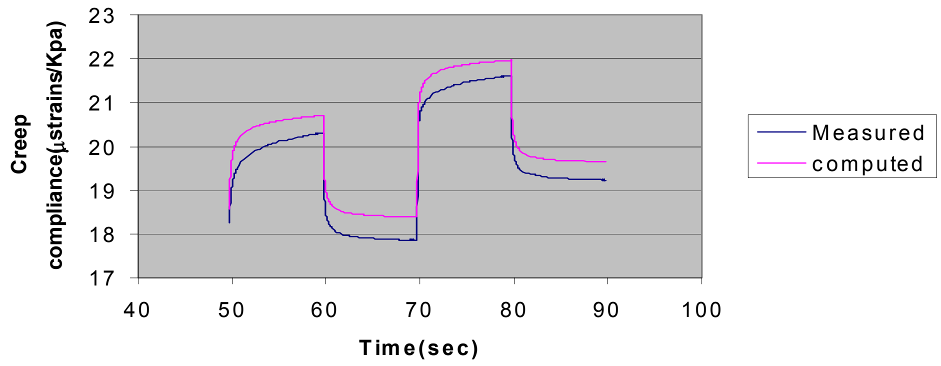


Figure 7.16 (a), (b) and (c): Measured and computed compliances for specimens with binder content of 4.7% and void level 5%.

Thus the model fits the accumulated permanent deformation (the sum of plastic and viscoplastic components) quite well as shown in Figure 7.18. The test result shows that the time dependence of the strain diminishes with increasing number of cycles. In particular, the viscoplastic component, while very substantial in the first few cycles, decreases sharply with the number of load applications. This observed behaviour might be due to the diminishing role of the binder, which imparts the time dependence properties to the mixture, at this relatively high temperature (50^{o}C).

(a)



(b)

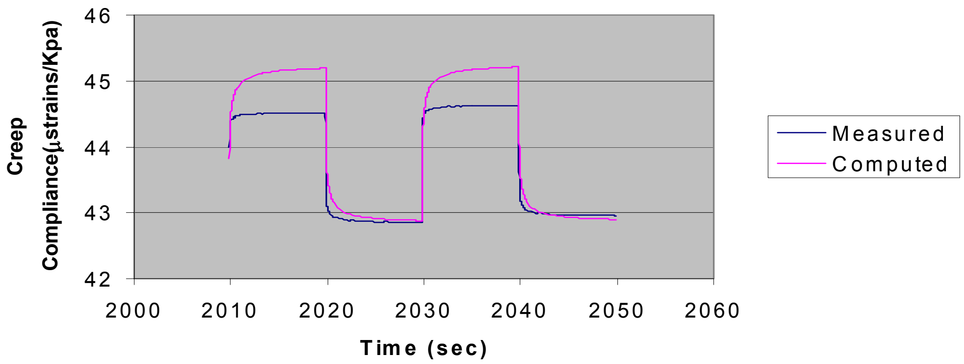


Figure 7.17 (a) and (b): Measured and computed compliances for specimens with binder content of 4.7% and void level 8%

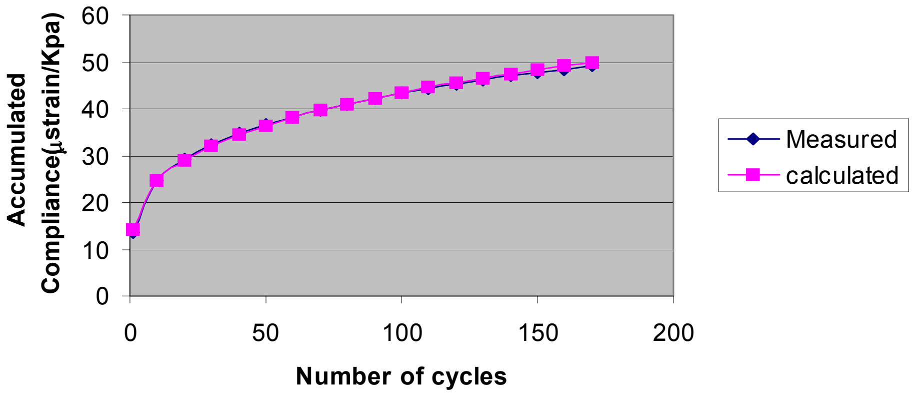


Figure 7.18 Accumulated axial compliance

The elasto-viscoplastic model described above provides a comprehensive approach for analysis of test results. A constitutive model can be developed for each of the strain components by relating them to the stress. Such models are known to be non-linear functions of stress[56]. One of the objectives of this study was to investigate the sensitivity of material parameters in the elasto-viscoplastic model to changes in binder content and void content, with a view of defining a simple index in terms of these parameters, which can be used as a measure of resistance to permanent deformation of asphalt concrete mixtures. This will be discussed in the next section.

7.3.3 Sensitivity of Material Parameters to Changes in Volumetric Properties

If the material parameters are sensitive to changes in volumetric composition, it will be possible to define some measure of resistance of asphalt concrete mixtures to permanent deformation in terms of these parameters. Such measure of resistance can be linked to mixture design for the purpose of ranking mixtures or to performance related specifications. The material parameters were determined for specimens with various levels of void content and binder content using the procedures described previously. The specimens were tested under the same loading conditions at temperature of 50°C. Table 7.2 shows the values of the parameters for some of the specimens.

Table 7.2Values of Material Parameters

Binder content	Void content	D_{ve}	a	m	$D_{vp}(1)$	n	$D_p(1)$	D_e
4.7	8	4.87	3.419	0.766	10.061	0.171	4.158	0.996
4.7	5	3.19	2.698	0.775	5.485	0.219	2.45	0.917
4.7	3	3.686	2.517	0.785	2.874	0.1	1.714	0.779
4.0	5	2.701	2.401	0.742	3.194	0.116	1.077	0.756
5.4	5	4.497	2.705	0.74	7.007	0.247	3.63	1.00

In investigating the sensitivity of the material parameters, emphasis was placed on the parameters m, n, $D_{vp}(1)$, and $D_p(1)$ because of their direct influence on the residual or permanent deformation. It can be seen from the Table 7.2 that, the value of m varied with in a narrow range from 0.74- 0.785, and seems insensitive to changes in binder content and void content.The value of n varied between 0.171 and 0.247. There is no clear trend in change of n with changes in void level. But the parameter n increased with an increase in binder content, showing somewhat consistent trend. It is worth to note that the parameters m and n describe the time dependence of the deformation, a property due to the binder. As such they might be sensitive to changes in binder type. But as the specimens in this study were all made from the same binder, it was not possible to verify this assumption.

The two parameters $D_p(1)$ and $D_{vp}(1)$ appear to be the most sensitive of all. $D_p(1)$ and D_{vp} (1) are the ratio of the first cycle plastic and viscoplastic axial strains to the applied deviatoric stress, respectively. As can be observed from table 6.2, these two parameters increased with increase in void level. They also increased with increasing binder content. These trends clearly follow the trend of the accumulated axial permanent deformation described in preceding chapter.

It has already been pointed out that the rate of accumulation of permanent deformation is more or less the same for specimens under study.This clearly indicates that it is the first cycle permanent strain that makes a difference in the total permanent deformation of the specimens rather than the rate of accumulation of the deformation. Thus in ranking mixtures made from the same material but with varying proportions of the components, the first cycle permanent strain would be more relevant than the creep rate.It has already been shown using the bounding surface concept that, modelling the accumulation of permanent strain based on the first cycle (or first few cycles) permanent strain gives very good correlation with experimental data.

7.3.4 Measure of Resistance

In general, the larger the values of D_{vp} (1) and $D_p(1)$, the larger is the permanent deformation. Also, relatively larger values of m and n would indicate specimen under going large time dependent deformation, be it viscoplastic or viscoelastic. It may thus be possible to define some index in terms of these four parameters that may be used to evaluate mixtures. Let a certain index I be defined as follows:

$$I = (m+n)(D_{vp}(1) + D_p(1)) \qquad 7.23$$

The value of I would be large if either or both of the factors (m+n) and ($D_{vp}(1)+D_p(1)$) are large, i.e., a large value of I is associated with a large deformation. If we define another index as an inverse of I, it would provide a measure of resistance to deformation. Let R be such measure of resistance and defined as follows:

$$R = \frac{1}{(m+n)(D_{vp}(1) + D_p(1))} \qquad 7.24$$

The larger the value of R, the smaller the deformation, i.e., the larger is the resistance to deformation. The parameters D_{vp} (1) and $D_p(1)$ are first cycle viscoplastic and plastic compliances respectively and as such can be considered as the inverse of viscoplastic and plastic moduli (the ratio of the applied stress to viscoplastic and plastic strains). Replacing the two compliance parameters in equation 7.24 with their inverses, denoted by H_{vp} and H_p, and simplifying results in the following expression for the resistance parameter, R.

$$R = \frac{H_{vp}H_p}{(m+n)(H_{vp} + H_p)} \qquad 7.25$$

The values of R were computed for specimens whose parameters are shown in Table 7.2. Figure 7.19 shows the variation of R with binder content. The index R has the same dimension as the modulus. R correctly ranked the specimens according to the accumulated axial permanent strain. For specimens tested under the same stress conditions, the lower the void content the lower the accumulated axial permanent strain. Also the lower the binder content, the lower is the accumulated axial permanent strain. However, as stated previously, permanent deformation or rutting in asphalt concrete is caused by both densification and shear deformation. It has been pointed out in the previous section that specimens with low void content (less than 3%) and low binder content are relatively more susceptible to shear deformation.In analysis of triaxial tests, the difference between the axial strain and the radial strain is often used as a measure of shear or distortional strain. In this study the ratio of permanent shear strain to the permanent volumetric strain was found to be constant as discussed in section 7.2. This ratio can be expressed in an incremental form as:

$$r = \frac{\Delta\varepsilon_D}{\Delta\varepsilon_V} \qquad 7.26$$

Where:

$\Delta\varepsilon_D$ = Permanent deviatoric strain increment

$\Delta\varepsilon_V$ = Permanent volumetric strain increment

A relatively high value of this ratio indicates specimen deforming with little volume change, i.e., shear deformation. Thus, this ratio may be incorporated into a resistance index in equation 7.25 above to provide a composite measure of resistance to permanent deformation, RI, defined in equation 7.27 below.

$$RI = \frac{H_{vp}H_p}{r(m+n)[H_{vp}+H_p]} \qquad 7.27$$

The composite index RI was calculated for the specimens under consideration. Figure 7.20 shows the variation of RI with void content and binder content. It can be observed that the variation of RI with void content as well as binder content tends to show some maximum value, indicating the existence of an optimum binder content and void content which gives the highest resistance. This behaviour is similar to that shown by the empirical Marshal parameters. The index is relatively high for the specimen with void content of about 5% and binder content of 4.7% (optimum according to Marshal method).

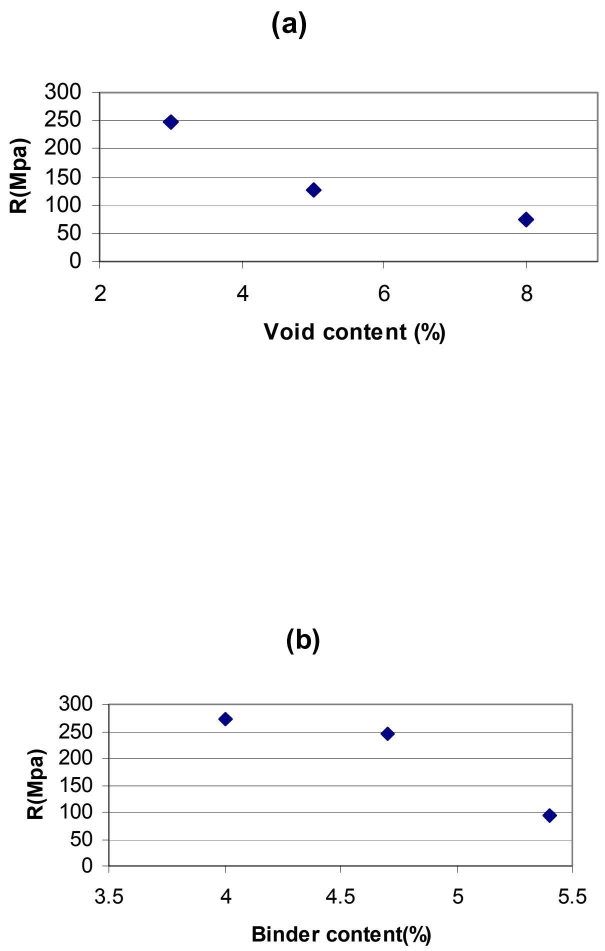


Figure 7.19 (a) and (b) Resistance Index, R

This provides a proof of validity for the conventional 4% void level, usually targeted in asphalt concrete compaction. This level of void content appears to be a compromise between the need to decrease deformation resulting from compaction by making the asphalt material dense and the need to decrease shear susceptibility by making it less dense. The index defined above can be linked to mixture design to evaluate and rank mixtures. The parameters of the index can be easily determined from few cycles of creep and recovery test. The index takes both the total permanent deformation and shear susceptibility into account.If proved for other materials by further tests, this index could provide a valuable tool in performance related specification of asphalt concrete mixtures as a simple measure of performance with regard to rutting.

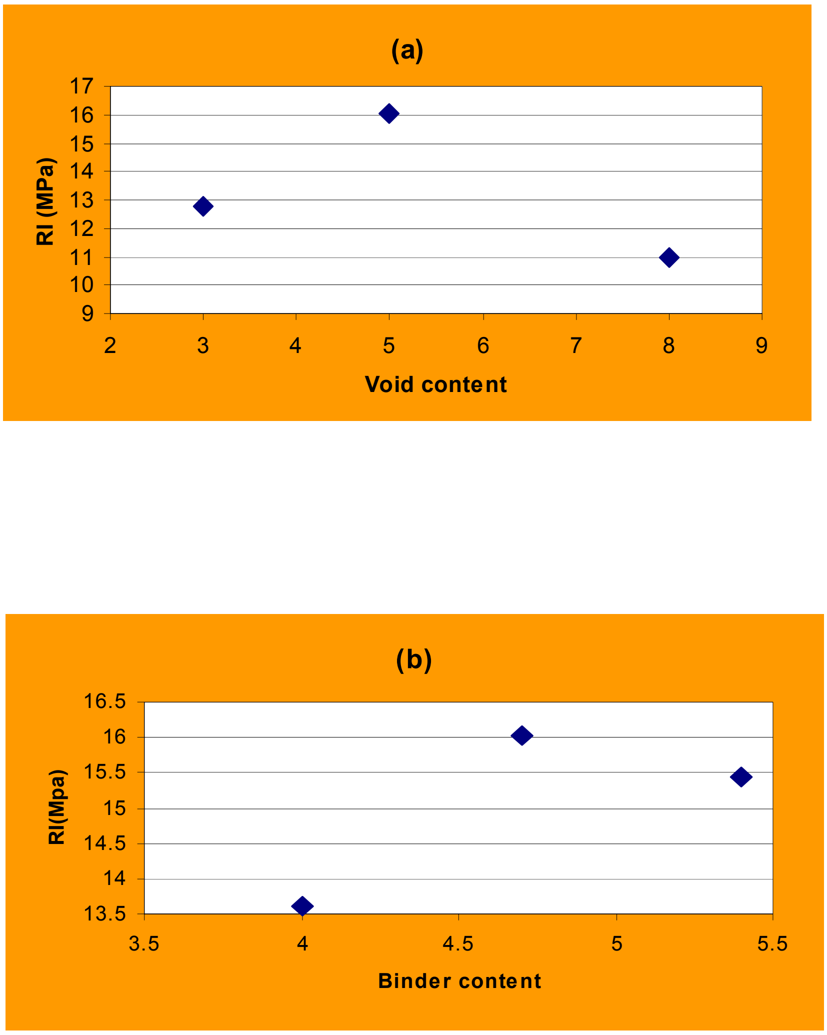


Figure 7.20 Composite resistance index RI

CHAPTER 8
CONCLUSIONS AND RECOMMENDATIONS

The research work reported in this thesis dealt with the study of the permanent deformation characteristics of asphalt concrete mixtures. Several asphalt concrete specimens were tested in repeated load triaxial and triaxial creep and recovery tests. Tests were conducted at two temperature levels. The effect of volumetric composition, loading and temperature on permanent deformation behaviour of asphalt mixtures was evaluated. Emphasis was placed on methods and parameters used to evaluate the resistance to permanent deformation of mixtures. These methods and parameters were evaluated with regard to their sensitivities to changes in volumetric composition. Substantial effort was made to understand the mechanism of deformation in asphalt concrete mixtures and to model the development of permanent deformation under repeated loading. An extensive review of literature related to permanent deformation properties of asphalt concrete was also conducted.

Based on triaxial creep and recovery test, a new index, which may be used to evaluate and compare mixtures for their resistance to permanent deformation is defined. The index is simple to calculate and is found to be sensitive to changes in binder content and void content, which might make it suitable for mixture design purposes.

8.1 Conclusions

Based on the literature review, testing and analysis of test results, and modelling effort undertaken in this research work, the following conclusions were made.

1 A large amount of literature on permanent deformation properties of asphalt mixtures exists. Most of these studies concentrated on evaluation of the effect of component material properties, most notably aggregate gradation, aggregate angularity, and binder type (grade) on permanent deformation (or rutting) properties of mixtures. The studies have come up with varying conclusions some of which are in contradiction with one another. The problem appears to be the fact that the studies used different approaches in testing and evaluation of

the test results. Different parameters were used in the literature to evaluate the resistance to rutting (permanent deformation) of asphalt mixtures. This makes it difficult to compare the various studies and draw firm conclusions.

2. Currently, there is no comprehensive model for deformation of asphalt concrete. Generally two approaches were used in an attempt to model asphalt concrete deformation; the continuum mechanics approach and the micromechanics approach. In the continuum mechanics approach, the theory of linear viscoelasticity and, to a limited extent, the theory of elasto-viscoplasticity were used. The theory of viscoelasticity might be appropriate to model deformation at low temperatures and high frequencies of loading. But at high temperatures and slow rate of loading, where rutting or permanent deformation is of crucial importance, this theory might not be appropriate because it fails to take account of the time independent plastic component of the strain. While elasto-viscoplasticity can be used to take account of most of the behaviour of asphalt concrete under load, it is sophisticated and requires substantial effort in material testing and computations. Asphalt concrete is a particulate composite material. The micromechanics approach has been applied to composite materials with some success and it appears to be a novel approach to take the distinct properties of aggregates, the binder, and their interface into account. However, research into the application of the micromecahnics approach appears to be just beginning.

3. In the field, asphalt pavements are subjected to three dimensional stresses. Therefore, in order to be able to predict the performance of asphalt concrete material based on laboratory test, the testing should be conducted under loading conditions which simulates the field loading conditions as closely as possible. Thus, it is necessary to conduct triaxial stress tests under conditions of temperature, loading rate, and stress level that mimic the field conditions under which the pavement is expected to serve.

4. The volumetric composition, i.e., binder content and void content, greatly influences the permanent deformation characteristics of asphalt concrete mixtures. This is evidenced by the results of repeated load triaxial test conducted on several specimens with varying levels of binder content and void content. In particular the combination of high binder content and low void content is found to produce a mixture that can become unstable and dialate. Dense mixtures with void levels of 3% or less are more susceptible to shear deformation.

5. The permanent deformation response of asphalt mixtures is highly dependent on the loading conditions. In particular, the effect of confining stress on permanent deformation is very significant. Thus it is necessary to find ways of estimating field confining stress and to use this in laboratory testing of materials.

6. Parameters that are traditionally used to evaluate the resistance of mixtures to permanent deformation such as the slope and intercept of power model are found to be not suitable for purposes of comparison of mixtures made from the same material but with varying proportions of the components. These parameters do not appear to be sensitive to changes in volumetric composition and do not show consistent trends. In addition the parameters are calculated based on uniaxial deformation but proper evaluation of mixtures for their resistance against rutting requires consideration of the lateral deformation as well. The difference in the accumulated permanent deformation of mixtures made from the same materials but with varying proportions of the components occurs in the first few cycles of loading and the rate of accumulation of permanent strain is practically the same.

7. Rutting is caused by both densification and shear deformation. The shear deformation manifests itself in the form of large lateral deformation relative to axial deformation in triaxial testing. Thus methods of mixture evaluation that are based only on uniaxial deformation may give misleading results. Therefore it is necessary to use methods that take both the axial and lateral deformation into account such as the stress-dilatancy theory and the ratio of deviatoric and volumetric strains described in this thesis to get better insight into the resistance of mixtures against permanent deformation.

8. The bounding surface plasticity concept is suitable for modelling the development of permanent deformation of asphalt concrete under repeated loading. It is suitable for taking mixed loading into account and can also be implemented in pavement structural analysis methods such as the finite element method should appropriate constitutive model for asphalt concrete becomes available.

9. The elasto-viscoplastic model based on strain decomposition approach provides a convenient method for analysis of creep and recovery test results and for study of the various components of strain. Results of creep and recovery tests clearly indicated that the strain

consists of elastic, plastic, viscoelastic and viscoplastic components. However, the magnitude of the viscoplastic component was found to diminish sharply after few cycles loading.The sum of the plastic and viscoplastic components of the strain, i.e. the permanent strain, as calculated using this model fits the measured permanent strain quite well.

10. The rutting resistance index defined in this study based on the strain decomposition approach is sensitive to changes in volumetric composition and it provides a simple method for evaluation of mixtures for their resistance to rutting. This index can be used at the mixture design stage as a simple measure of performance with regard to rutting and may enable selection of rut resistant mixture.

8.2 Recommendations for Further Research Work

Asphalt concrete is a complex material whose properties depend on composition, level and rate of loading, temperature and other environmental factors. As yet, there is no comprehensive constitutive model for asphalt concrete that takes all the relevant factors into account. Testing and material characterization under realistic conditions is time consuming and expensive. Proper prediction of permanent deformation requires the development and use of more advanced material models. On one hand there is a need to develop and use simple measures of performance for purposes of mixture design and selection, and on the other hand a more comprehensive material model is required for implementation in pavement structural analysis models for the purpose of calculation of the response of the material to various loading conditions and thereby predict the distress. Based on observations made during this research work, the following recommendations are made:

- The triaxial test appears to be the only realistic method for characterization of asphalt concrete materials. But it is time consuming and expensive. Research should be directed towards making this test faster and more efficient. It is also necessary to standardize the test procedure so that test results can be compared. Further, the determination of appropriate levels of confining stress should be given extra attention.
- Simple measures of performance such as the rut resistance index defined in this study are valuable for purpose of mixture design and selection. The applicability of this index to other mixture types and its use in performance related specifications should be explored.

- Substantial research effort should be made to develop a comprehensive constitutive model for asphalt concrete. Due consideration should be given to the micromechanics approach in developing such constitutive model.

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APPENDIX: VOLUMETRIC COMPOSITION OF SPECIMENS TESTED

Table 1: Void Content

Binder Content = 4.7%			
Specimen	Target void content	Measured void content	Average void content
1-1	3	2.82	2.63
1-2	3	2.44	
2-1	3	2.79	2.93
2-2	3	3.07	
3-1	3	2.51	2.77
3-2	3	3.03	
3-3	3	2.8	2.8
1-1	5	5.09	5.1
1-2	5	5.11	
2-1	5	4.93	5.07
2-2	5	5.21	
2-3	5	5.4	5.7
2-4	5	6.00	
3-1	5	4.89	4.84
3-2	5	4.79	
3-3	5	5.3	5.3
1-1	8	8.23	8.12
1-2	8	7.98	
2-1	8	9.04	8.69
2-2	8	8.34	
3-1	8	8.26	8.16
3-2	8	7.97	

Table 2: Void content

Binder content= 5.4%			
Specimen	Target void content	Measured void content	average void content
2-1	3	2.64	3.03
2-2	3	3.42	
1-1	5	5.44	5.3
1-2	5	5.15	
2-1	5	4.76	5.2
2-2	5	5.64	
3-1	5	5.18	5.13
3-2	5	5.07	
1-1	8	7.47	7.73
1-2	8	7.99	
2-1	8	8.14	8.48
2-2	8	8.82	

Table 3: Void content

Binder content = 4.0%			
Specimen	Target void content	Measured void content	Average void content
1-1	5	5.9	5.79
1-2	5	5.67	
2-1	5	4.99	5.21
2-2	5	5.42	
2-3	5	5.69	5.69
3-1	5	4.8	5.1
3-2	5	5.4	
1-1	8	9.02	8.83
1-2	8	8.64	

Table 3: Void content

Binder content = 4.0%			
Specimen	Target void content	Measured void content	Average
2-1	8	8.42	8.34
2-2	8	8.26	

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