EVALUATION OF RESISTANCE TO PERMANENT DEFORMATION IN THE DESIGN OF BITUMINOUS PAVING MIXTURES

by

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ABSTRACT

The failure of recipe specifications to guarantee good performance together with increased devolvement of responsibility for road construction and maintenance to the private sector, and also the harmonisation of product standards with the European Community, have produced greater scope and need for the use of rational methods for both the design and specification of bituminous paving mixtures in the U.K.

This research has been concerned with the development and use of a simple mechanical test for assessing resistance to permanent deformation, principally for the purposes of mixture design but also with consideration to application in performance specification. Attention focussed on a repeated load axial compression test developed in earlier research at the University of Nottingham. The simplicity and relatively low cost of this test make it suitable for routine use and it has been shown to correspond well in the ranking of mixtures with small and medium scale wheeltracking tests.

However, it has been found that a particular test configuration may discriminate disproportionately between mixtures of different types, or even with different aggregates, with regard to their likely performance in the road. Binder stiffness, as generated by test temperature and load duration, and the application of confinement were shown to be of particular significance in this respect. It has also been demonstrated that the method of specimen compaction can affect the result of the test.

These are important considerations, particularly for performance specification where a quantitative assessment of performance must be made, and emphasise the need for standardisation of procedures for both specimen preparation and testing.

Recommendations for test conditions have, therefore, been made based on the findings of the research, and it has been confirmed that the Percentage Refusal Density equipment is an acceptable means of specimen preparation, at least for the purposes of mixture design.

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LIST OF ABBREVIATIONS

AAMAS	Asphalt-Aggregate Mixture Analysis System
Agg.	Aggregate
AI	Asphalt Institute
ARRB	Australian Road Research Board
BACMI	British Aggregates Construction Materials Industries
BS	British Standard
CBR	California Bearing Ratio
CC	County Council
CHSST	Constant Height Simple Shear Test
CROW	Centre for Research and Contract Standardisation in Civil and Traffic Engineering
CRR	Centre de Recherches Routières
DBM	Dense Bitumen Macadam
Dia.	Diameter
DOT	Department of Transport
Eq^n.	Equation
ESAL	Equivalent Standard (80 kN) Axle Load
HDM	Heavy Duty Macadam
HRA	Hot Rolled Asphalt
IBM	International Business Machines (*company name*)
LAMBS	Large Aggregate Mixes for Bases
LCPC	Laboratoire des Ponts et Chaussées
LVDT	Linear Variable Differential Transformer
Max.	Maximum
Min.	Minimum
MRL	Materials Reference Library
NAPA	National Asphalt Pavement Association
NAT	Nottingham Asphalt Tester
NCHRP	National Cooperative Highway Research Program

LIST OF ABBREVIATIONS (continued)

NDM	Nuclear Density Meter	
PC	Personal Computer	
PI	Penetration Index	
PRD	Percentage Refusal Density	
PTF	Pavement Test Facility	
RBA	Refined Bitumen Association	
Ref.	Reference	
Req^{d}.	Required	
RLA Test	Repeated Load Axial Test	
SABITA	South African Bitumen and Tar Association	
SBR	Styrene-Butadiene-Rubber	
SBS	Styrene-Butadiene-Styrene	
SHRP	Strategic Highway Research Program	
SMA	Stone Mastic Asphalt	
TRL	Transport Research Laboratory	
TRS	Tarmac Roadstone Southern	
UK	United Kingdom	
US(A)	United States (of America)	
VFB	Voids Filled with Bitumen	(see Equation 1.2)
VMA	Voids in Mineral Aggregate	(see Equation 1.1)

LIST OF SYMBOLS

d	Sieve size
D	Maximum aggregate size
F	Filler content, percentage by total mass of aggregate
h	Depth or thickness
hr.	Hour
H_Z	Hertz
in.	Inch
kg	Gramme × 10^3
M_B	Binder content, percentage by total mass of mixture
m	Metre
km	Metre × 10^3
mm	Metre × 10^{-3}
μm	Metre × 10^{-6}
n	An exponent between 0 and 1
n	Increment in load applications or duration at which strain or deformation is recorded
N	Number of increments at which strain or deformation is recorded
N	Newton
kN	Newton × 10^3
Pa	Pascal
kPa	Pascal × 10^3
GPa	Pascal × 10^9
Pen	Penetration in m × 10^{-4}
PSI	Pounds per square inch (1 PSI ≡ 6.895 × 10^3 Pascal)
t	time
v	velocity
V_B	Binder content, percentage by volume
V_V	Air void content, percentage by volume
°C	Degrees Celsius

LIST OF SYMBOLS (continued)

Δ *(x)*	Change *(in value of x)*
ϵ	Strain or deformation
µstrain	Strain × 10^{-6}
Σ *(x)*	Sum *(of values of x)*
$	US Dollars
£	Pounds Sterling
%	Percent

CHAPTER ONE
INTRODUCTION

1.1 The Importance of an Effective Transport System

An effective transport system is essential for the economic and social development of any nation. Such development in Britain prior to the mid-eighteenth century was hampered by the state of the road network which had decayed since the departure of the Romans. Those roads which did exist were impassable for much of the year. Although ships plied the coastal trade it was frequently cheaper and quicker to import goods into London by sea from Europe than to carry them overland from the interior of Britain (1). The beginnings of industrialisation gave rise to a pressing need for a means for the movement in bulk and at low cost of raw materials, coal for fuel in particular, and finished goods. This need was met first by the construction of the canals, beginning in the 1760s and then, from the 1830s, by the growth of the railways, thus ensuring the success of Britain's industrial revolution.

It was not until the development of the internal combustion engine that use of the highways for the transportation of passengers and freight began to increase. The growth in motorised transport was rapid after the First World War and by 1939 there were almost half a million goods vehicles on Britain's roads (1). The construction of the motorway network, beginning with the Preston bypass which opened in 1958, and the general development of the road system has contributed greatly to the success of road transport. The facility to collect and deliver from door to door at low cost and the personal mobility afforded by the motor car has led to the dominance over other forms of land transport which the roads enjoy today. Recent statistics (2) show that 89 percent of inland freight and 94 percent of passenger mileage in the UK are carried by road.

1.2 The Use of Bituminous Materials in Road Construction

An early problem associated with motorised traffic was the generation of dust from unbound road surfaces, which caused a considerable nuisance. A solution was found in the application of tar to bind the surface or the use of naturally occurring asphalts such as Trinidad Lake Asphalt (3). The principle of using such binders in combination with mineral aggregates as a road construction material has endured and flourished, though bitumens refined from petroleum have now replaced tar and natural asphalts for the vast majority of road paving applications. In addition to their qualities as a surfacing material, well designed bituminous mixtures are capable of withstanding high traffic loads and flexible pavement construction, in which the main structural component is the bituminous layer, is extensively used. In 1990, flexible construction accounted for 98 percent of the length of new trunk road for which contracts were let in England (4). In 1993, 31 million tonnes of bituminous material were used in road construction (57%) and maintenance (43%), while the total road paving demand for ready-mixed concrete, the principal alternative road construction material, was approximately 6 million tonnes (4).

1.3 Pavement Design

Developments in UK road pavement construction had occurred during the late eighteenth and early nineteenth centuries due chiefly to the efforts of two Scotsmen, Thomas Telford (1757-1834) and John Loudon M^{c}Adam (1756-1836). Both men employed sound, though differing, engineering principles in their road construction techniques. Telford's roads were of substantial construction with the emphasis on providing a strong foundation, while M^{c}Adam relied on the strength of the native soil, perhaps not unreasonable for the loads at that time, provided it was kept dry and he, therefore, concentrated on constructing an impermeable surface.

Despite this early recognition of the road pavement as an engineering structure it is only comparatively recently that principles of structural design have been applied for the determination of component layer thickness based upon estimation of loading and material properties. Developments both in techniques for the analysis of pavement structures and in the characterisation of materials, much of which derived from research carried out in the 1960s and 1970s (eg 5,6), have led to the introduction of analytical methods for the design of flexible pavements, such as the Shell method (7) which was first published in 1978. Prior to the advent of analytical procedures pavement thickness design was largely empirical, being based on the observed performance of existing roads. The chief limitations of this approach are that designs cannot reliably be produced for traffic loadings in excess of those already carried by in-service pavements and that new or improved materials cannot quickly or easily be assessed and incorporated. However, wholly empirical procedures were in use for the design of trunk roads in the UK until 1984, when a method for the design of flexible pavements based upon an analytical interpretation of existing performance data was published by the then Transport and Road Research Laboratory (8).

Figure 1.1 shows, schematically, a typical flexible pavement structure comprising bituminous layers placed upon a granular sub-base overlying the subgrade.

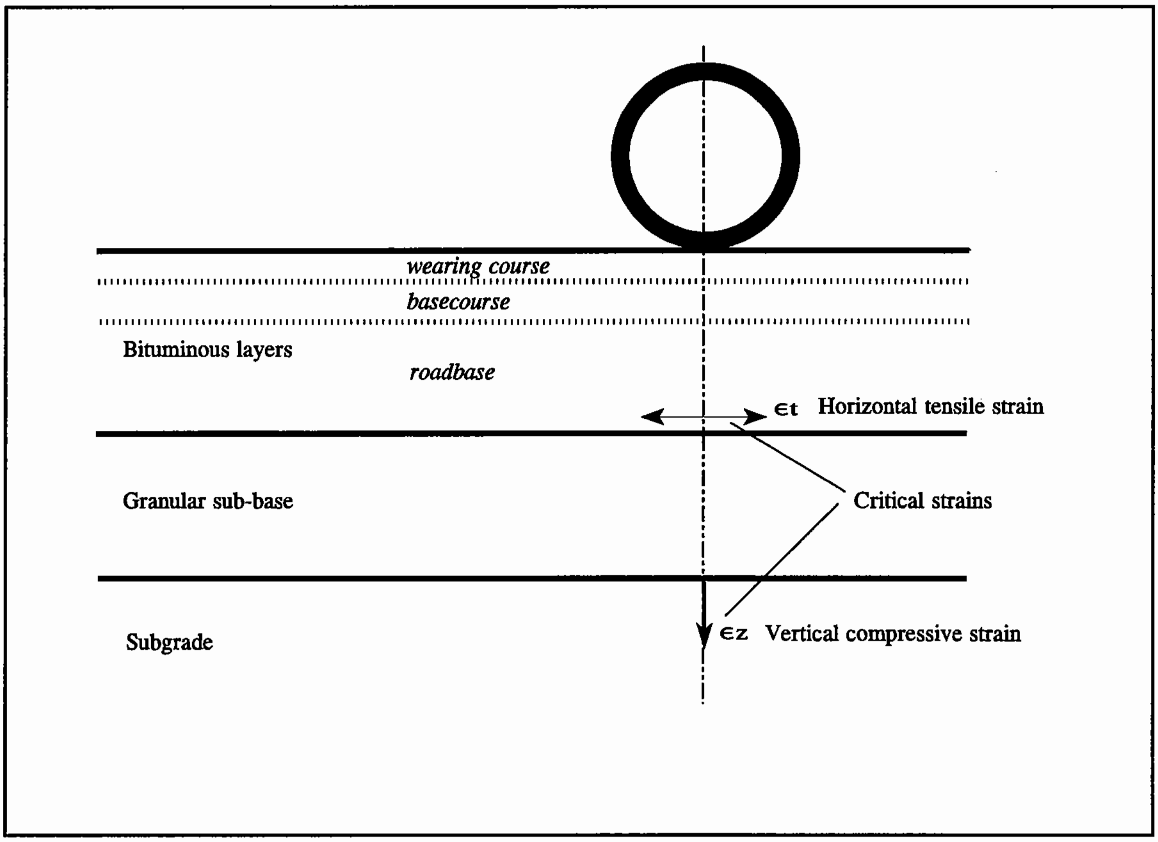

Figure 1.1 Typical Flexible Pavement Structure

There are two principal forms of structural distress which occur in flexible pavements as a result of vehicle loading ; cracking in the bituminous material and the formation of longitudinal depressions, or ruts, apparent at the surface in the paths repeatedly tracked by vehicle wheels. Cracking in bituminous mixtures has been shown to be a fatigue phenomenon dependent upon the tensile strain induced in the material (9). In this form of construction the maximum, and therefore critical, value of this strain generally occurs horizontally at the bottom of the roadbase under trafficking (10).

Rutting may be caused by deformation at depth in the pavement structure which is reflected in the profile at the surface or by permanent deformation which occurs internally within the bituminous material. The former mechanism is usually termed structural rutting since it represents a failure of the pavement to reduce the load on the formation to an acceptable level, while the latter is termed non-structural since it is confined to the bituminous layers. The development of rutting in flexible pavements, often a combination of the effects of both mechanisms, has been empirically correlated to the level of vertical compressive strain which is developed at the top of the subgrade under vehicle loading (11).

Analytical methods for the design of flexible pavements, such as the Shell method (7) and the Nottingham method (10), are based upon guarding against deterioration and/or failure through fatigue cracking or rutting by limiting the magnitudes of the critical strains, indicated on Figure 1.1, which are developed in the structure. Though the observed rutting from which the correlations with vertical subgrade strain were established would have included contributions from all layers (11), the use of this parameter as a criterion in thickness design serves principally to guard against structural rutting. Non-structural rutting is dependent upon the bituminous mixture's resistance to internal deformation rather than its thickness. This is reflected to an extent in the Nottingham design method where factors based upon the relative deformation resistance of different mixture types are applied to the allowable subgrade strain to account for the contribution to total rutting from the bound layers.

1.4 Permanent Deformation of Bituminous Mixtures

Permanent deformation which takes place within a bituminous mixture develops primarily through shear displacement (12,13), though there may also be a degree of densification under traffic, particularly in pavements which are not adequately compacted during construction. Such deformation is able to take place because of the complex stress-strain behaviour of the bitumen binder. In general terms, bitumen behaves as an elastic solid at low temperatures and high frequencies of loading, as a viscous fluid at high temperatures and low loading frequencies and exhibits visco-elastic behaviour in the intermediate range. In consequence, the response of a bituminous mixture to an applied load is in part viscous, the degree of viscous behaviour being dependent upon both temperature and the duration of loading. An element of the deformation which is induced under the application of a vehicle load is, therefore, irrecoverable and with repeated load applications this permanent deformation accumulates leading to the formation of ruts. Since the bitumen behaves visco-elastically under the greater part of service loading conditions and does not, therefore, rigidly bind the aggregate in the matrix, the permanent strain in the mixture may also include an element of irrecoverable, instantaneous plastic strain in the aggregate fraction of the material. Figures 1.2 and 1.3 show, respectively, an idealised strain response to an applied stress pulse and the accumulation of permanent strain under repeated loading.

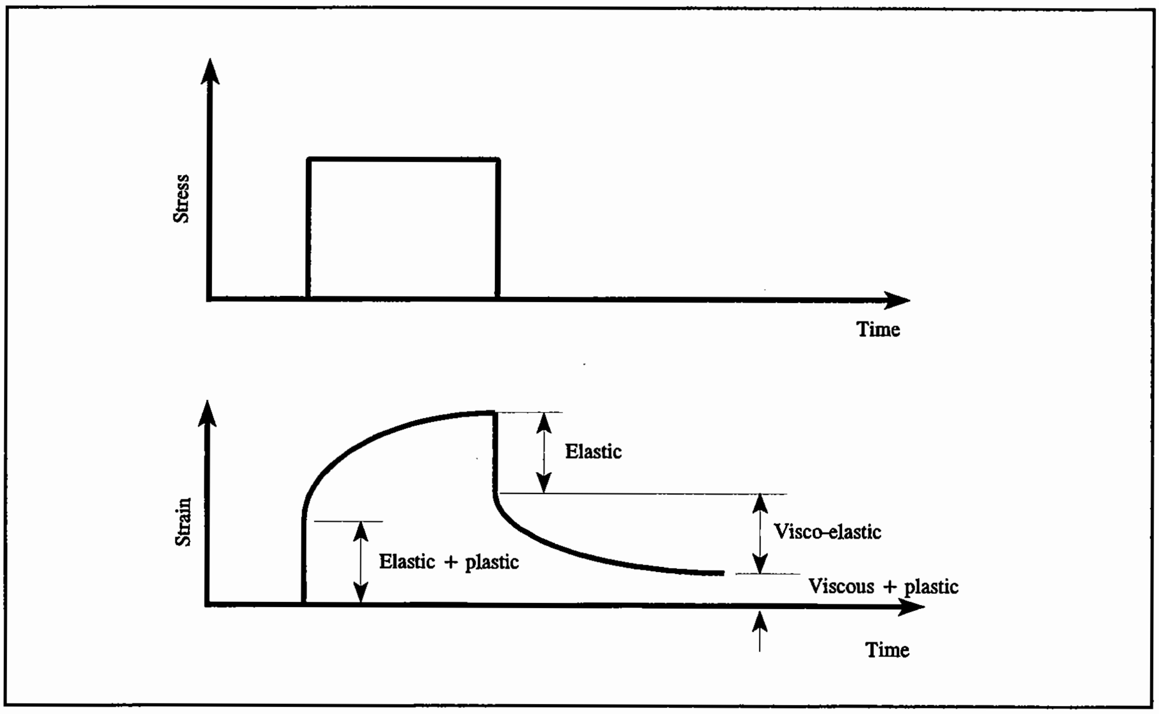

Figure 1.2 Idealised Strain Response of a Bituminous Mixture

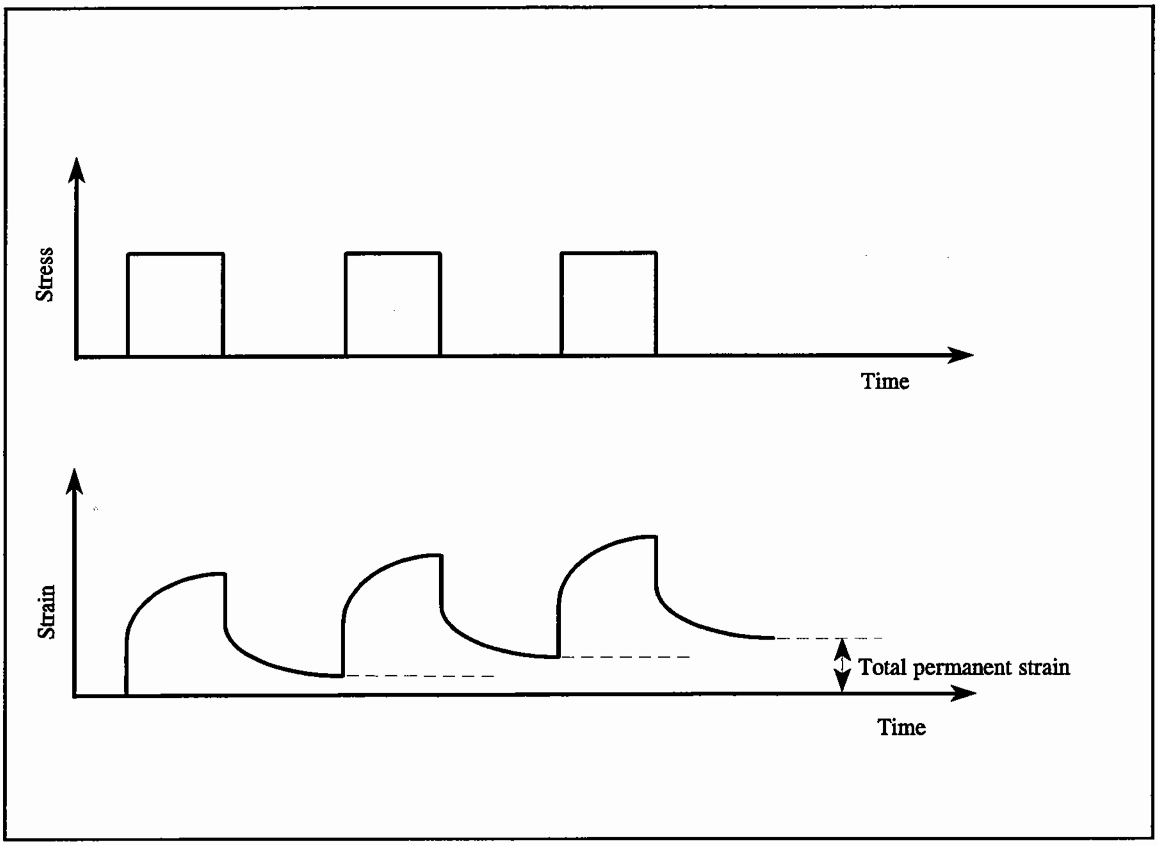

Figure 1.3 Accumulation of Permanent Strain under Repeated Loading

Rutting due to deformation within the bituminous material can generally be distinguished from structural rutting by the profile generated at the surface, which is characterised by the formation of "shoulders" at the edge of the ruts, as shown in Figure 1.4, due to lateral displacement of the material.

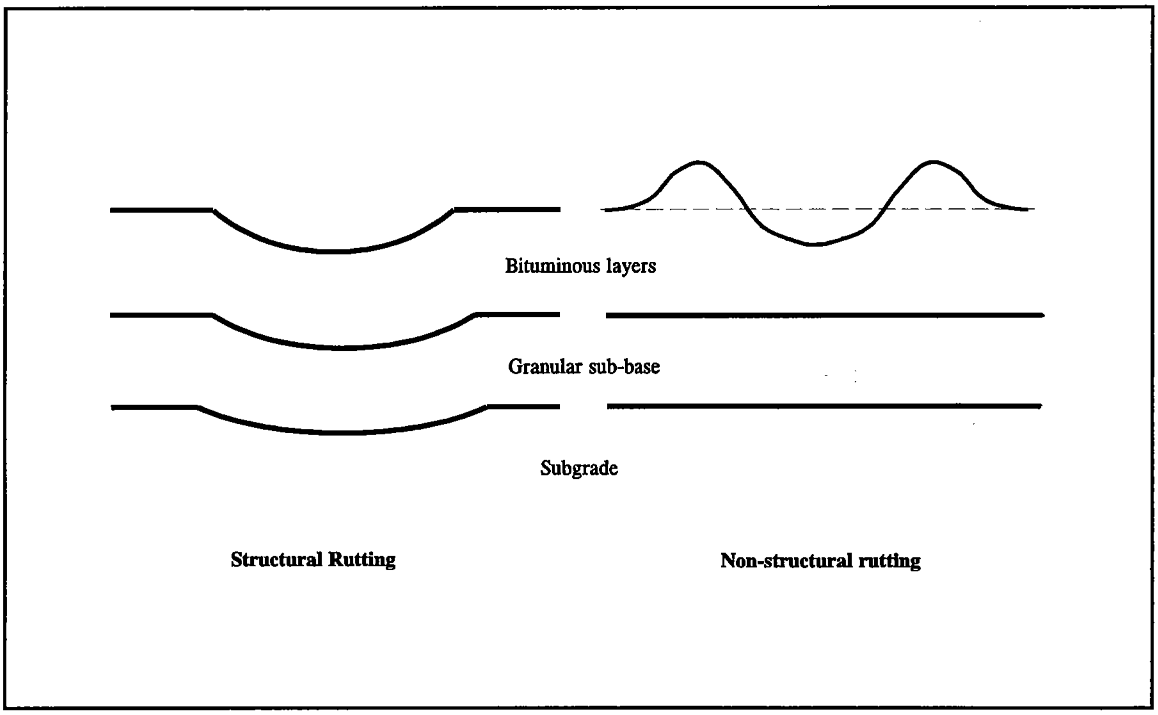

Figure 1.4 Structural and Non-Structural Rutting

1.5 Mechanical Properties of Bituminous Mixtures

The mechanical properties of bituminous mixtures are strongly dependent upon the properties of the binder and the volumetric proportions of the three mixture components ; aggregate, bitumen and air voids. Both mixture stiffness, which is a measure of load spreading ability analogous to Young's Modulus for linear elastic materials, and resistance to fatigue cracking may be reasonably estimated for the purposes of structural design from a knowledge of basic binder properties and volumetric composition (14,15). The required binder properties are determined using data from the standard tests for Penetration and Softening Point which provide pseudo-viscosity measures widely used for grading of bitumens (16,17,18). The basic parameters for characterisation of volumetric composition are the volume of binder, V_B , and the volume of air voids, V_V , and from these two other frequently used parameters may be determined:

Voids in Mineral Aggregate, $$VMA = V_B + V_V \tag{1.1}$$

Voids Filled with Bitumen, $$VFB = \frac{V_B}{VMA} \times 100\% \tag{1.2}$$

The reported values of these parameters, which are represented visually in the form of a component diagram in Figure 1.5, depend upon the method of determination of sample density, maximum theoretical density and binder content, from which they are calculated.

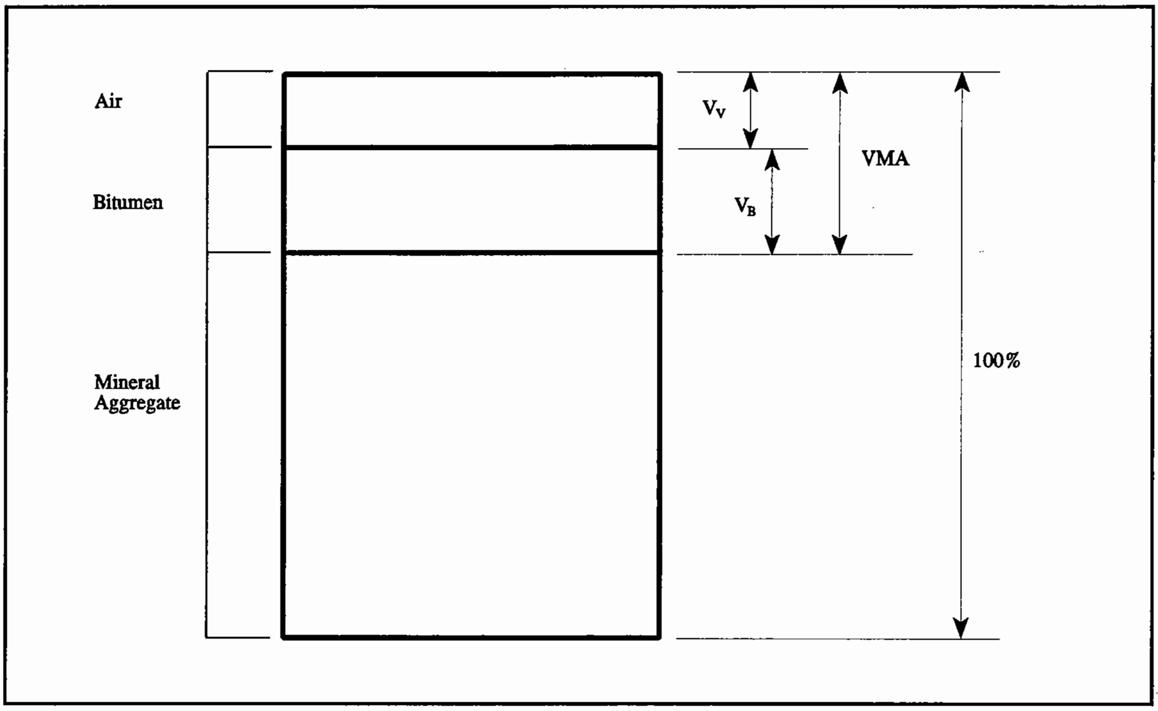

Figure 1.5 Representation of Volumetric Composition of a Compacted Bituminous Mixture

The resistance of a bituminous mixture to permanent deformation is also dependent upon binder properties and volumetric composition. Bitumen stiffness has a strong influence on this aspect of performance and it is under conditions of low binder stiffness that permanent deformation tends to occur, or at least accumulate at the highest rate. At low binder stiffness, associated with high temperature and/or long load duration due to the visco-elastic nature of the material, the aggregate fraction of the mixture has a more significant effect on response and, therefore, aggregate-related parameters such as grading, shape and texture also influence deformation resistance. Table 1.1, which is reproduced from a report prepared by Sousa et al (19), provides a useful summary of the factors which affect resistance to permanent deformation. The potential for interaction between the various mixture parameters is great and it is generally accepted that it is not possible to estimate deformation resistance from a knowledge of composition and that some form of mechanical test is required for evaluation of this characteristic (20).

Table 1.1 Factors Affecting Rutting Resistance of Bituminous Mixtures

(reproduced from reference 19)

	Factor	Change in Factor	Effect of Change in Factor on Rutting Resistance
Aggregate	Surface Texture	Smooth to rough	Increase
	Gradation	Gap to continuous	Increase
	Shape	Rounded to angular	Increase
	Size	Increase in maximum size	Increase
Binder	Stiffness[a]	Increase	Increase
Mixture	Binder content	Increase	Decrease
	Air void content[b]	Increase	Decrease
	VMA	Increase	Decrease[c]
	Method of compaction	-[d]	-[d]
Test / field conditions	Temperature	Increase	Decrease
	State of stress/strain	Increase in tyre contact pressure	Decrease
	Load repetitions	Increase	Decrease
	Water	Dry to wet	Decrease if mix is water sensitive

[a] Refers to stiffness at temperature at which rutting propensity is being determined. Modifiers may be utilized to increase stiffness at critical temperatures, thereby reducing rutting potential.

[b] When air void contents are less than about 3 percent, the rutting potential of mixes increases.

[c] It is argued that very low VMA's (e.g., less than 10 percent) should be avoided.

[d] The method of compaction, either laboratory or field, may influence the structure of the system and therefore the propensity for rutting.

1.6 Bituminous Mixtures Used in the UK and their Specification

There are two generic types of hot-mixed bituminous material which are used in the greater part of major road construction in the UK, these being dense bitumen macadams (DBM's) and rolled asphalts. DBM's are an example of continuously graded materials, so called because the distribution of aggregate particle sizes in the mixture is continuous. Figure 1.6 shows the aggregate grading curve for a typical DBM and Figure 1.7 shows an idealised section through a material of this type.

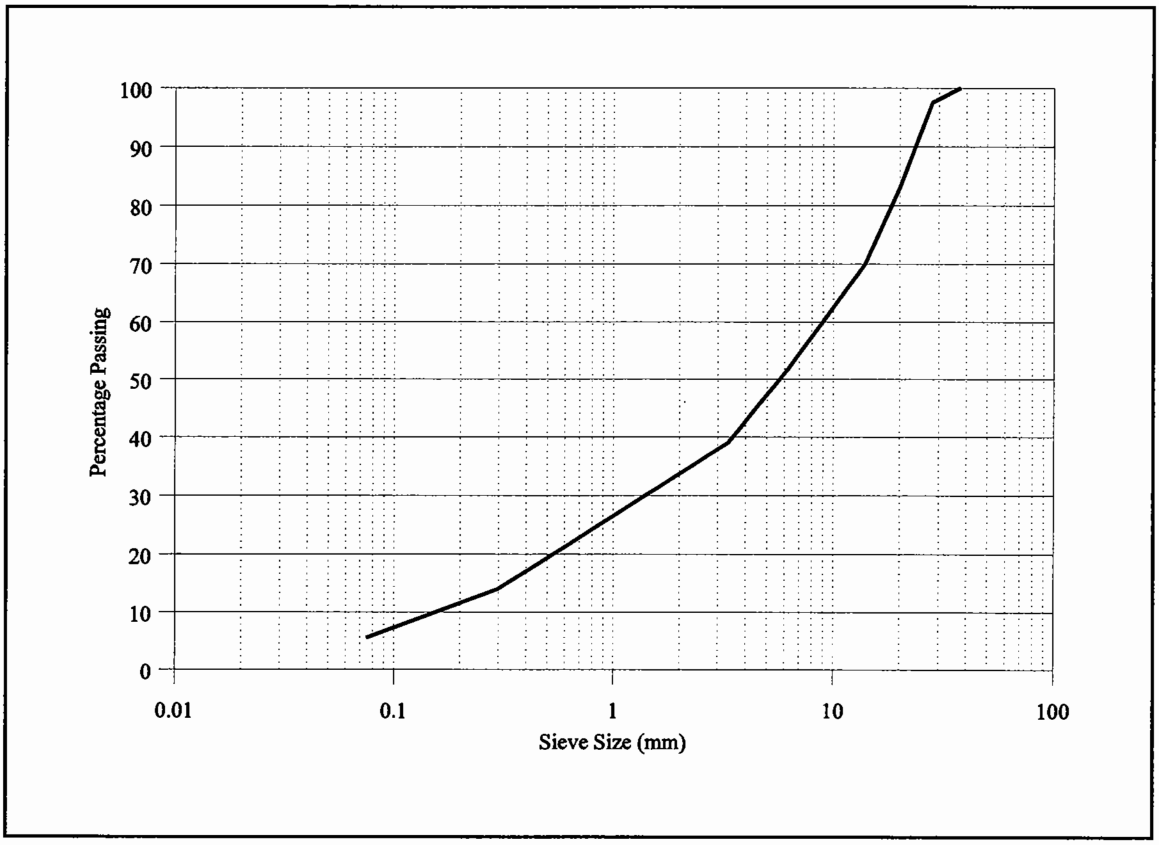

Figure 1.6 Aggregate Grading Curve : Typical 28mm DBM Roadbase

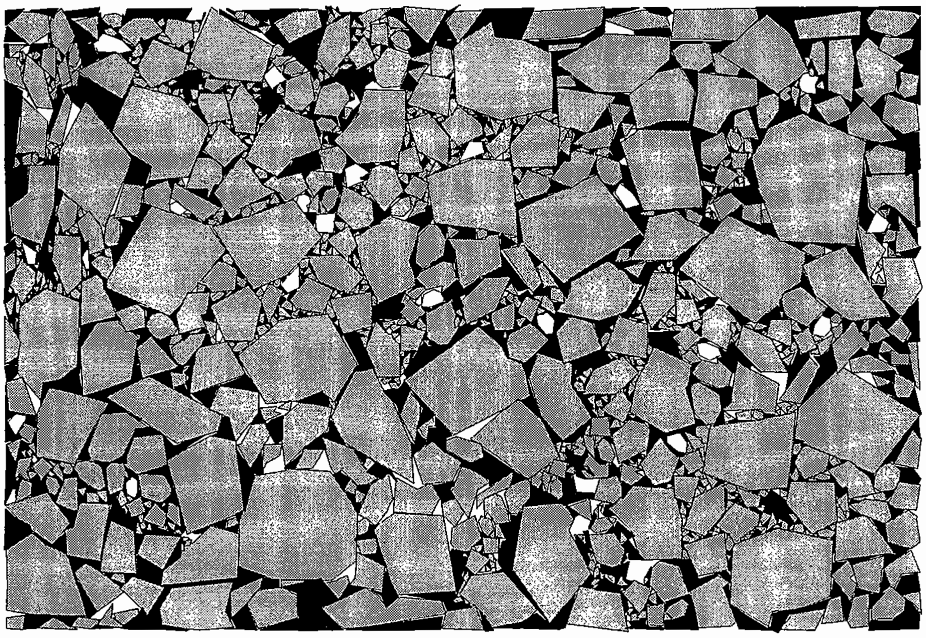

Figure 1.7 Idealised Section through a Continuously Graded Material

It can be seen from Figure 1.7 that there is stone to stone contact between the coarse aggregate particles, forming a continuous framework throughout the mixture. This framework, often referred to as the aggregate skeleton, provides the principal mechanism for the material to transmit load.

In contrast, rolled asphalts are gap graded, the term referring to the omission of a size or range of sizes in the aggregate particle distribution, which appears as a "gap" in the aggregate grading curve. This is illustrated in Figure 1.8, which shows a grading for a typical hot rolled asphalt wearing course. A schematic representation of a gap graded mixture is given in Figure 1.9, which shows the coarse aggregate particles distributed in a mortar composed of bitumen, fine aggregate and filler. As there is little or no contact between coarse aggregate particles the mixture transmits primarily load through the mortar which must, therefore, be stiff.

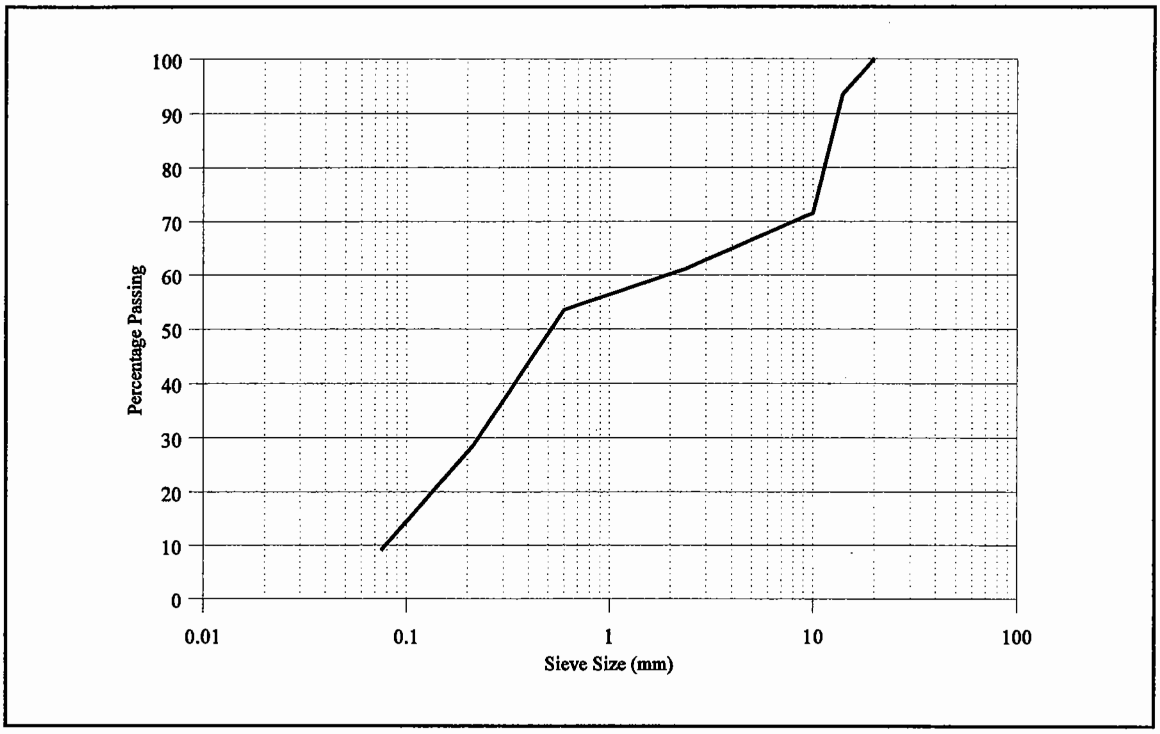

Figure 1.8 Aggregate Grading Curve : Typical 30/14 HRA Wearing Course

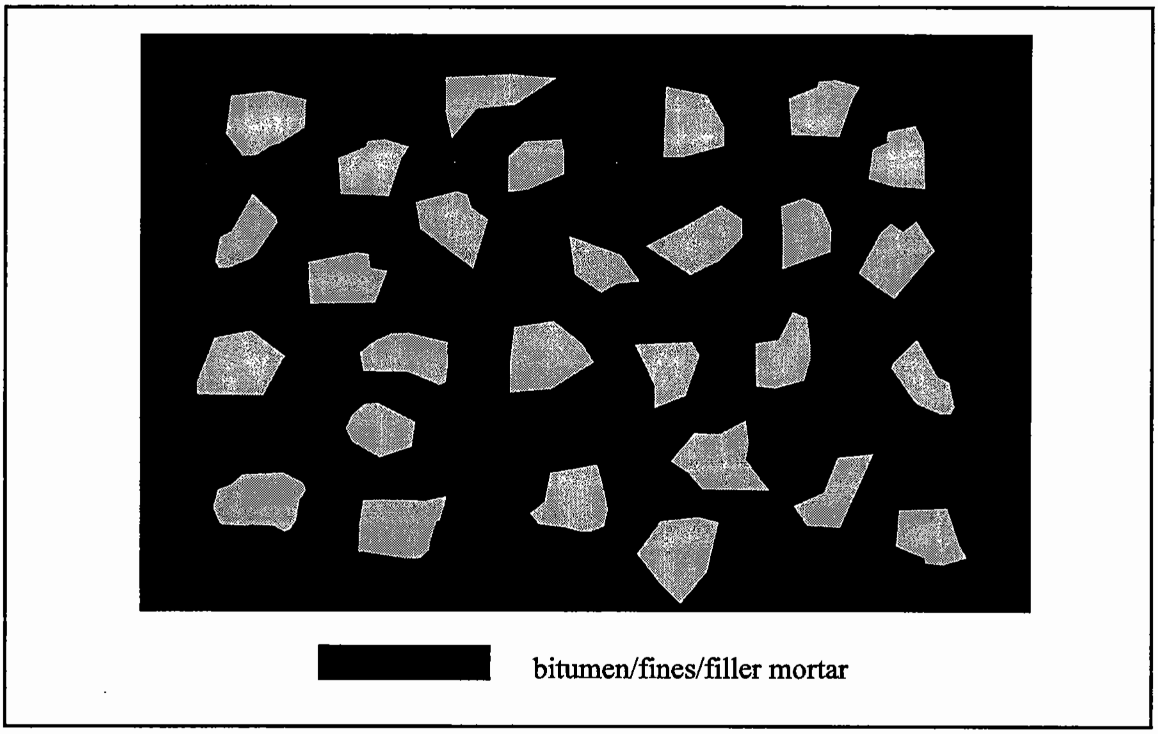

Figure 1.9 Idealised Section through a Gap Graded Material

This representation would generally apply to rolled asphalts used in the UK, which have coarse aggregate contents of up to 60 percent. There are, however, other types of gap graded mixtures, such as stone mastic asphalt (SMA) and porous asphalt which are widely used in Europe, which have a sufficiently high coarse aggregate contents (> 70%) to ensure that a continuous aggregate skeleton is formed.

Both DBM's and rolled asphalts are specified according to British Standards, BS 4987 (21) and BS 594 (22) respectively, which control the composition of the mixture in terms of aggregate grading and binder content, though the option of determining binder content using a modified version of the Marshall mixture design method (see Chapter Two) does exist for rolled asphalt wearing courses only. The development of both the materials and their specifications has been empirical (3,23), based on previous experience of mixtures which have performed satisfactorily rather than on implementation of rationally developed principles of mixture performance.

This type of specification, known as a recipe specification, may to an extent be used as a surrogate for proper volumetric proportioning but cannot take into account the differences in mechanical properties of materials made with aggregates from different sources, which are of prime importance when considering resistance to permanent deformation. Moreover, in a recent critical review of the specification and performance of DBM's, Haydon (23) has reported that problems of permanent deformation in roadbase and basecourse materials which complied with the BS 4987 (21) recipe specification have been largely attributable to deficiencies in volumetric proportioning. A highly publicised incidence of severe early deformation during construction of the A47 Norwich Southern Bypass in 1991 brought the issue to attention and, according to Haydon, subsequent investigations revealed the problem to be more widespread than had been realised since deformation usually occurs gradually and is rectified, without remark, as routine maintenance. As a consequence the Department of Transport (DOT) has introduced into its own specification (24) basic volumetric requirements with which DBM mixtures for use on motorways and trunk roads must comply, over and above the recipe requirements of BS 4987, in order to ensure a minimum level of deformation resistance.

1.7 The Costs of Roads

The highway network provides a service for its users which has costs as well as benefits. The nature and scale of those costs will depend on, among other things, the quality of the roads themselves. For example, it is known that in developing countries, due to the generally poor state of the roads, vehicle operating costs represent a high proportion of total transport costs (25). In Britain where the workforce is, as a whole, highly skilled and industry well developed, one of the main costs arises from loss of productivity due to delays in the movement of both goods and personnel.

Maintenance on the highway is a significant cause of delay and the introduction of the principle of whole life costing for pavements (26) has highlighted the long term economic benefits of forms of construction which cause the least disruption to traffic through closure for repair, due to the high value of peoples' time which is wasted by delay. The potential for reduction in whole life costs despite likely increase in construction costs was the principal reason for the recent draft revision to the DOT's specification for pavement thickness design (27) which introduced the option to increase the design life for flexible pavements from 20 years to 40 years before major strengthening would be required (28).

Bituminous materials are used in the majority of road construction and maintenance works in this country, and while permanent deformation in these materials does not necessarily constitute a structural failure it does, if severe, have a major effect on pavement serviceability and maintenance will be required to restore the profile. Clearly any improvements which could be effected to make this maintenance less frequent and less extensive would not only increase the engineering performance of the pavement but would yield economic benefits as well.

1.8 The Scope for Improvement

While prevention of structural rutting and fatigue cracking in flexible pavements may be addressed through thickness design, resistance to permanent deformation in the bituminous material can only be achieved through attention to the design of the mixture itself. Under the present system in the UK there is little scope or incentive for mixture design since a contractor must meet the composition prescribed by the recipe specification, and provided this is achieved he will receive payment with no further obligation. The use of this type of specification effectively precludes innovation and the development of new materials and, perhaps more importantly, it does not guarantee good performance, as illustrated by the failures cited by Haydon (23).

However, the DOT has signalled its intention to move towards a system of performance specification for bituminous materials (29) in which materials are specified by their mechanical properties without regard to the composition used to obtain those properties. This is in keeping with the current process of harmonisation of product standards within the European Community, under which it has been directed that construction products should, as far as practicable, be specified in terms of performance since national method or recipe specifications may constitute a technical barrier to trade (30). It is also consistent with the DOT's policy of transferring greater risk to the supplier (31).

This form of specification gives the producer much greater freedom and an incentive to develop cost effective materials, for which a potential market has been provided by the government's private finance initiative with the introduction of schemes where a concessionaire is responsible for the design, construction, operation and maintenance of road projects (31). In the development, and perhaps also marketing, of new or improved materials producers will need a tool to assess the performance of their products. For resistance to permanent deformation a mechanical test will be required, which to gain widespread acceptance and use must be effective but simple. Such a test, capable of measuring fundamental properties, will also be required for implementation of a system of performance specification.

1.9 The Aim of the Research

This research has been concerned with the potential use of mechanical testing for assessment of resistance to permanent deformation, primarily for use in mixture design but also with a view to application in performance specification. The work has been based largely on the evaluation of an unconfined uniaxial repeated load compression test using equipment developed by Cooper (32), and proposed for use in a method of mixture design developed at the University of Nottingham (33).

The early part of this research, described in Chapters Three and Four, which compared the performance of this test with others, was made possible through contract work for the US Strategic Highway Research Program (SHRP), (152). This research contract required well defined tasks to be performed, but provided the opportunity to carry out supplemental test programmes to augment the findings from the basic test programme.

The remainder of the research was carried out under the LINK Bitutest project (153). This project was funded by both government and industry, and had the aim of developing simple, implementable methods of test for the determination of mechanical properties of bituminous paving materials. The sponsors and participating organisations are shown in Table 1.2. The active participation of the industrial partners was an important element of the research, not least because it provided the opportunity to carry out site-based testing programmes.

Table 1.2 LINK Bitutest Project Sponsors and Participants

Project Leader	University of Nottingham
Public Sector (Provision of funding)	Engineering and Physical Sciences Research Council, Department of Transport
Industrial Partners (Provision of funding and technical support)	Cooper Research Technology Ltd.
	Esso Petroleum Company Ltd.
	Foster Yeoman Ltd.
	Mobil Oil Company Ltd.
	Nynas UK AB
	Shell Bitumen
	SWK Pavement Engineering Ltd.
	Tarmac Quarry Products Ltd.
	Wimpey Minerals Ltd.
Associate Partners - all County Councils (Provision of technical support)	Buckinghamshire
	Cambridgeshire
	Cheshire
	Hereford & Worcester
	Kent
	Lincolnshire
	Norfolk
	Northamptonshire
	North Yorkshire
	Shropshire
	Staffordshire
	Suffolk
	Warwickshire

CHAPTER TWO

PERMANENT DEFORMATION CONSIDERATIONS IN MIXTURE DESIGN

2.1 Introduction

The permanent deformation resistance of a bituminous mixture is dependent upon its composition. Performance is influenced by the properties of the components, such as the texture, grading, size and shape of the aggregate and the stiffness of the binder, as well as the relative proportions, by volume, of the mineral aggregate, bitumen and air voids. Rational methods for determining composition to optimise performance are widely used outside the UK. These methods of mixture design generally employ some form of mechanical testing to characterise or evaluate the properties of a mixture, particularly with respect to permanent deformation, and often include controls on certain aspects of composition intended to ensure an adequate level of performance.

The purpose of mixture design is to produce, economically, a mixture which has adequate stiffness, fatigue resistance, durability, workability and resistance to permanent deformation for its intended application. In order to illustrate the process of mixture design, and demonstrate the relative significance of resistance to permanent deformation in different approaches, a brief overview of selected procedures is given in Appendix A.

It is, however, resistance to permanent deformation which is of prime concern here and there are three elements of any mixture design procedure which are of significance in assessing or ensuring this characteristic. These are the control of composition, the method of mechanical testing and the method of preparation of the test specimen. The way in which each of these aspects is addressed in some of the methods of mixture design currently available or in use is summarised briefly in Table 2.1 below, and discussed in more detail in the following sections.

Table 2.1 Permanent Deformation Considerations in Methods for the Design of Bituminous Paving Mixtures

(Sources : references 33,42,43,44,45,47,52,60,70,81,94)

Country	Method / Specifier	Compositional Controls				Compaction Method *Specimen Configuration*	Mechanical Test	Remarks
		Binder Properties	Aggregate Characteristics	Aggregate Grading	Volumetric Composition			
USA	Marshall / Asphalt Institute (AI)	None specified by Asphalt Institute	None specified by Asphalt Institute	AI advises against gradings too close to max. density (Fuller, n = 0.45) due to potential low voids & instability	$V_V \nless 3.0\%$ Max. VFB (Value varies with traffic) Min. VMA (mainly for durability / fatigue)	Marshall impact hammer No. of blows varies with traffic *Cylindrical, 102mm dia. 64mm high Max. aggregate size ≯25mm.*	Marshall Stability & Flow @ 60°C	Fatigue / durability led methodology
USA	Hveem / Asphalt Institute	None specified by Asphalt Institute	None specified by Asphalt Institute	AI advises against gradings too close to max. density (Fuller, n = 0.45) due to potential low voids & instability	$V_V \nless 4.0\%$	California Kneading Compactor *Cylindrical, 102mm dia. 64mm high Max. aggregate size ≯25mm*	Hveem Stabilometer @ 60°C	Formulations showing bleeding or flushing after compaction are rejected

Table 2.1 (continued)

Country	Method / Specifier	Compositional Controls				Compaction Method *Specimen Configuration*	Mechanical Test	Remarks
		Binder Properties	Aggregate Characteristics	Aggregate Grading	Volumetric Composition			
USA	AAMAS / NCHRP (*Effectively superseded by SHRP Superpave*)	None specified	≮ 60% of agg. > 4.75mm to have ≮2 fractured faces. Natural sand content ≯ 20/25% (varies with traffic). 100% crushed agg for tyre pressures > 200psi.	Continuous, dense gradings based on Fuller eq^n (n = 0.45) with controls to prevent potential low voids & instability	V_V ≮ 3.0% @ refusal density obtained with gyratory compactor	Gyratory Compactor (optionally California Kneading Compactor) *Cylindrical 100mm dia. for max. agg. size ≯ 25mm 150mm dia. for max. agg. size ≯ 38mm*	Gyratory @ 60°C & Unconfined uniaxial compression, static load, @ 40°C	Gyratory used to assess variation in shear susceptibility with compaction
USA	Superpave/ SHRP	Performance grade selected according to climate & traffic	Requirements for coarse and fine aggregate angularity based upon design traffic and depth of material in pavement.	Continuous, dense gradings based on Fuller eq^n (n = 0.45) with controls to prevent potential low voids & instability	V_V ≮ 2% @ gyratory compactive effort which varies with traffic & climate	SHRP Gyratory Compactor *Cylindrical 150mm dia.*	Simple Shear Tester *(provides input data for predictive methodology)*	Level of design varies with traffic. Basic level is volumetric only.
UK	Nottingham University	None specified	None specified	Continuous Modified Fuller eq^n, n = 0.5, 0.6 & 0.7	Min. V_V dependent on compactibility, but ≮ 3.0%	PRD (vibrating hammer) *Cylindrical 150mm dia.*	Unconfined uniaxial repeated load @ 40°C	Deformation led methodology

Table 2.1 (continued)

Country	Method/ Specifier	Compositional Controls				Compaction Method *Specimen Configuration*	Mechanical Test	Remarks
		Binder Properties	Aggregate Characteristics	Aggregate Grading	Volumetric Composition			
South Africa	LAMBS / SABITA	40/50 pen recommended	$\nless$ 75% of aggregate to have $\nless$ 2 fractured faces. Minimise use of rounded natural sand.	Guidance for Continuous, Modified Fuller eq^n, n = 0.4 to 0.7 (advises n > 0.7 may lead to segregation)	$V_V \nless 3\%$ VFB $\ngtr$ 80% Min. VMA (mainly for durability / fatigue)	Hugo Hammer (rotating impact hammer) *Cylindrical* *150mm diameter* *Max. agg size 37.5 - 53mm*	Unconfined uniaxial repeated load @ 40°C	Deformation led methodology. Use of large aggregates for lower binder demand & crushing costs.
Finland	ASTO / VTT	Softening Point > 50°C (Use of modifiers)	Aggregate selection based on resistance to wear by studded tyres & weathering / adhesion properties	Continuous (AC) Fuller eq^n, n = 0.45 - 0.8 Gap (SMA) advice on ratios of size fractions	VFB 80 - 90% $V_V \nless 1\%$ Specific surface area of fine aggregate $\ngtr$ 5000 m^2kg^{-1} to prevent potential low voids & instability	Gyratory Compactor *Cylindrical* *100mm dia. for max. agg. size $\ngtr$ 20mm* *150mm dia. for max. agg. size $\ngtr$ 32mm*	Unconfined uniaxial static (for AC) & repeated load (for SMA)	Where proven materials are used only volumetric design is required
						Steel wheel / segment roller compactor *Slab* *700 × 500 × 60mm*	Wheeltracker	

Table 2.1 (continued)

Country	Method/ Specifier	Compositional Controls				Compaction Method *Specimen Configuration*	Mechanical Test	Remarks
		Binder Properties	Aggregate Characteristics	Aggregate Grading	Volumetric Composition			
Jordan	TRL *(review of use of Asphalt Institute Marshall method)*	None specified by TRL	None specified by TRL	Recipe for continuous grading coarse of max density curve	$V_V \not< 3\%$ @ refusal obtained with Marshall hammer	Marshall hammer *Cylindrical, 102mm dia. 64mm high*	Marshall Stability	Study criticised AI Marshall procedure for producing mixtures too rich in binder
France	LCPC	Not specified by LCPC	Not specified by LCPC	Procedure applicable in principle to all mixture types	Volumetric requirements vary according to application	Pneumatic tyred roller compactor *Slab 500 × 180 × 100mm*	Wheeltracker @ 50°C or 60°C *(Test only when rutting risk is high)*	Performance based procedure.
Australia	Austroads / ARRB	Not specified by ARRB	Not specified by ARRB	Procedure applicable in principle to all mixture types	$V_V \not< 3\%$ (@ 350 cycles of gyratory for heavy traffic)	Gyratory (Gyrpoac) *Cylindrical 100 or 150mm dia.*	Unconfined uniaxial repeated load @ 50°C	Level of design varies with traffic. Basic level is volumetric only. Method not yet fully developed.

Table 2.1 (continued)

Country	Method/ Specifier	Compositional Controls				Compaction Method *Specimen Configuration*	Mechanical Test	Remarks
		Binder Properties	Aggregate Characteristics	Aggregate Grading	Volumetric Composition			
Belgium	CRR	Penetration grade depends upon coarse aggregate content of mixture	Requirement for min. percentage of sand to be crushed dependent upon layer (w/c, b/c or r/b) & traffic	Ratio of coarse : fines from voidage analysis. Filler content to give req^d properties of mastic	$V_V \nless 3\%$ in w/c $V_V \nless 5\%$ in b/c $5\% \le V_V \le 7\%$ in r/b	Marshall *Cylindrical, 102mm dia. 64mm high*	Marshall Stability & Flow @ 60 °C	Based on detailed analysis of voidage in aggregate structure
Holland	CROW	No details	No details	No details	No details	Gyratory or vibrating hammer preferred. *Cylindrical*	Unconfined uniaxial repeated load	Performance based procedure being developed to replace Marshall

2.2 Compositional Controls

2.2.1 Aggregate Characteristics

The observation that *"...... mixes containing aggregate particles having a 'hard, glassy surface texture' tended to be unstable and deform plastically, whereas oil mixes made with aggregates of a 'rough, irregular surface texture' were less inclined to deform and, thus, more stable"*, (34) led Francis Hveem to the realisation that some form of mechanical testing would be required in addition to a compositional procedure for the method for designing "oil mixtures"[1] which he was developing in California in the late 1920s.

Hveem's observation is supported by a considerable body of evidence, based on laboratory research (35,36,37) and field studies (38,39), which shows that the use of aggregate which is angular in shape and/or coarse in texture is generally beneficial for the deformation resistance of a particular mixture formulation. This applies equally to the fine aggregate fraction in gap graded mixtures as to mixtures which rely on coarse aggregate friction and interlock (40,41).

The Belgian CRR (42) and South African LAMBS (43) methods of mixture design presented in Table 2.1 use, respectively, requirements and recommendations for the amount of crushed aggregate to be included in the mixture to promote resistance to deformation. Requirements for aggregate characteristics were also used in the US National Cooperative Highway Research Program (NCHRP) Asphalt-Aggregate Mixture Analysis System (AAMAS). The AAMAS (44) has now effectively been superseded by the SHRP Superpave mixture design system (45), in which a number of "consensus" properties are specified for mineral aggregates. The "consensus" properties - so called because they were established through consultation with a panel of acknowledged pavement experts (46) - of relevance for rutting resistance are coarse aggregate angularity and fine aggregate angularity, which are specified to provide adequate internal friction in the mixture. The required value of angularity for both coarse ($\geq$ 4.75mm) and fine ($\leq$ 2.36mm) fractions is determined according to the design

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[1]*According to Vallerga and Lovering (34) these oil mixtures were a combination of fairly good quality gravel and slow-curing liquid asphalts(bitumens)which could be either mixed on the road or in plant, and were spread by blade and compacted by traffic.*

traffic and the position of the layer in the pavement structure as shown in Tables 2.2 and 2.3 below.

Table 2.2 SHRP Superpave Criteria for Coarse Aggregate Angularity
(reproduced from reference 45)

Traffic (ESALs)	Depth From Surface	
	< 100 mm	> 100 mm
$< 3 \times 10^5$	55/-	-/-
$< 1 \times 10^6$	65/-	-/-
$< 3 \times 10^6$	75/-	50/-
$<1 \times 10^7$	85/80	60/-
$<3 \times 10^7$	95/90	80/75
$<1 \times 10^8$	100/100	95/90
$>1 \times 10^8$	100/100	100/100
Note: "85/80" denotes that 85 percent of the coarse aggregate has one fractured face and 80 percent has two fractured faces		

ESAL : Equivalent Standard (80 kN) Axle Load,

Table 2.3 SHRP Superpave Criteria for Fine Aggregate Angularity

(reproduced from reference 45)

Traffic (ESALs)	Depth From Surface	
	< 100 mm	> 100 mm
$< 3 \times 10^5$	-	-
$< 1 \times 10^6$	40	-
$< 3 \times 10^6$	40	40
$<3 \times 10^7$	45	40
$<1 \times 10^8$	45	45
$>1 \times 10^8$	45	45
Note: Criteria are presented as minimum percent air voids in loosely compacted fine aggregate		

The scope of the majority of the mixture design methods included in Table 2.1 does not extend to presenting requirements for aggregate characteristics, though such requirements may in practice be imposed by the ultimate specifying authority. For example, the authorative Asphalt Institute procedure for Marshall mixture design (47) contains no specific recommendations on aggregate properties, though it does say increasing the amount of crushed materials usually increases stability, but it is known that such properties have been specified by individual US states using the Marshall method (37).

2.2.2 Aggregate Grading

Specification or guidance on the design of aggregate gradings is generally limited to the continuous dense gradings of asphaltic concretes, which are materials similar to the DBM's used in the UK, derived from the Fuller equation (48) :

$$P = \left(\frac{d}{D} \right)^{n} \qquad (2.1)$$

where P = percentage of material passing a sieve of size d mm

D = maximum particle size (mm)

n = an exponent between 0 and 1

Such gradings are favoured because they produce a dense aggregate structure with a high degree of interparticle contact, thus providing an excellent aggregate skeleton for the transmission of load and resistance to shear deformation. Typically the maximum density grading, that with minimum VMA, is obtained with a value of the exponent n between 0.45 and 0.5, dependent upon the packing characteristics of the aggregate (49). However, while minimum VMA may be desirable for resistance to deformation in the dry aggregate due to the increased number of contact points, very dense gradings may produce mixtures which are susceptible to deformation at high levels of compaction. Cooper et al (49) have described the mechanism whereby at very low void contents the binder/filler mortar is forced between the coarse aggregate particles reducing the number of contact points and thus reducing resistance to permanent deformation.

To avoid this situation the Asphalt Institute mixture design manual MS-2 (47) recommends that gradings should be adjusted away from the maximum density curve to increase the VMA of the mixture. More positive steps were used in the AAMAS which, being primarily intended for the design of dense graded hot-mix based on the n = 0.45 curve on the assumption that this gives maximum density, introduced gradation controls which placed upper limits on the fine aggregate (< 4.75mm) content and controlled the filler (< 75μm) content in order to ensure adequate void space in the mixture.

The SHRP Superpave system also uses control points at selected sieve sizes, but has additionally introduced a restricted zone in the fine fraction. The purpose of the restricted zone, which forms a band either side of the n = 0.45 curve from the 2.36mm sieve to the 0.3mm sieve (for 25mm and 37.5mm nominal maximum size gradings the restricted zone extends to the 4.75mm sieve), is both to prevent the grading approaching that curve too closely in order to ensure adequate VMA, and also to prevent fine sands representing too high a proportion of the total sand fraction. In practice it may encourage the use of crushed sands, which is considered beneficial (50). The boundaries of the aggregate restricted zone for mixtures of different nominal maximum size are presented in Table 2.4 below. Nominal maximum size is defined as the sieve one size larger than the first to retain more than 10 percent of the aggregate.

Table 2.4 Superpave Boundaries of Aggregate Restricted Zone

(reproduced from reference 45)

Sieve Size Within Restricted Zone	Minimum and Maximum Boundaries of Sieve Size for Nominal Maximum Aggregate Size (Minimum / Maximum Percent Passing)				
	37.5 mm	25.0 mm	19.0 mm	12.5 mm	9.5 mm
4.75 mm	34.7 / 34.7	39.5 / 39.5	-	-	-
2.36 mm	23.3 / 27.3	26.8 / 30.8	34.6 / 34.6	39.1 / 39.1	47.2 / 47.2
1.18 mm	15.5 / 21.5	18.1 / 24.1	22.3 / 28.3	25.6 / 31.6	31.6 / 37.6
600 μm	11.7 / 15.7	13.6 / 17.6	16.7 / 20.7	19.1 / 23.1	23.5 / 27.5
300 μm	10.0 / 10.0	11.4 / 11.4	13.7 / 13.7	15.5 / 15.5	18.7 / 18.7

As an example[2], the gradation controls for a 19.0 mm nominal maximum size material are presented in Table 2.5 and the corresponding grading chart with control points and restricted zone is shown in Figure 2.1.

Table 2.5 Superpave Gradation Controls For 19.0 mm Nominal Maximum Size Mixture
(reproduced from reference 45)

Sieve Size	Control Point (Percent Passing)	
	Minimum	Maximum
75 µm	2	8
2.36 mm	23	49
12.5 mm	-	90
Nominal maximum (19.0 mm)	90	100
Maximum (25 mm)	100	-

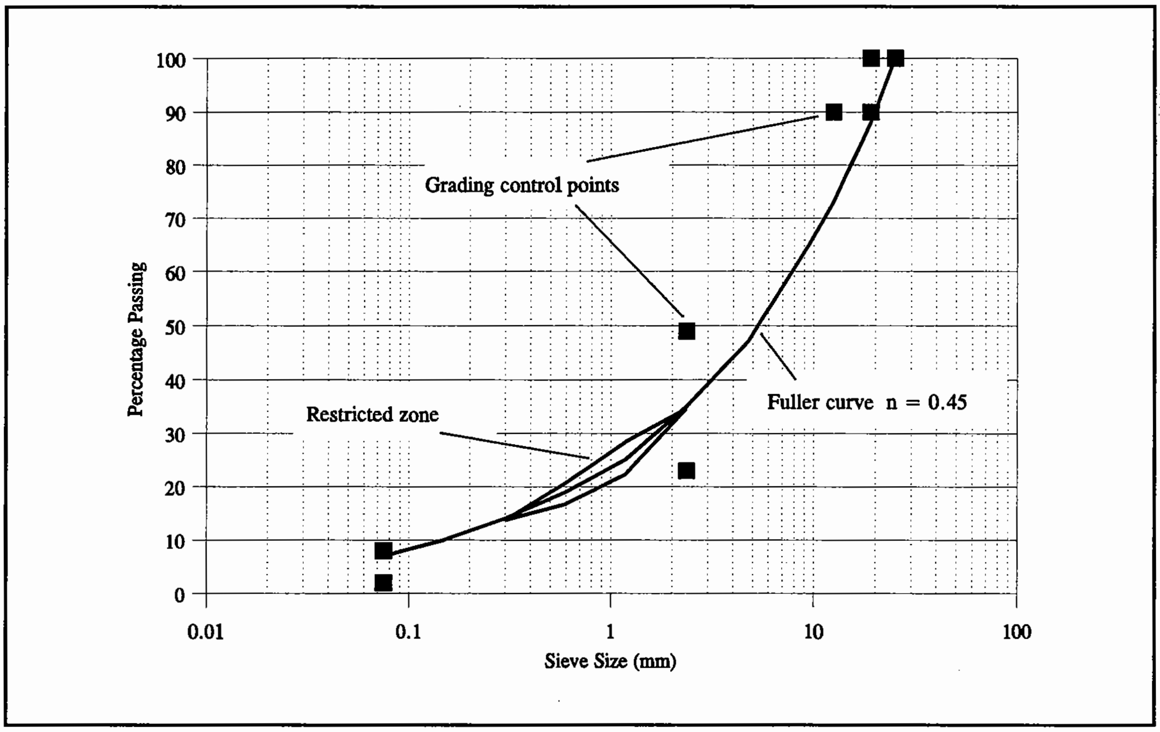


Figure 2.1 Illustration of Superpave Grading Control Measures

[2] *The existing Superpave restricted zones and gradation controls do not apply to other types of paving mixtures eg SMA or porous asphalt, and SHRP recommends that past experience and engineering judgement be used to develop specific gradation controls for such mixtures (45).*

An increase in VMA can be achieved through the use of gradings which are either fine or coarse of the maximum density curve. Due to their higher specific surface area, fine gradings will generally have a greater binder demand than coarse gradings for a given aggregate source. This can be detrimental for resistance to permanent deformation as an excess of binder/fines/filler mortar may result in a mixture which can readily be compacted to very low levels of residual air voids with resultant loss of contact between coarse aggregate particles. The use of a grading coarse of the n = 0.45 Fuller curve was recommended by the Transport Research Laboratory (TRL) in a review of mixture design in Jordan (51), for the purposes of obtaining improved permanent deformation resistance. The advantages of a coarse gradation were cited as provision of adequate VMA, good internal friction, low binder demand, good workability and tolerance of variations in binder content.

The approach adopted in the Nottingham method (33), based on earlier research by Cooper et al (49) aimed at maximising resistance to permanent deformation, is to increase the VMA of the mixture by using values of the exponent n greater than 0.45 to generate gradings coarser than the maximum density curve. A modified version of the Fuller equation which allows the filler content of the mixture to be fixed independently is used, which gives the designer greater control and avoids the generation of unrealistic filler contents.

$$P = \frac{(100 - F)(d^n - 0.075^n)}{(D^n - 0.075^n)} + F \tag{2.2}$$

where P = percentage passing a sieve of size d mm

D = maximum aggregate size

F = filler content (passing 0.075mm sieve)

n = an exponent between 0 and 1

The values of n used to generate target gradings are 0.5, 0.6 and 0.7.

The South African Bitumen and Tar Association (SABITA) have incorporated this method into their procedure for the design of large aggregate mixtures for bases (LAMBS), though they note that use a value of n of 0.7 may lead to mixtures which are prone to segregation.

The VTT / ASTO method developed in Finland (52) recommends the use of n values from 0.45 to 0.8 in the standard Fuller equation for the design of asphaltic concrete gradings. This method also provides guidance for the design of gradings for stone mastic asphalt, which is a high stone content gap graded mixture, in the form of recommended proportion ratios of individual size fractions.

2.2.3 Binder Properties

Determination of an appropriate binder content for the designed or selected aggregate grading is the prime function of the majority of mixture design methods. In terms of binder properties, however, while the value of using a high viscosity binder for resistance to permanent deformation is generally recognised, few methods make specific recommendations. South Africa and Finland use pseudo-viscosity measures, the former recommending the use of a 40/50 penetration grade while the latter requires a Ring and Ball Softening Point of over 50°C from the point of view of resistance to deformation.

The most comprehensive specification of binder properties is in the SHRP Superpave system. One of the major products of the SHRP research was a suite of tests which are used for classification of binders by their performance characteristics, and this has enabled the introduction of an asphalt specification which categorises binders by performance grade (45). The required grade of binder is determined according to the climate in which it is to be used, and this may be assessed by geographical location or from pavement or air temperature data. The grading of the binder is based on properties which provide resistance to fatigue and low temperature cracking as well as permanent deformation. The selection of the appropriate grade is made on the assumption that traffic speeds will be high (90 km/hr). However, in what appears to be a principally deformation based consideration, design for slow moving traffic is accommodated by selecting a binder from a higher performance grade.

2.2.4 Volumetric Composition

Recognition that sound basic volumetric composition is essential if the desired mechanical properties of a bituminous paving mixture are to be obtained is evinced by the fact that the majority of the design procedures incorporate volumetric controls or recommendations. Though many of the methods make use of the same volumetric parameters (see Chapter One), the potential for variation in the methods used for determination of sample density and theoretical maximum density may give rise to differences in the reported values of those parameters.

The volume measurement for calculation of density may be obtained by mensuration but is more usually found from displacement in water, in which case it will be affected by the extent to which the water is able to permeate the voids in the mixture. This depends on whether the specimen is sealed during immersion and, if it is not, on the permeability of the specimen. Theoretical maximum density may be calculated from the specific gravities of each of the components and their proportions in the mixture. Determination of the specific gravity for the aggregate fractions is a lengthy process which requires great care in the test procedures, but the main shortcoming of this method is that in the mixed material some of the binder may be absorbed into the aggregate which results in an error in the estimation of its volume. This is usually compensated by making an allowance for the water absorption of the aggregate, on the basis that its absorption of bitumen will be similar. Alternatively, the theoretical maximum density of the mixed material may be determined using the Rice method (53), in which air is evacuated from a loose mixture sample under water. This method is generally preferred as it is a direct determination on the mixture in which any short-term absorption will have taken place. Whether or not binder absorption is accounted for affects the value of V_V, and hence both VMA and VFB. The implications of this have been investigated (54), but it is important to recognise that values of volumetric parameters determined by different methods may not be directly comparable.

Volumetric analysis is the key element of the Belgian CRR method, while the Nottingham and SABITA methods use volumetric criteria to screen out unsuitable formulations prior to mechanical testing. Both the SHRP Superpave and proposed Australian ARRB methods, which employ different levels of design according to the anticipated traffic, use volumetric analysis as the first level of design and permit the design of mixtures for lightly trafficked roads by this means alone. A similar approach is adopted in the Finnish ASTO/VTT method in which materials with a history of use are subject to only a volumetric check.

With the exception of the Hveem method, which is based upon estimating the surface area of the aggregate in the mixture in order that the quantity of binder required for coating can be determined, the approach used in the procedures presented in Table 2.1 is to assess or control the volume of voids in the mineral aggregate and the degree to which they are filled with binder, or binder/filler mortar. As the majority of the mixture design methods deal primarily with continuously graded materials the majority of volumetric criteria have been developed for application to this type of mixture. However, the common principle for ensuring resistance to permanent deformation which is adopted in these methods is to prevent overfilling of the voids to avoid loss of shear resistance by the mechanism described in Section 2.2.2 above. The Asphalt Institute Manual MS-2 (47), for example, defines one of the objectives of mixture design as being to:

> *"determine an economical blend and gradation of aggregates and asphalt* (bitumen) *that yields a mix having : .... Sufficient voids in the total compacted mix to allow for a slight amount of additional compaction under traffic loading without flushing, bleeding, and loss of stability...."*

In its version of the Marshall design procedure the Asphalt Institute imposes a minimum value of VMA dependent upon the nominal maximum aggregate size. The VMA is a measure of the void space in the aggregate structure, however it gives no indication of the degree of filling of the voids since it is simply the total of the binder volume, V_B, and air void volume, V_V. Moreover, minimum VMA is specified by the Asphalt Institute since it *"allows enough asphalt to be used to obtain maximum durability without the mixture flushing"*, rather than primarily as a means of ensuring resistance to deformation.

The parameter which measures the degree of filling of voids is VFB, voids filled with bitumen. Several methods make use of VFB criteria, with maximum values being of benefit for permanent deformation, while minimum values are generally for fatigue and/or durability. The Superpave VFB criteria are shown in Table 2.6 below.

Table 2.6 Superpave Criteria for Voids Filled with Bitumen

(reproduced from reference 45)

Traffic Level (ESALs)	Design VFB (percent)
$< 3 \times 10^5$	70 - 80
$< 3 \times 10^6$	65 - 78
$< 1 \times 10^8$	65 - 75
$> 1 \times 10^8$	65 - 75

The Finns, however, in recommending a range for VFB of 80 - 90% propose that values of VFB lower than 70% would be detrimental for resistance to deformation. In contrast, Haydon has suggested 70% as a maximum value of VFB for resistance to deformation in DBM's (23).

As part of its study in Jordan, TRL carried out an extensive analysis of cores to determine the effects of mixture parameters on performance. This showed that deformation had occurred in mixtures with a value of VFB in excess of 82 - 85%. However, in selecting a parameter for use in mixture design the TRL chose residual air voids, V_V, in favour of VFB (maximum proposed value of 75%) since accurate determination of VFB requires that the proportion of the binder which is absorbed into the aggregate is known.

Most of the methods presented in Table 2.1 require or recommend a minimum air void content, with 3% being the most commonly specified value. Several field studies have shown rutting in asphaltic concretes to be associated with void contents below this value (38,51,55,56,57,58,59).

The basis of the design philosophy in the Belgian CRR method is determination of the amount of binder/filler mastic required to fill the voids in the aggregate structure sufficiently for fatigue and durability without overfilling which would prejudice resistance to permanent deformation. This method was originally developed for use with dense, continuous "sand skeleton" (ie fine) mixtures, though recently an analytical procedure has been introduced (60) for determining the voidage in any aggregate structure based upon packing of the different size fractions and the angularity of the aggregate, which makes it applicable to any mixture type. An air void content of 3% is recommended for wearing courses, where impermeability and durability are of concern, with values of 5% and 5 - 7% being advised for basecourse and roadbase respectively.

A similar approach to the design of fine dense bituminous mixtures, HRA in particular, based on filling the voidage in the aggregate structure, which is determined with consideration to aggregate packing characteristics, has been developed at Heriot-Watt University (61). This method also recommends a minimum residual air void content of 3% for resistance to deformation.

The acceptable range of air voids in the Nottingham method varies according to the compactibility of the mixture, which is assessed by the air void content at refusal density determined using the Percentage Refusal Density (PRD) test (62), but the minimum permissible value is 3%. The minimum void requirement in the Superpave system is slightly lower at 2%, under the maximum effort applied with the SHRP gyratory compactor. Lower still is the minimum value of 1% specified for DBM mixtures at refusal density, established using the PRD test, in the amendment to the Specification for Highway Works (24), which was introduced following the investigation of the failure on the A47 referred to in Chapter One.

2.3 Specimen Compaction

There are three aspects of the method of compaction which are of significance. These are the structure developed in the material and its influence on measured resistance to deformation, the limitations on the maximum size of aggregate due to the dimensions of the compaction mould, and the level of density which is achieved.

2.3.1 Effect of Compaction Technique on Measured Resistance to Permanent Deformation

If mechanical testing in mixture design is to be of any value then it is important that the specimens which are tested are representative of the material in its compacted state in situ. There are several methods of specimen compaction employed in the design procedures listed in Table 2.1, which range in sophistication from simple impact techniques such as the Marshall hammer, which is described in Section 2.3.3, to gyratory compactors designed to emulate the kneading action of site rollers. Figure 2.2 illustrates the principle of operation of a gyratory compactor, in which the specimen is confined in a circular mould and subjected to a vertical load applied through parallel end platens while the mould is rotated at a constant angle of inclination to the vertical.

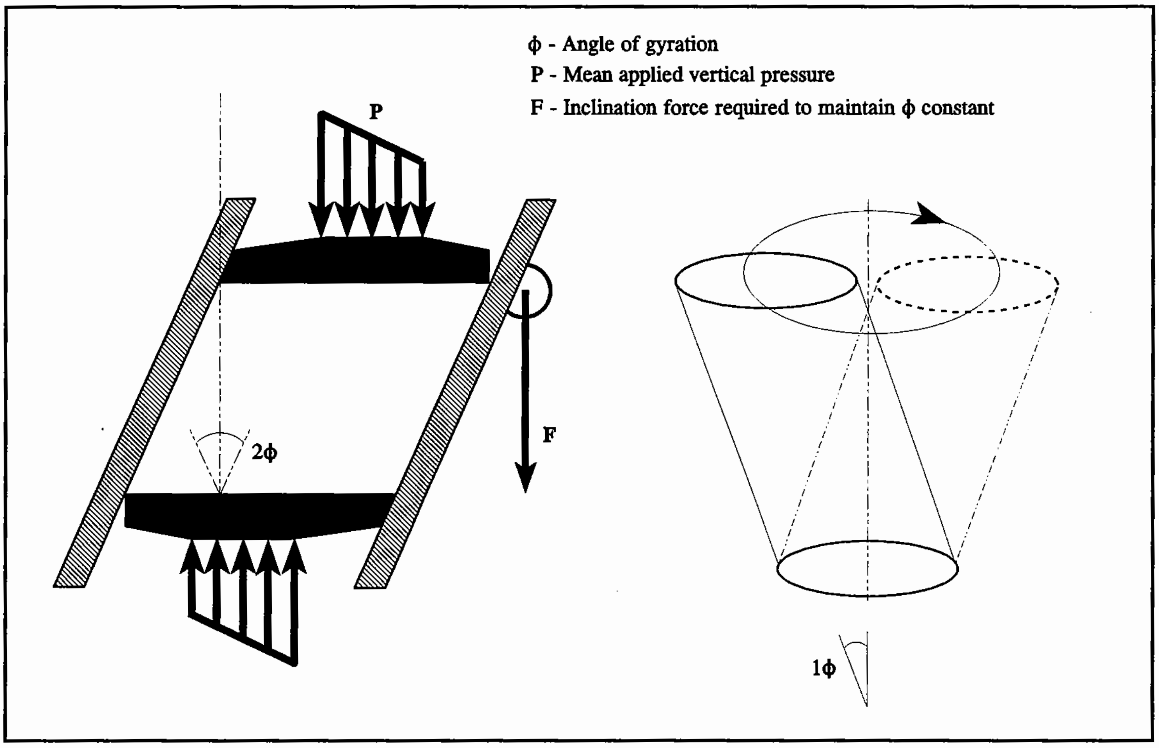


Figure 2.2 Principles of Gyratory Compaction

Several mixture design procedures use more directly simulative rolling techniques, while the kneading compactor used in the Hveem method (47) was developed to obtain better replication of site compaction than had been achieved using impact methods (34). The PRD apparatus utilised in the Nottingham mixture design method comprises a hand held vibrating hammer fitted with a circular tamping foot which is used is used to compact specimens within a split cylindrical mould. The PRD test (62) is used to assess the refusal density of mixtures because it is capable of achieving very high levels of compaction but the equipment, shown in Figure 2.3, was adopted as a means of specimen preparation largely because of its low cost and simplicity.

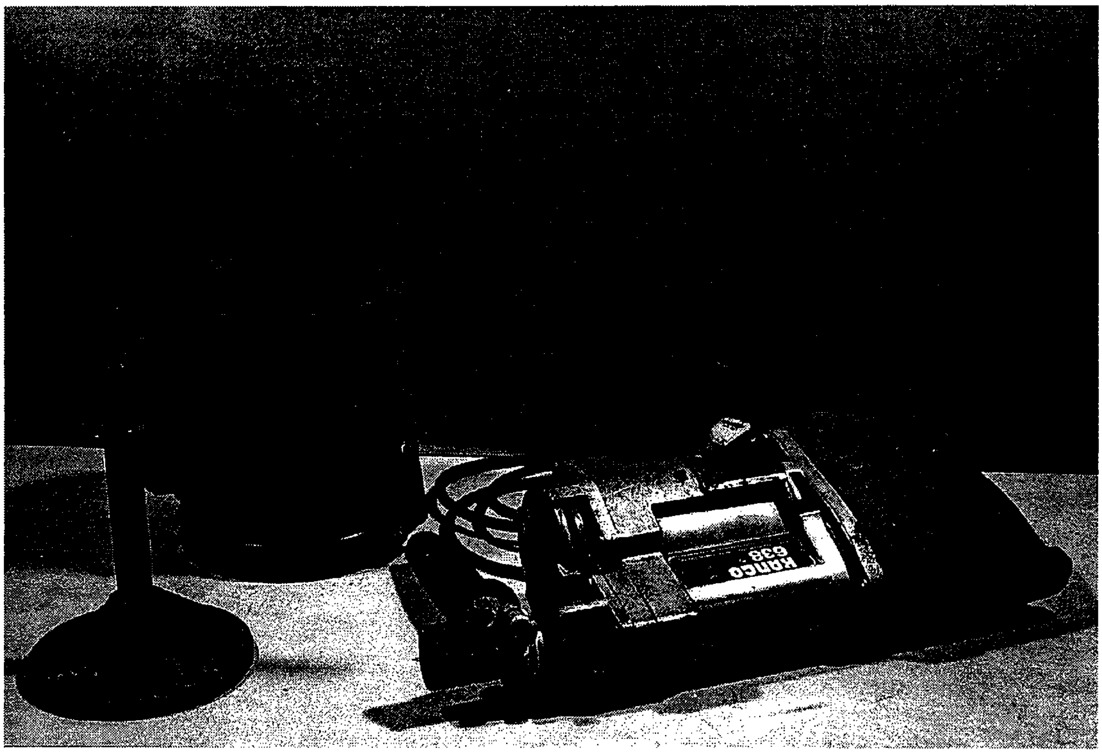


Figure 2.3 PRD Apparatus

It has been shown that different compaction processes can produce different preferred orientations in the aggregate (63,64,65) which strongly influence resistance to permanent deformation. Several researchers have reported differences in results obtained in various forms of permanent deformation test according to the method of compaction used (66,67,68,69,70). However, this work has been largely restricted to methods of compaction available in research laboratories and though some work has been undertaken by TRL (71), and by SHRP researchers (72), there is a shortage of data which permits a direct comparison between the properties of exactly the same material at the same void content subjected to both laboratory and site compaction techniques. This has, therefore, been investigated as part of this research and is discussed in some detail in Chapter Seven.

2.3.2 Limitation of Aggregate Size

Limitation of maximum aggregate size is a concern which has been expressed particularly in relation to the use of Marshall and Hveem where the 4 inch (102mm) diameter moulds have constrained the maximum size of aggregate to 1 inch (25mm). Acott (73) has observed that this has probably led to a situation in the USA where mixtures are designed to fit the mould rather than the pavement requirements. Certainly it has restricted the use of large aggregate mixtures which are perceived as offering improved rutting resistance (74), though some states, notably Pennsylvania and Kentucky, have attempted to overcome the problem simply by increasing the size of the Marshall apparatus to accommodate a 6 inch diameter specimen (75,76).

One of the reasons for SHRP's choice of the gyratory compactor for manufacture of specimens for the Superpave system was the facility to accommodate aggregate up to 50mm maximum (37.5mm nominal) size through the use of 150mm diameter moulds (50).

2.3.3 Compacted Density of Test Specimens

In Section 2.2.4 on volumetric composition, the importance for permanent deformation of providing sufficient void space in the mixture to prevent overfilling was discussed. It should be recognised that this must be achieved through attention to the structure of the material at high density and does not equate to reducing the level of compaction. Studies by the TRL on the performance of DBM's have shown that resistance to permanent deformation increases with improved compaction, provided the overfilled state is not approached too closely (77,78,79). Furthermore, it is likely that material which is not well compacted during construction will suffer subsequent densification under the action of traffic. It is, therefore, recognised that for designs performed in the laboratory to be valid, the density of the laboratory compacted material must reflect that which is ultimately achieved on site (80), particularly if volumetric criteria are to be of any value.

The Marshall mixture design procedure is widely used, in various forms, throughout the world, its popularity being due largely to the simplicity of the equipment and procedures. Marshall compaction conforming to the Asphalt Institute method (47) consists of applying a number of blows from an impact hammer having a flat circular tamping face of 3⅞ in. (98.4mm) diameter and a mass of 10 lb (4.5kg) from a drop height of 18 in. (457mm), to each face of a specimen confined in a mould of 4 in. (102mm) diameter at a temperature which would produce a kinematic viscosity of 280 ± 30 centistokes (approximately equivalent to a dynamic viscosity of 2.8 Poise) in the bitumen. The number of blows applied to each face is 35, 50 or 75 dependent upon the traffic category (light, medium or heavy), and the minimum required air voids at the appropriate level of compaction is 3%. However, it has been found that rutting in Marshall designed pavements is frequently associated with low air void contents (ie less than 3%, as discussed in Section 2.2.4 above) as a result of the in situ density being higher than the value which was used in the design (38,51,55,56,57). It would appear, therefore, that the standard Marshall compaction procedure does not match the density which may be attained on site, whether through compaction during construction or subsequent densification under traffic. This was recognised in the TRL's mixture design study in Jordan and the recommendation was made that the air void content of the designed mixture be checked to be at least 3% at a "refusal" density obtained by continuing the Marshall compaction process until no further densification occurred. In practice this may take several hundred blows.

The use of the PRD test to check air voids at refusal density in the UK DOT specification for DBM's has already been mentioned, and the AAMAS procedure's air void requirement was at refusal density obtained under gyratory compaction. A similar approach is adopted in the Superpave and Australian Road Research Board (ARRB) methods, which both use gyratory compaction, in that there is a requirement for a minimum void content at high compactive effort.

In the Superpave system the density at which binder content is determined corresponds to an air void content of 4%, and this is achieved in the laboratory design through applying a number of gyrations of the SHRP gyratory compactor. The number of gyrations is specified according to design traffic and climate. There is a further requirement for the mixture to maintain in excess of 2% air voids at a maximum compactive effort, which is also specified in terms of number of gyrations, dependent upon traffic and climate.

Under the ARRB procedure (81) specimens for volumetric analysis are compacted with 50, 80 or 120 cycles of the gyratory depending on whether the design traffic is classed as light, medium or heavy. There is an additional requirement for heavily trafficked mixtures that the air void content shall not be less than 3% after 350 cycles in the gyratory compactor.

In the Nottingham method the PRD equipment is used as a means of test specimen compaction, and degree of compaction is one of the design variables. Target specimen densities for each mixture formulation are given in terms of the percentage of refusal density, at values of 100%, 96% and 93%, this approach being chosen to accommodate the differences in compactibility of different aggregates. Appropriate volumetric criteria, including minimum voids, are selected according to the air void content - between 1% and 3% - at 100 PRD of the mixture with a grading exponent, n, of 0.5 and a binder content of 3.5% by mass of mixture. The use of the 100 PRD, or refusal density, compaction level is intended to ensure adequate properties, resistance to permanent deformation in particular, at very high density.

Haydon (23) has stated that the minimum void content value of 1% used in the DOT specification (24), while appearing low, is adequate because the refusal density at which it is determined, obtained by the PRD test, *"represents a high level of compaction which is unlikely to be reached under design traffic levels"*. A comparison reported by Shuler and Huber (82) which showed densities achieved by vibrating hammers, of similar type to that used in the PRD test, to be higher than those from standard Marshall, Hveem (kneading) and gyratory procedures would appear to confirm this view. It was concluded that the vibrating hammer compaction *"should be capable of producing mixtures with higher density than could be obtained after trafficking"*, which would be of value in assessing potential for mixtures to reach low void content in service, but that in designing at this "refusal" density air void criteria would have to be reduced below the usual 3 - 5% or a lean, unworkable mixture would result. However, the TRL, during the mixture design review in Jordan, found that 500 blows from the Marshall hammer gave higher densities than the PRD apparatus. A recent study in Holland (83) demonstrated that the Dutch Marshall hammer, German impact hammer, gyratory and the PRD equipment all achieved the same final density provided the process continued long enough, and concluded with the recommendation that volumetric requirements be based upon this "final" value.

It would appear, therefore, that while the importance of representative density is recognised, an appropriate value is difficult to quantify and there may in practice be significant differences between the various approaches which have been adopted in the methods assessed above.

2.4 Mechanical Testing

While volumetric criteria may be used to ensure that a particular mixture formulation is adequately structured for resistance to deformation, they cannot give an indication of the likely level of performance because, as discussed in Chapter One, of the number of other factors which affect rutting resistance and their potential interaction. For this purpose mechanical testing is required.

Hveem's method of mixture design was initially based on establishing the binder content necessary to provide a coating of adequate film thickness for a given aggregate and grading. However, it was because simple determination of this optimum binder content gave no assurance that the resulting mixture would resist deformation under traffic that a test to characterise the "stability" of the mixture was developed. The result was the Hveem Stabilometer, in essence a simple triaxial cell, which provides a measure of volumetric strain under an axially applied load. The evolution of the equipment, which is shown in Figure 2.4, and test procedures are well documented (34,47).

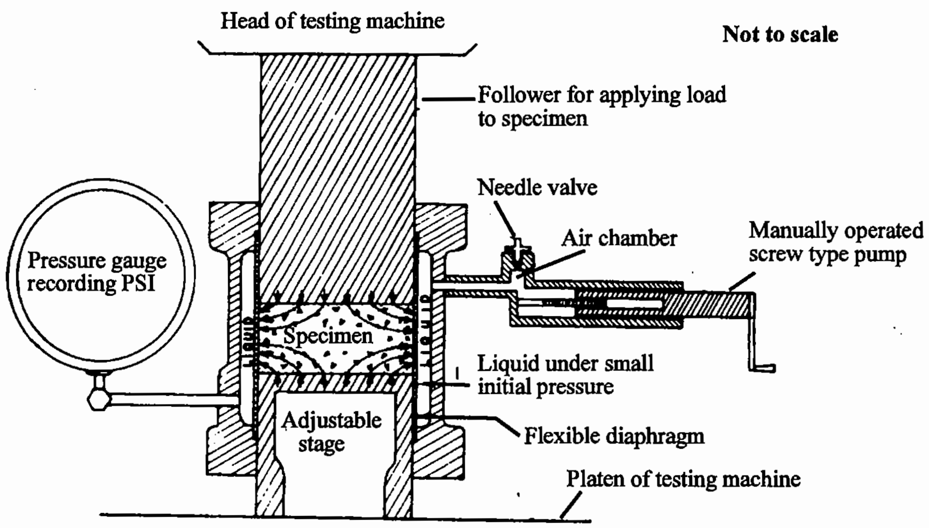


Figure 2.4 The Hveem Stabilometer (reproduced from reference 47)

The basis of the Marshall procedure, developed from the early 1940's, is a simple mechanical test for the measurement of stability, though whether this is intended to reflect the same properties as Hveem's test is not clear, since the general use of the term stability in the literature does not seem consistent. In the Marshall test (47,80), shown schematically in Figure 2.5, cylindrical specimens are loaded diametrally at a constant rate of deformation (2 inches per minute), at a temperature of 60°C, between steel jaws which partly encircle the specimen. The maximum load sustained by the specimen before failure is termed the stability and the deformation in the specimen to the point of maximum load is termed the flow. Stability is generally found to increase with increasing binder content up to a maximum value after which it declines rapidly. Flow generally increases with increasing binder content.

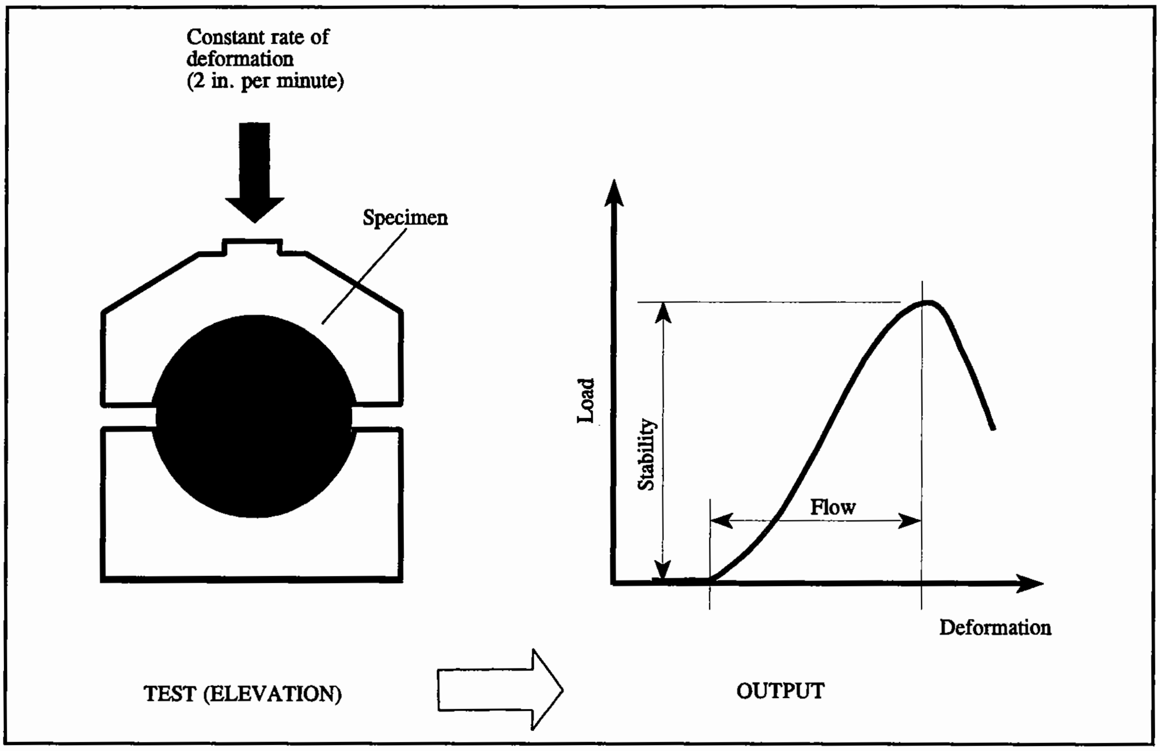


Figure 2.5 The Marshall Test

Brown et al (33) have observed that the philosophy of the Marshall procedure is to determine the maximum amount of bitumen which the mixture can tolerate without sacrificing stability, thereby ensuring a mixture which should be both durable and resistant to fatigue cracking. However, whether Marshall stability is a good measure of a mixture's resistance to permanent deformation is open to question, since there is evidence which purports to show correlation between Marshall stability and in situ rutting (55,84) and laboratory wheeltracking tests (85), while other researchers have reported a poor correlation between Marshall stability and other permanent deformation tests (20,70,86), and also the incidence of rutting in high stability mixtures (87).

Neither the Hveem nor Marshall stabilities can be described as fundamental mixture properties, and their use has been based upon empirical correlations with performance. Such methods cannot, however, reliably be used beyond the scope of the conditions under which they were originally developed. Concern in the USA over the continued use of Hveem and, in particular, Marshall for the design of mixtures subjected to greater vehicle loadings and higher tyre pressures than would have been contemplated when the procedures were developed, has resulted in the use of more fundamental tests in mixture design. This is illustrated in the philosophy of the AAMAS methodology, in which it was recognised that "*a more rational approach for mixture design would be to use those engineering properties that are related to distress*", and that to "*design mixtures based on performance-related criteria it is necessary to use a test that measures those engineering properties*" (44).

One of the objectives of the SHRP research was to develop accelerated performance tests for bituminous mixtures for application in mixture design and analysis (88), and as part of the development of a test for resistance to deformation an extensive review of existing test methods was carried out. Table 2.7, which is reproduced from the SHRP Summary Report on Permanent Deformation (19), provides a useful summary of the principal test methods and their relative merits. However, it must be borne in mind that the main purpose of deformation testing in the Superpave design system is to provide input data for a model which aims to predict the level of rutting in the mixture in service. Rut prediction constitutes a level of sophistication beyond both simple mixture design, where the aim is to optimise and compare the performance of candidate mixtures, and also beyond performance specification where the aim is some form of quantitative assessment of mixture behaviour. The requirements for a test to be used in a predictive methodology will, therefore, generally be more onerous. The SHRP researchers have stated (19) that their criteria in assessing test methods were the facility to reproduce states of stress which cause rutting in the field and, to a lesser extent, simplicity, and the subjective rankings which they assigned should be viewed in this light.

Table 2.7 Comparison of Test Methods for Evaluation of Permanent Deformation (reproduced from reference 19)

Test Method	Sample Shape	Measured Characteristics	Advantages and Limitations	Field Simulation	Simplicity	Overall Ranking
Uniaxial static (creep)	Cylindrical	Creep modulus vs time Strain vs time	Widespread, well known Easy to implement : equipment generally available in labs State of stress contains shear components in the Mohr-Coulomb representation More technical information	5	1	3
Uniaxial repeated		Resilient modulus Permanent deformation vs cycles Poisson's ratio	Better expresses traffic conditions Equipment is more complex			
Uniaxial dynamic		Dynamic modulus Damping ratio Poisson's ratio Permanent deformation vs cycles	Capability of determining the damping as a function of frequency for different temperatures			
Triaxial static	Cylindrical	Creep modulus vs time Strain vs time	More states of stress can be obtained State of stress contains shear components in Mohr-Coulomb representation	4	3-4	2
Triaxial repeated		Resilient modulus Permanent deformation vs cycles Poisson's ratio	Better expresses traffic conditions Equipment is more complex Requires a triaxial chamber			
Triaxial dynamic		Dynamic modulus Damping ratio Poisson's ratio Permanent deformation vs cycles	Capability of determining damping as a function of frequency for different temperatures			

Table 2.7 Comparison Of Test Methods For Evaluation Of Permanent Deformation (continued)

Test Method	Sample Shape	Measured Characteristics	Advantages and Limitations	Field Simulation	Simplicity	Overall Ranking
Test tracks	Slabs	Rut profile vs number of passes In depth strain/stress profile	States of stress duplicate field conditions Fundamental material properties cannot be obtained Good method for verification of predictive models Requires special equipment that can be costly	1	6	Not suitable for routine use
Diametral static (creep)	Cylindrical	Creep modulus vs time Permanent deformation vs time	Easy to implement Field cores can be easily obtained Shear stress field not uniform State of stress is predominantly tension Equipment is relatively simple in static test For repeated and dynamic tests, the complexity of the equipment is similar to that of triaxial repeated and dynamic equipment			
Diametral repeated		Resilient modulus Permanent deformation vs cycles		6	2	4
Diametral dynamic		Dynamic modulus Damping ratio Permanent deformation vs cycles				
Hollow cylindrical		Dynamic axial modulus Dynamic shear modulus Axial damping ratio Shear damping ratio Axial permanent deformation vs cycles Shear permanent deformation vs cycles	Almost all states of stress can be duplicated Capability of determining damping as a function of frequency for different temperatures for shear as well as axial Sample preparation is tedious Expensive equipment Cores cannot be obtained from pavement	2	5	Not suitable for routine use

Table 2.7 Comparison Of Test Methods For Evaluation Of Permanent Deformation (continued)

Test Method	Sample Shape	Measured Characteristics	Advantages and Limitations	Field Simulation	Simplicity	Overall Ranking
Simple shear static (creep)	Cylindrical	Shear creep modulus vs time Shear permanent deformation vs time	Shear stress can be directly applied to the specimen Cores can easily be obtained from existing pavements	3	3-4	1
Simple shear repeated		Shear permanent deformation vs cycles Resilient shear modulus	Better expresses traffic conditions			
Simple shear dynamic		Dynamic shear modulus Shear permanent deformation vs cycles Damping ratio	Capability of determining the damping ratio as a function of frequency for different temperatures Equipment not generally available			

Note : Damping refers to lag between application of stress and strain response, which is a characteristic of viscous behaviour

From their review of the literature, the SHRP researchers identified shear deformation as the primary cause of pavement rutting, with potentially some contribution from densification under traffic. Analysis of pavement structures indicated that high shear stresses occur near the surface of the pavement at the edge of the tyres and principally for this reason a test capable of directly applying shear stresses was preferred.

In the Superpave mixture design system the extent of performance testing is dependent upon the level of design, which is determined according to the anticipated traffic, as shown in Table 2.8 below.

Table 2.8 Superpave Mixture Design Philosophy

(reproduced from reference 50)

Traffic, ESALs	Testing Requirements
ESALs $\leq 10^6$	volumetric design (Level 1)
$10^6 <$ ESALs $\leq 10^7$	volumetric design + performance prediction tests (Level 2)
ESALs $> 10^7$	volumetric design + enhanced performance prediction tests (Level 3)

The testing regimes for permanent deformation, which have been established to provide the necessary degree of material characterisation for the modelling appropriate to the design level are shown in Table 2.9.

Table 2.9 Superpave Permanent Deformation Testing Regime

(abstracted from reference 45)

Traffic, ESALs	Permanent Deformation Tests
$10^6 < \text{ESALs} \leq 10^7$	Repeated shear test at constant ratio of shear to compressive stress Simple shear test at constant height at $T_{eff}^{(PD)}$ Frequency sweep test at $T_{eff}^{(PD)}$
$\text{ESALs} > 10^7$	Frequency sweep test at constant height at 4°, 20° and 40°C Uniaxial strain test at 4°, 20° and 40°C Volumetric test at 4°, 20° and 40°C Simple shear test at constant height at 4°, 20° and 40°C Repeated shear test at constant stress ratio at $T_{eff}^{(PD)}$

$T_{eff}^{(PD)}$ is the design pavement temperature appropriate for permanent deformation.

All these tests are carried out using the SHRP shear test device, the principles of which are illustrated schematically in Figure 2.6. Details of the equipment, test procedures and their evolution have been widely reported by SHRP (19,45,88,89).

Although simplicity was reportedly one of the criteria in the selection of the test method, the shear test device developed by SHRP is a sophisticated and expensive piece of apparatus capable of applying static, repeated, dynamic and ramped loads both axially and in shear, with or without confinement in a temperature controlled environment (89). The testing regimes are also lengthy and complex, requiring a number of specimens, which for shear testing must be bonded to the loading platens. Haydon (23) has recently criticised the development and use of the shear tester for mixture design on the cost of the device (in excess of $US 350 000), the length of time required to carry out a design and the need for staff trained to a higher level than would otherwise be required in a commercial laboratory. Similar concerns have been raised in a recent publication on the implementation of Superpave, produced by the National Asphalt Pavement Association (NAPA) which represents the hot mix producers and contractors in the USA (90). NAPA have welcomed the Level 1 volumetric design procedures but taken the view that the SHRP shear tester, while being a useful research tool, is not a practical proposition for routine mixture design or quality control, for broadly the same reasons as stated by Haydon.

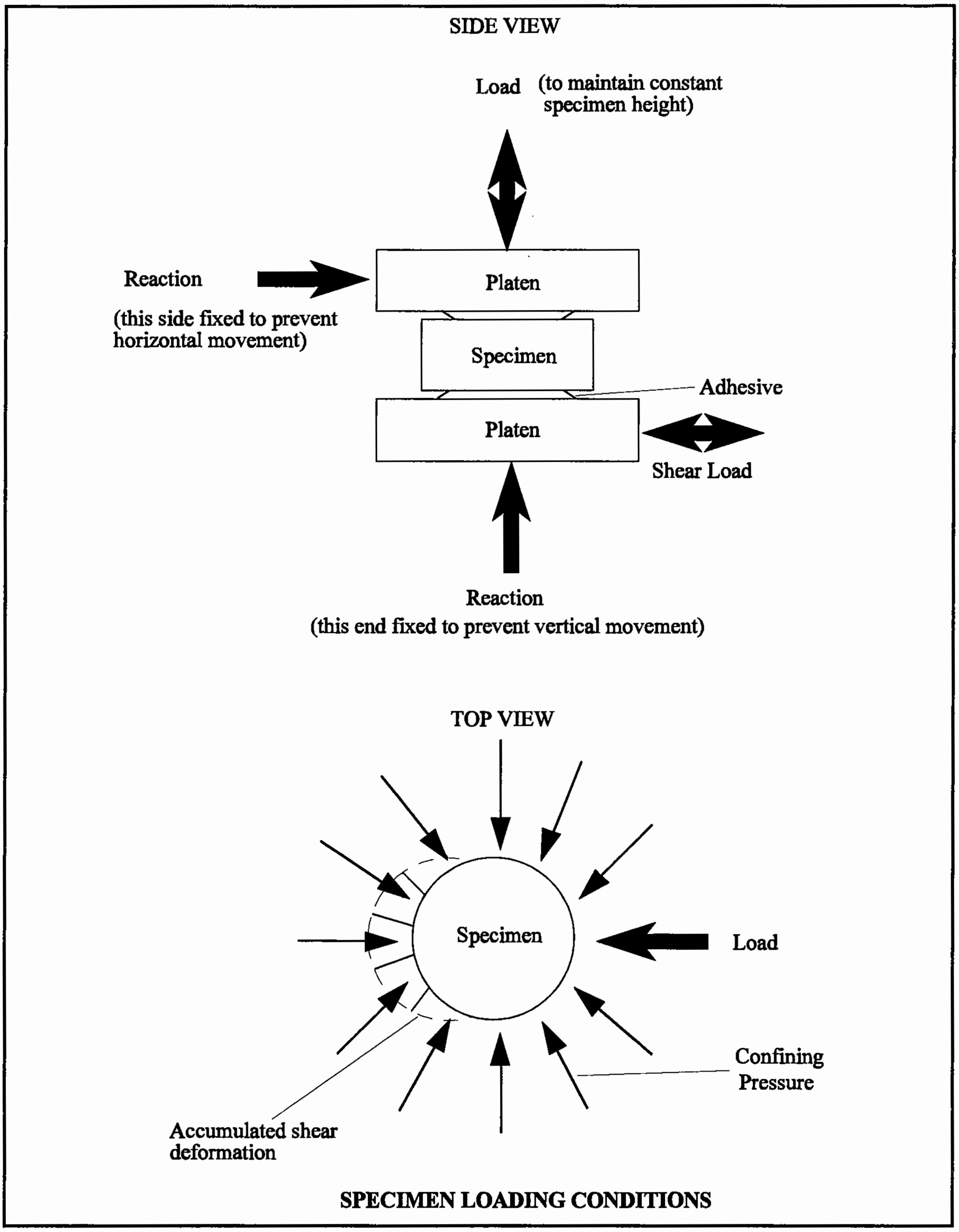


Figure 2.6 Superpave Shear Test Device

The NAPA report has gone as far as suggesting simpler alternative devices including wheeltrackers and a repeated load axial compression test. Haydon has also argued that the latter form of test may be theoretically acceptable for use on basecourse and roadbase materials, which would not be subjected to such high shear stresses.

This is a pertinent practical observation with regard to the situation in the UK where it is known that replacement of wearing course on trunk road and motorway is generally governed by texture requirements for skidding resistance rather than permanent deformation considerations. Haydon has also noted that the traffic levels on this network are such that under the Superpave system virtually all of it would require the most detailed level of design.

The authors of the Superpave deformation methodology appear to have accepted that the full mixture characterisation procedure is as yet unsuited to routine use and have suggested a simplified procedure based only on the use of the constant height repetitive simple shear test (CHSST) and an abridged analysis (89,91). Although this is deemed a simplified procedure it still requires the use of the SHRP shear tester, though it seems likely that a simpler and cheaper portable version of the equipment will be produced (89).

Interestingly, Sousa et al (92) have observed that in the CHSST, in which the height of the specimen is constrained to remain constant so that no volume change occurs, mixtures which perform well in the field when placed with void contents of 5 - 7%, have failed quickly when tested at a void content of 5%. This was attributed to the fact that, in the field, further compaction takes place under trafficking which improves the performance of the mixture up to the point where the voids become overfilled and it was concluded from this that compaction of test specimens to representative field density is of vital importance (see Section 2.3.3). However, on this basis it may be argued that methods of test in which densification can take place would better represent the performance of material in situ.

Simplicity of equipment and procedure are undoubtedly factors in the success and longevity of the Marshall test and these, together with the existence of a large database built up through long experience of use, were cited as reasons for its selection as a verification test in the Belgian CRR mixture design method. In 1985, van der Heide, in discussing the use of the Marshall method in Holland (93) stated that the use of theoretically sound tests for mixture design was desirable but impractical since design was carried out by contractors whose laboratories were unlikely to be capable of carrying out complex tests accurately. Since that time, the Dutch have undertaken research to develop a performance-based method for the replacement of Marshall in order to cope with increases in traffic, vehicle loads and tyre pressures and to make use of new materials and technology. While sophisticated tests for mixture characterisation are in use in research in Holland, the relevant CROW working group recognised they would not be suitable for daily practice under a contractual system where the contractor performs the mixture design due to the complexity, costs, time and level of operator expertise required (70). They concentrated, therefore, on assessment of simple test methods and having considered, among other things, simulation of stress states they ultimately recommended a repeated load unconfined, uniaxial compression test for assessing resistance to deformation.

Of the twelve mixture design methods presented in Table 2.1, half make use of an unconfined uniaxial compression test, with either static or repeated loading, to assess resistance to deformation.

2.5 Summary

2.5.1 The Importance of Resistance to Permanent Deformation in Mixture Design

Mixture characteristics which favour different aspects of performance often conflict. For example, low binder contents are generally beneficial for resistance to deformation (78) while high binder contents enhance durability and resistance to fatigue cracking. Mixture design is, therefore, usually a compromise.

There are differences in the emphasis placed on resistance to permanent deformation between the various design methods reviewed in the preceding sections. It has already been noted that the primary aim of the Marshall method, as specified by the Asphalt Institute, is to ensure a mixture which is durable and resistant to fatigue through using as much bitumen as possible without sacrificing stability. Hveem was evidently of the same opinion, judged from the following quote attributed by Vallerga and Lovering (34) : *"... it seems to be the best rule to use a dense, uniformly graded mixture without an excess of dust and to add as much oil or asphalt as the mixture will tolerate without losing stability".* Finn et al (59) have, however, stated that subsequent development of the Hveem method has seen local adaptation to suit the particular climatic and traffic conditions, with its use in California, where it originated, having been tailored to produce mixtures which do not rut in a hot desert environment.

The Nottingham method is geared to producing lean, rut resistant mixtures which have sufficient binder for fatigue resistance. The South African LAMBS method, which appears to be largely derived from the Nottingham method, places similar emphasis on permanent deformation resistance. Haydon (23) has suggested that deformation, together with workability, should be the focus of mixture design since other properties can be reasonably estimated from volumetric composition.

Both the Nottingham and South African methods are intended primarily for the design of roadbase and basecourse materials since it is recognised that for wearing courses other considerations, including a greater requirement for durability, may apply. In the TRL study in Jordan (51) cracking in the surfacing, attributed to embrittlement of the binder under the harsh environmental conditions, was a major problem of mixture performance in addition to severe rutting. It was recognised that the use of a binder content which would mitigate cracking would probably lead to very rapid failure through deformation. The preferred solution in this situation was to design the mixture for resistance to deformation, and prevent or retard cracking through attention to the pavement construction, which in this case involved the use of a surface dressing to seal the surface from air and ultra-violet light.

2.5.2 Mixture Design Philosophy

The majority of the methods reviewed were developed from and/or are intended primarily for the design of continuously graded materials. As a result some of these methods appear more like a specification for asphaltic concrete than a method of mixture design. The Superpave system for example, while using performance tests which are in principle applicable to any mixture type, includes, in addition, stringent recommendations or requirements for aggregate properties and grading, binder properties and volumetric criteria.

In contrast, the French LCPC method (94) is a more genuinely performance-based system, requiring only that a mixture meets established performance criteria. Goacolou et al (95) have described how this approach to mixture design permitted the use of local aggregates, which did not comply with French national specifications, in the construction of a section of motorway and resulted in a solution which was both economically and environmentally beneficial due to the reduced requirement for aggregate extraction. This type of approach is made possible by the contractual system in France, as described by Bonnot (96), which promotes the development and use of innovative materials. The organisation of the industry in France is also an important factor. Several large contracting organisations dominate the market and have generally invested in their own testing and research facilities.

2.5.3 Conclusions

Marshall designed mixtures have been found to be prone to rutting and dissatisfaction in general with empirical methods of design has led to the development of more fundamental procedures.

Mechanical testing for the assessment of resistance to permanent deformation is a key element of such procedures. The tests used must be effective but should also be simple if they are to be widely accepted. This is essential if mixture design is to be carried out routinely by material producers and suppliers, which would be the case under a system of performance specification.

CHAPTER THREE
EVALUATION AND COMPARISON OF SIMPLE TEST SYSTEMS

3.1 Wheeltracking Tests

3.1.1 Use of Wheeltracking Tests

Wheeltracking tests have an obvious appeal for the assessment of resistance to permanent deformation since they are simple in principle and simulative in nature, and this form of test is used in both the French LCPC and Finnish ASTO methods of mixture design reviewed in the previous chapter. In the development of the LCPC wheeltracker, simplicity was recognised as being of prime importance if the test was to be widely used (97), and this has been achieved as the equipment is not only used in France by commercial organisations as well as the LCPC itself but has been adopted elsewhere in Europe (98) and also in the USA (99,100). In the test, which is shown in Figure 3.1, a specimen measuring 500mm in length, 180mm in width and, usually, 100mm in depth is tracked by a pneumatic tyred wheel with an inflation pressure of 6 bar and an applied load of 5 kN (101).

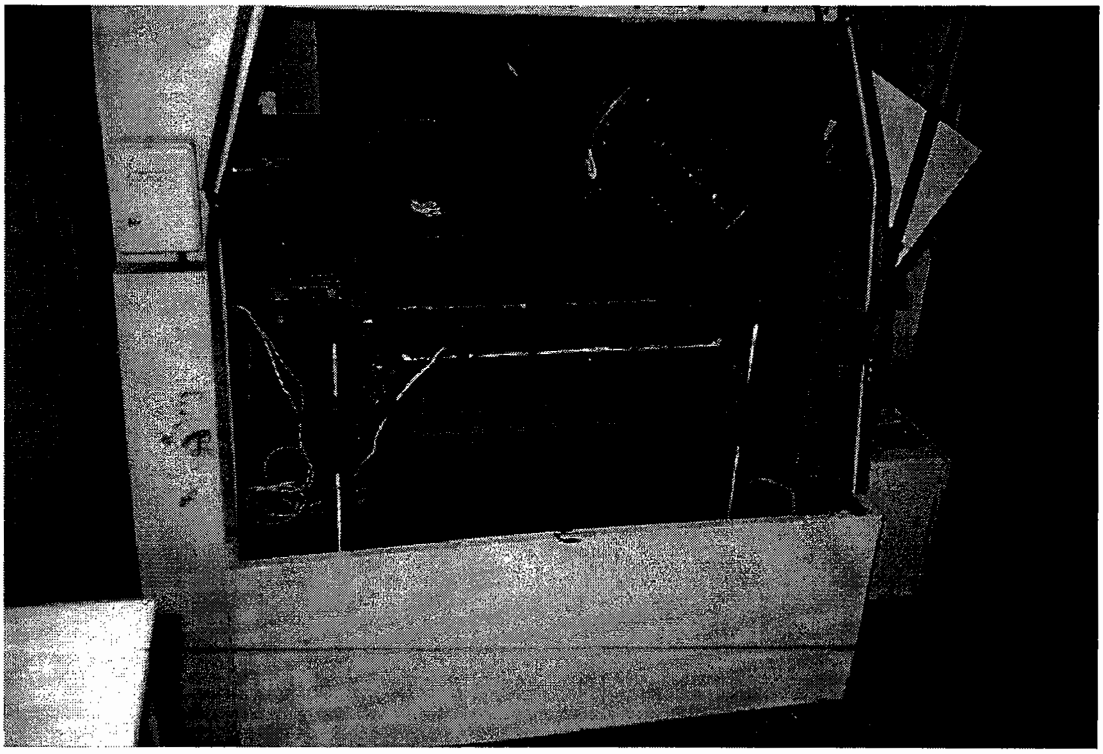


Figure 3.1 LCPC Wheeltracking Device

Tests in which the width of the specimen is small in relation to the width of the wheel, as in this case, have been criticised because of the degree of lateral confinement provided by the rigid test mould, which influences stress distribution and rut formation (91,102). This is, perhaps, unimportant in tests which are intended as index tests rather than for use in predictive methodologies, and there is evidence to suggest that the LCPC test is able to rank the performance of mixtures (99) and also correlates with observed performance (100). Wheeltracking tests in use for some years in Holland (102) and the US state of Georgia (103) also use narrow beam specimens, having widths of 110mm and 125mm respectively. The latter test, which is claimed both to reflect observed performance and rank mixtures effectively (103), has been advocated as a simpler and cheaper alternative to the simple shear tester in the Superpave system (90).

3.1.2 Evaluation of a Simple Wheeltracking Test

The opportunity to evaluate a mechanically simple wheeltracking test was provided by a research contract which formed part of the US Strategic Highway Research Program (SHRP). The test equipment used, shown diagrammatically in Figure 3.2, was derived from that developed by the TRL, primarily for assessing the rutting resistance of HRA wearing course mixtures (104).

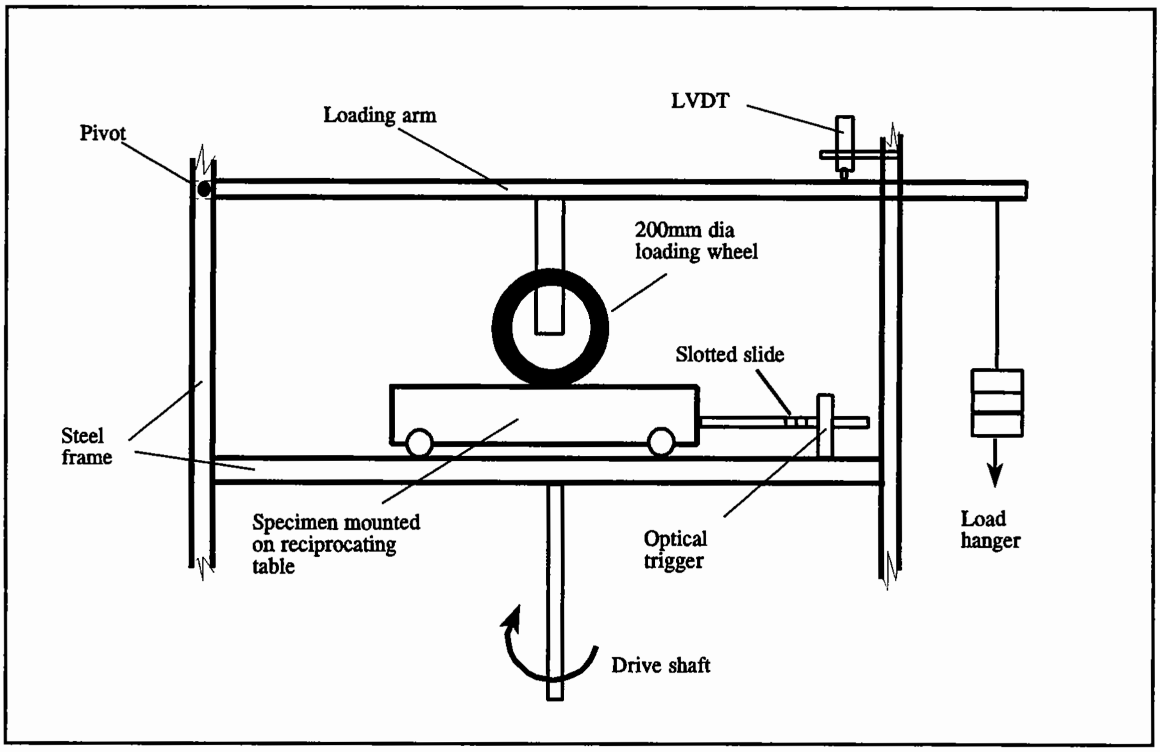


Figure 3.2 Wheeltracking Apparatus

The test specimen, which may be either a 200mm diameter core or a slab 404 × 280mm in plan, is confined in a rigid mould and driven to and fro beneath a loaded wheel which is mounted on a pivoted lever arm through which a constant load is applied. The wheel has an overall diameter of 200mm, a width of 50mm and is fitted with a tyre of solid rubber which measures 80 on the Dunlop hardness scale. Rotation of the drive shaft from an electric motor is translated to a linear reciprocating motion by means of a cam, the geometry of which fixes at 225mm the distance of travel of the trolley to which the specimen is attached. The rate of oscillation is governed by the speed of the motor and is set at 42 passes per minute. To record the development of rutting in the specimen as the test progresses the vertical position of the wheel is monitored by means of a linear variable differential transformer (LVDT) mounted on the lever arm. A slotted slide attached to the specimen passes through an optical trigger, causing the LVDT signal to be captured at eleven positions at 10mm centres along the central 100mm length of the wheel's travel in each pass. The data is acquired via an analogue-digital converter and written directly to a microcomputer which stores the data periodically throughout the test.

3.1.3 Test Programme

The test programme for evaluation of the wheeltracking test was specified by the University of California Berkeley (UCB), SHRP's main contractor for the development of test systems for bituminous mixtures. In common with much of the SHRP research, the test programme followed a statistically derived experimental plan which covered a range of mixture and test variables. In this case the variables were aggregate type, aggregate grading, binder type, binder content, air void content and test stress level. The possible values and reference codes for the variables are shown in Table 3.1.

Table 3.1 Variables in SHRP Programme for Evaluation of the Wheeltracking Test

Variable	Possible Values	Reference Code	Remarks
Bitumen Type	Boscan	B	High PI
	Valley	V	Low PI
Binder Content	Optimum	0	Hveem design
	High	1	Marshall design
Aggregate Type	Texas chert	T	Dredged gravel
	Watsonville granite	W	Crushed rock
Aggregate Grading	Medium	M	See Figure 3.3
	Coarse	C	See Figure 3.3
Air Void Content	Low	0	4%
	High	1	8%
Test Stress	Low	0	650 kPa
	High	1	950 kPa

Thirty two combinations of these variables were to be tested with each test replicated once to give a total of 64 planned tests. The experimental plan is shown in Table 3.2.

Table 3.2 SHRP Experimental Plan for Evaluation of the Wheeltracking Test

Test Number	Binder Type	Binder Content	Aggregate Type	Aggregate Grading	Air Void Content	Test Stress
1	B	0	W	M	0	0
2	B	0	T	M	0	1
3	B	0	W	C	0	1
4	B	0	T	C	0	0
5	V	0	W	M	0	1
6	V	0	T	M	0	0
7	V	0	W	C	0	0
8	V	0	T	C	0	1
9	B	1	W	M	0	1
10	B	1	T	M	0	0
11	B	1	W	C	0	0
12	B	1	T	C	0	1
13	V	1	W	M	0	0
14	V	1	T	M	0	1
15	V	1	W	C	0	1
16	V	1	T	C	0	0
17	B	0	W	M	1	1
18	B	0	T	M	1	0
19	B	0	W	C	1	0
20	B	0	T	C	1	1
21	V	0	W	M	1	0
22	V	0	T	M	1	1
23	V	0	W	C	1	1
24	V	1	T	C	1	0
25	B	1	W	M	1	0
26	B	1	T	M	1	1
27	B	1	W	C	1	1
28	B	1	T	C	1	0
29	V	1	W	M	1	1
30	V	1	T	M	1	0
31	V	1	W	C	1	0
32	V	1	T	C	1	1

3.1.4 Materials

Since the research formed part of SHRP, the aggregates and binders were supplied from the SHRP materials reference library (MRL) which had been established to distribute selected standard materials, sampled centrally, to researchers working in various institutions.

Aggregates : The aggregates were a crushed granite from Watsonville in California, and a dredged gravel from the Gulf Coast, Texas. These were selected in the expectation of giving different performance in terms of resistance to permanent deformation, the former being rugous and angular and the latter being smooth and rounded.

Aggregate Gradings : Two target aggregate gradings were specified, nominally medium and coarse, derived from the specification for asphaltic concretes used in the state of California. They are presented graphically in Figure 3.3. A tolerance of only ± 1% was permitted on the percentage passing each specified sieve size.

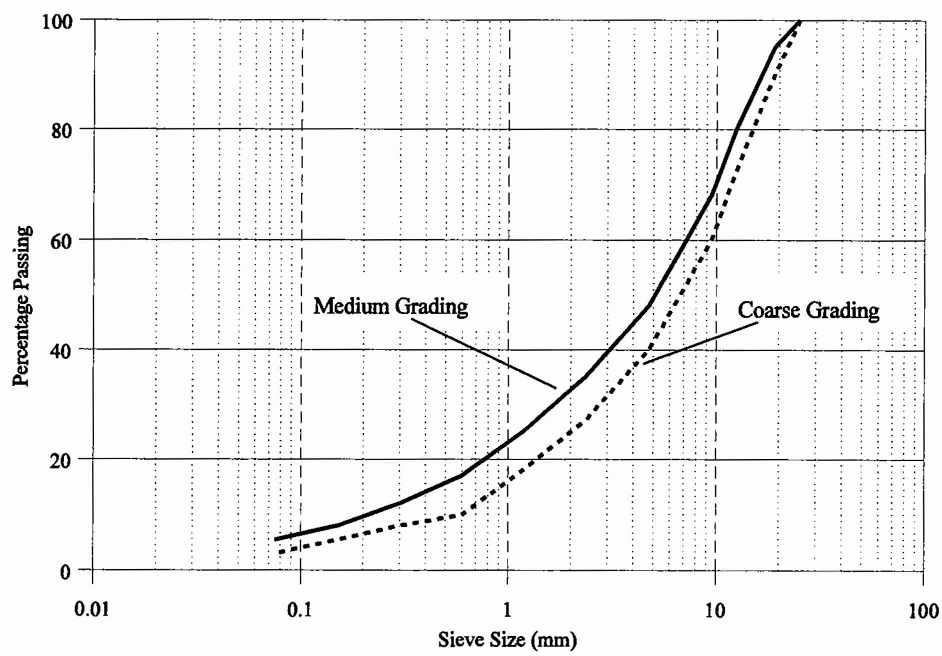


Figure 3.3 Target Aggregate Gradings

Bitumens : The two bitumens supplied were Boscan from Venezuela and Valley from California. These binders exhibit different rates of change of viscosity with temperature, as indicated by the difference in the values of Penetration Index (PI), a commonly used measure of temperature susceptibility (105). The basic binder data are shown below :

Bitumen	Penetration	Softening Point (°C)	Penetration Index
Valley	53	47	-1.34
Boscan	70	48	-0.53

The Californian Valley bitumen, having the lower PI, is the more temperature susceptible of the two.

Binder Contents : The optimum and high binder contents were determined by UCB for each bitumen/aggregate combination by Hveem and Marshall design respectively. The actual binder contents used for each mixture are shown in Table 3.3. In each case the same binder content was used for both medium and coarse gradings.

Table 3.3 Binder Contents Used in Specimen Manufacture

Aggregate	Binder	Binder content by mass of mixture (%)	Mixture Reference
Texas chert	Valley	4.1	V0T
Texas chert	Valley	4.8	V1T
Texas chert	Boscan	4.3	B0T
Texas chert	Boscan	5.0	B1T
Watsonville granite	Valley	4.9	V0W
Watsonville granite	Valley	5.5	V1W
Watsonville granite	Boscan	5.1	B0W
Watsonville granite	Boscan	5.7	B1W

3.1.5 Specimen Manufacture

Test specimens were manufactured as slabs measuring 404mm × 280mm in plan and 75mm deep, and batch masses were calculated to give the appropriate void content when compacted to this thickness. The principle of using equiviscous mixing and compaction temperatures was adopted to ensure that for each mixture the bitumen would be of similar viscosity during these processes. Target kinematic viscosities were 170 ± 20 centistokes (approximately equivalent to a dynamic viscosity of 1.7 Poise) and 280 ± 30 centistokes (2.8 Poise), based on original binder properties, for mixing and compaction respectively, these values being consistent with the recommendations of the Asphalt Institute (47). In accordance with a specimen preparation protocol approved by UCB, binder and aggregate were mixed in heated mixers and then stored loose at 60°C for fifteen hours, before being reheated to compaction temperature.

The loose material was compacted within rigid moulds using the roller compactor shown in Figure 3.4.

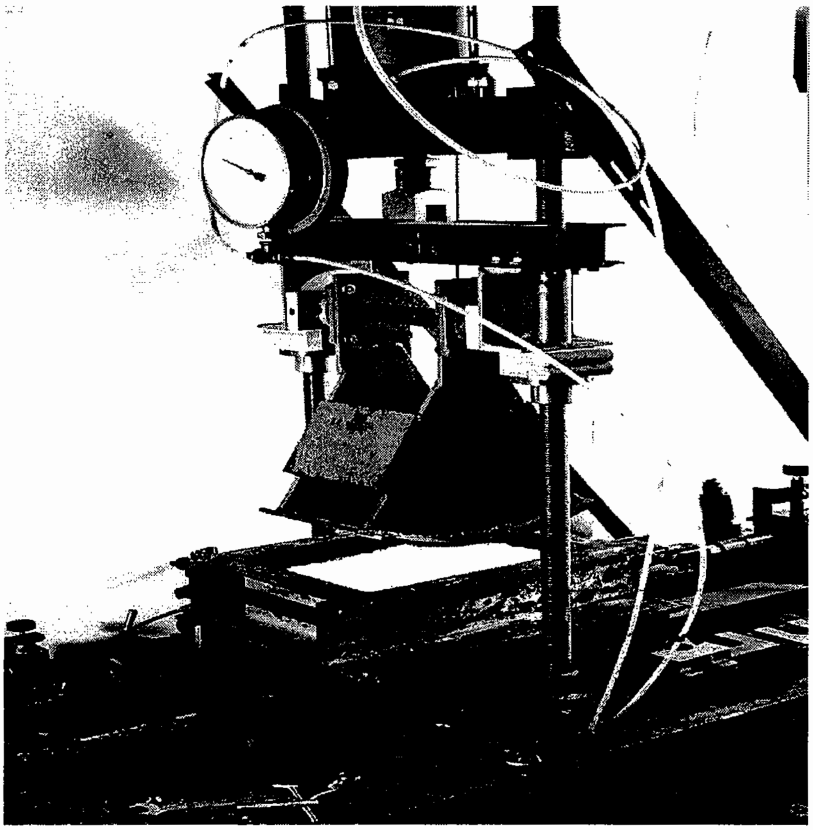


Figure 3.4 Laboratory Roller Compactor

This roller compactor, which was designed to emulate the action of site plant, comprises a pivoted steel roller segment which is forced into contact with the hot mixture by a vertically mounted pneumatic actuator. A horizontal ram drives the mould back and forth to simulate a to-and-fro rolling action. Final compaction to obtain a level surface to the slab was achieved by placing a heated aluminium plate, of the same plan dimensions as the specimen, on the material just prior to completion of compaction. Compaction to a thickness below 75mm was prevented by fitting two notched jigs over the top of the mould, as shown in Figure 3.5.

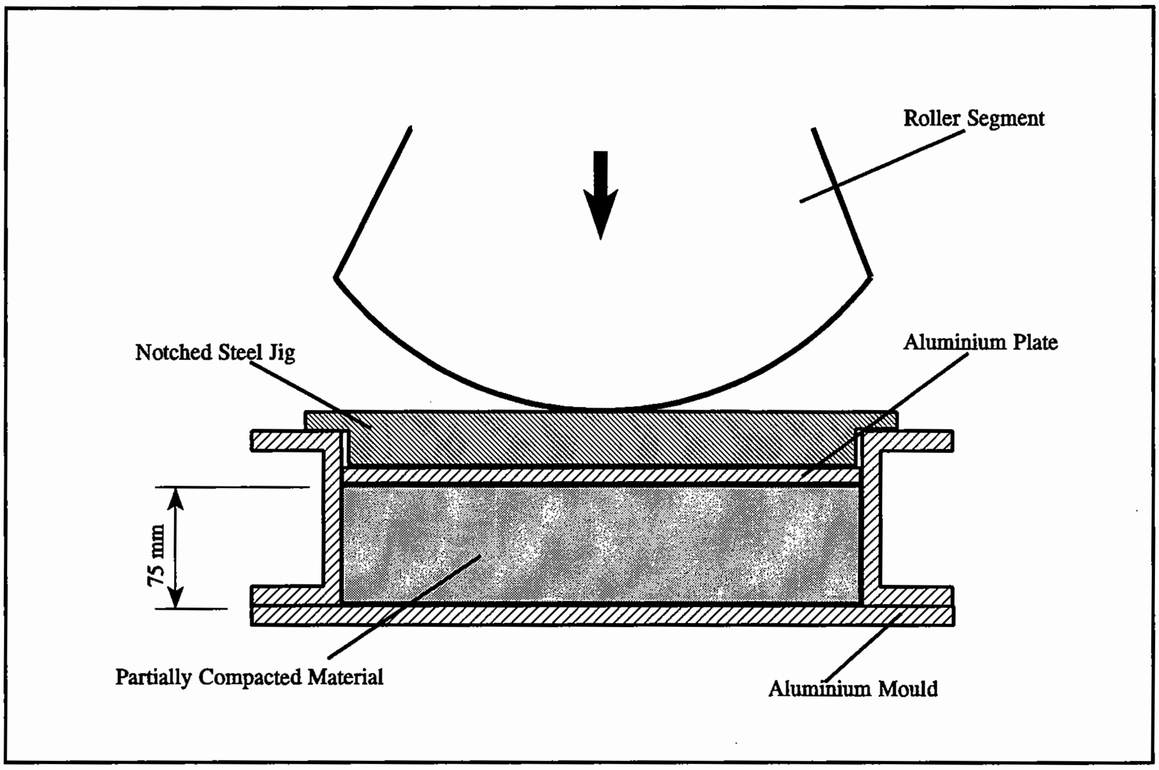


Figure 3.5 Section Showing Arrangement for Final Compaction

3.1.6 Test Conditions

All testing in this programme was carried out at a temperature of 40°C and each test was run for a duration of 5000 load passes (approximately two hours). Tests were performed with applied loads of 554 N and 809 N. The contact area of the tyre was measured, by imprint, as 850mm^2 which gave corresponding contact stresses of 650 kPa and 950 kPa.

3.1.7 Results of Wheeltracking Tests

The data from a typical test series comprising two replicate tests is presented graphically in Figure 3.6. The plots, the form of which are consistent with that generally obtained from permanent deformation tests (see Chapter Six), show the accumulation of permanent deformation with increasing number of wheel passes. The value of permanent deformation recorded was an averaged value obtained, at every 20th pass, by calculating the area of the rutted section using Simpson's rule from the eleven individual readings taken over the central 100mm of the tracked length, as illustrated in Figure 3.7. The average depth of the deformation was then computed, given the base length of 100mm and assuming that the section was rectangular over this length.

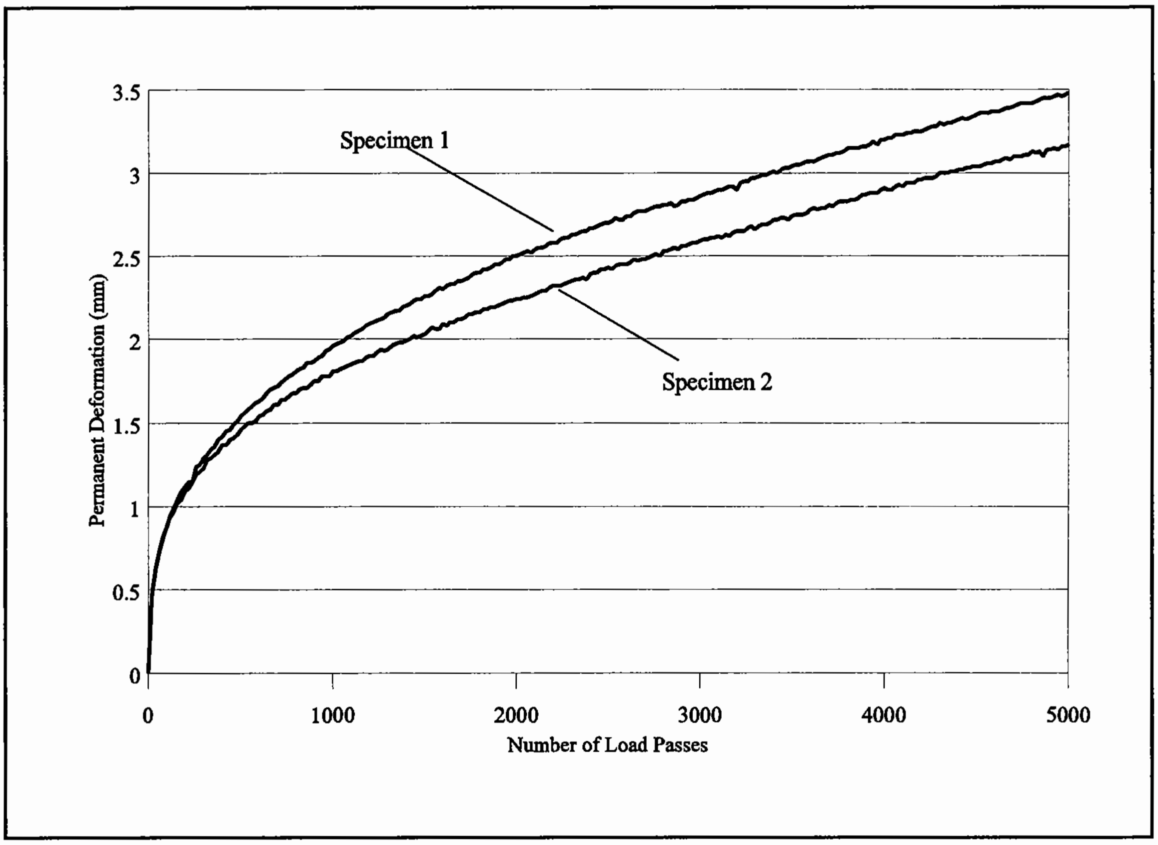
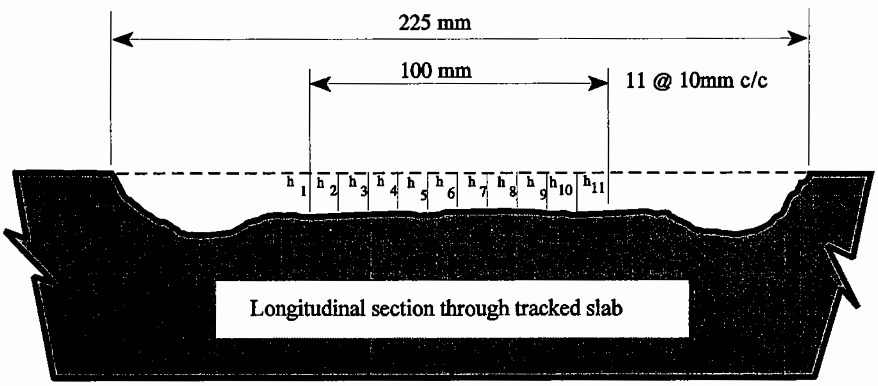


Figure 3.6 Test No. 16 : Typical Data Plots



$$\bar{h} = 1/_{30}\ (h_1 + 4h_2 + 2h_3 + 4h_4 + 2h_5 + 4h_6 + 2h_7 + 4h_8 + 2h_9 + 4h_{10} + h_{11})$$

Figure 3.7 Use of Simpson's Rule to Obtain Average Permanent Deformation

The results reported to UCB from each test were the permanent deformation (mm) at the end of the test and the rate of increase in permanent deformation (mm/hr) between 2000 and 4000 load passes. This interval was selected for determination of the rate of deformation because in all cases the linear phase of the curve was well established by this stage in the test, as can be seen in Figure 3.6. Linear regression was used to determine the gradient of the curve, and hence the rate of deformation, over this range. As the rate of tracking was constant at 42 passes per minute the deformation rate could be expressed in mm per hour. A similar parameter is used in the Dutch test (102).

3.1.8 Analysis of Results

The statistically designed test programme stipulated by UCB did not permit explicit evaluation of the effect of any one variable, nor did it include a satisfactory degree of replication for such an analysis. In order to permit further analysis certain assumptions had to be made to reduce the number of variables and effectively increase the degree of replication. These assumptions and their justification are discussed below.

3.1.8.1 Aggregate Grading

Although UCB's test programme specified two distinct target gradings, in practical terms the differences between these two grading curves, as shown in Figure 3.3, would constitute a rather tight specification envelope for a production material. On this basis it was considered reasonable to disregard grading as an explicit variable

3.1.8.2 Void Content

The void content for each tracked specimen was taken as the average of separate determinations made on four 100mm diameter cores taken from the slab, outside the tracked section, after testing. Figure 3.8 shows the position of the cores. A fifth core was taken through the tracked section to determine, as a matter of interest, whether densification had occurred under tracking though no evidence of this was found. After trimming the cores were allowed to dry in air at room temperature for a minimum of 24 hours before determination of density. Measurement of volume was by displacement in water with the cores sealed during immersion. Theoretical maximum densities for each mixture were determined by UCB using the Rice method (53).

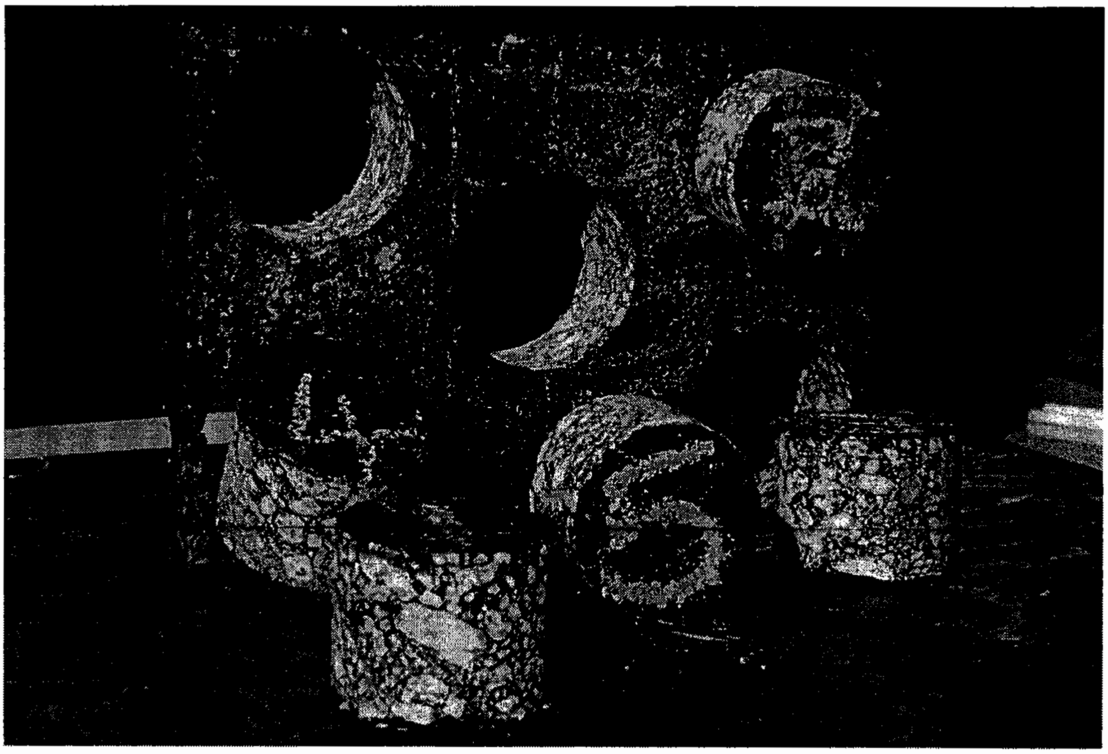


Figure 3.8 Location of Cores Through tracked Slab

Differences in the compactibility of the mixtures caused difficulty in achieving the levels of void content specified in UCB's test plan. Additional specimens were, therefore, manufactured and tested, primarily to provide further opportunity to obtain the target voids though if this could not be achieved it was hoped the additional replication would give a better indication of the effect of this parameter on performance. The ranges and mean values of void content for each mixture are shown in Table 3.4.

Table 3.4 Air Void Content Data

Mixture Reference	Air Void Content (%)			
	Maximum	Minimum	Mean	Standard Deviation
B0T	9.2	4.4	7.3	1.5
B0W	7.9	2.7	5.5	1.8
B1T	9.6	4.5	7.2	1.6
B1W	8.3	1.9	4.7	1.8
V0T	12.5	4.5	8.7	2.0
V0W	8.5	4.9	6.5	1.0
V1T	10.7	5.4	8.1	2.0
V1W	8.5	3.2	5.5	1.6

There is, however, no indication from the test results of any consistent effect of void content on performance, as demonstrated in Figures 3.9 and 3.10 which show data from two test series. These results were not untypical and are consistent with data reported by Sosnovske et al (99) which shows little or no effect of air voids on performance in the LCPC test, except at very high void contents (16 - 20%) which would indicate poorly compacted material.

Since the effect of void content could not be discerned, perhaps because over the range of values obtained its effect is small compared to other parameters or the variability inherent in the test, it was considered that this could be excluded as an explicit variable.

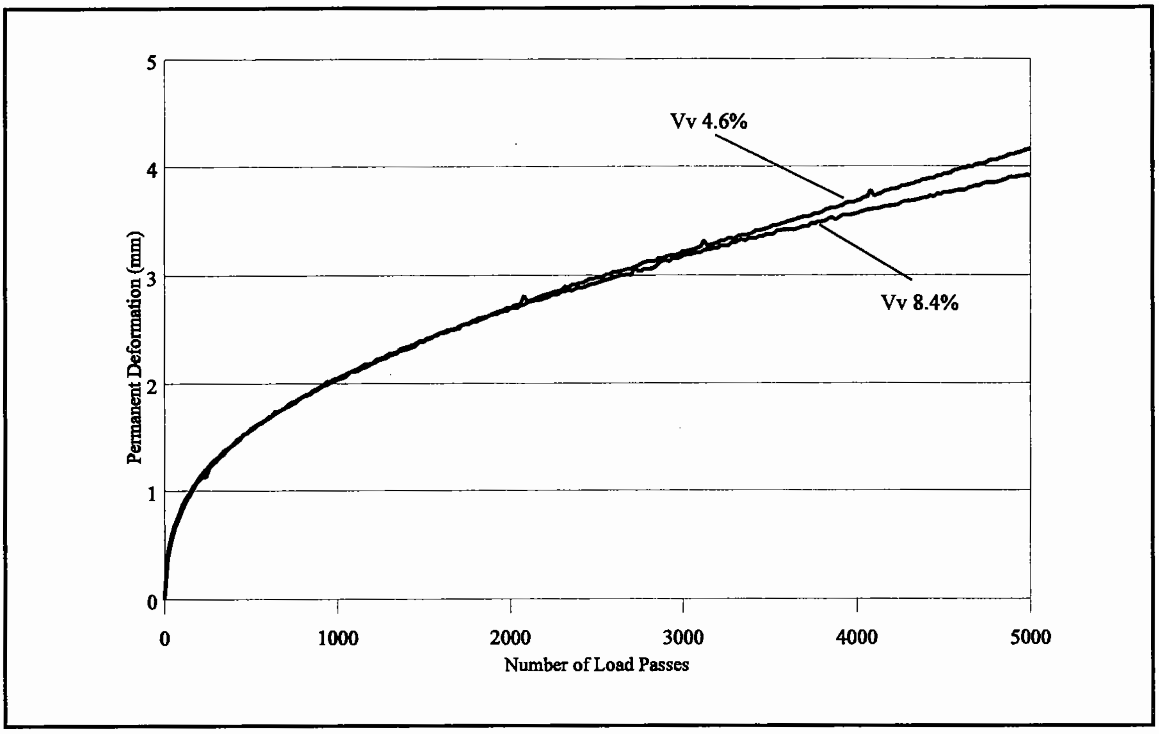


Figure 3.9 Variation in Void Content and Performance : Test No. 8, Mixture V0T

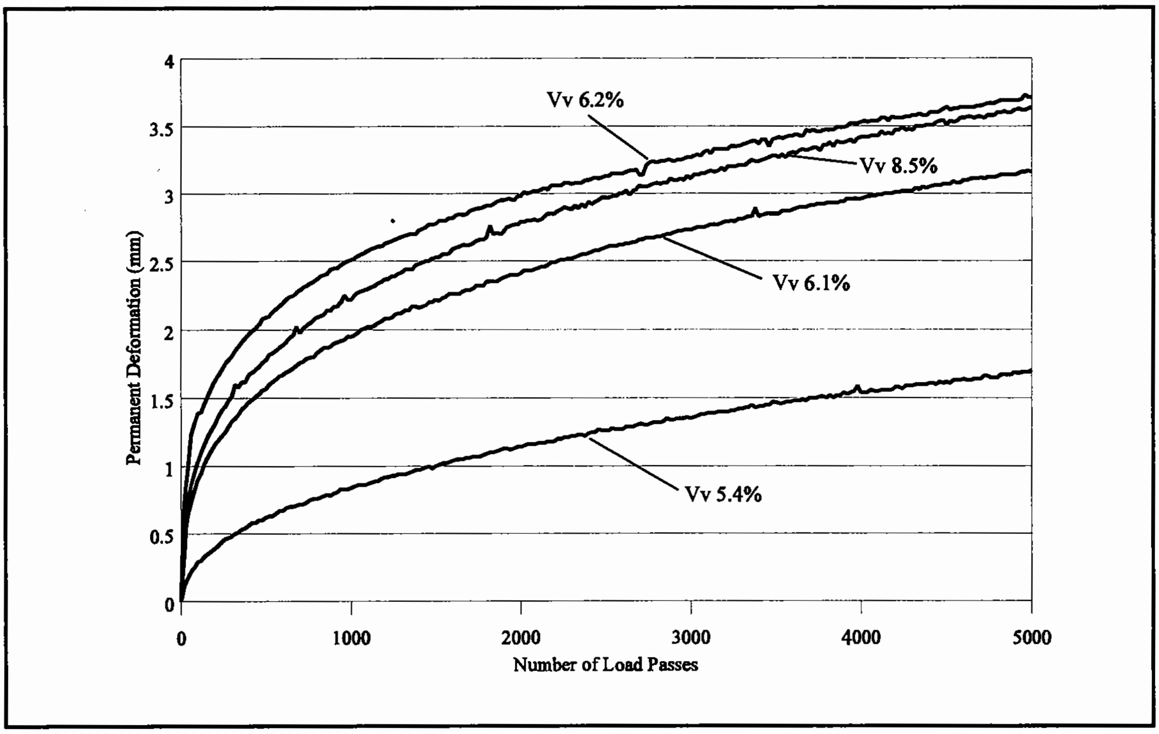


Figure 3.10 Variation in Void Content and Performance : Test No. 29, Mixture V1W

3.1.8.3 Normalisation of Test Stress

Deformation rate was chosen as the parameter to compare performance of different mixture combinations since, unlike the total permanent deformation, it is not affected by initial start-up errors. In order to reduce the data further, the deformation rate was normalised by dividing by the appropriate value of contact stress, thus allowing results of tests at both stress levels to be included in the same analysis. The assumption implicit in combining results from the two stress levels in this manner is that the behaviour of the material is linear for the given test temperature and rate of loading. This assumption is supported by data presented by Pagen (106) and has been used by van de Loo (86) in comparison of uniaxial and wheeltracking tests.

3.1.9 Discussion of Results

Having eliminated void content, aggregate grading and test stress level as explicit variables it was possible to compare the performance of mixtures which differed only in bitumen type, binder content and aggregate type. The values of normalised deformation rate, which are the mean of at least eight results, for each mixture combination, are presented in Table 3.5, and also shown graphically in Figure 3.11.

Table 3.5 Normalised Deformation Rates

Mixture Reference	Mean Normalised Deformation Rate (m/GPa.hr)	Standard Deviation (m/GPa.hr)
B0T	0.320	0.138
B1T	0.524	0.312
V0T	1.163	0.274
V1T	1.460	0.456
B0W	0.224	0.121
B1W	0.310	0.236
V0W	0.499	0.182
V1W	0.686	0.186

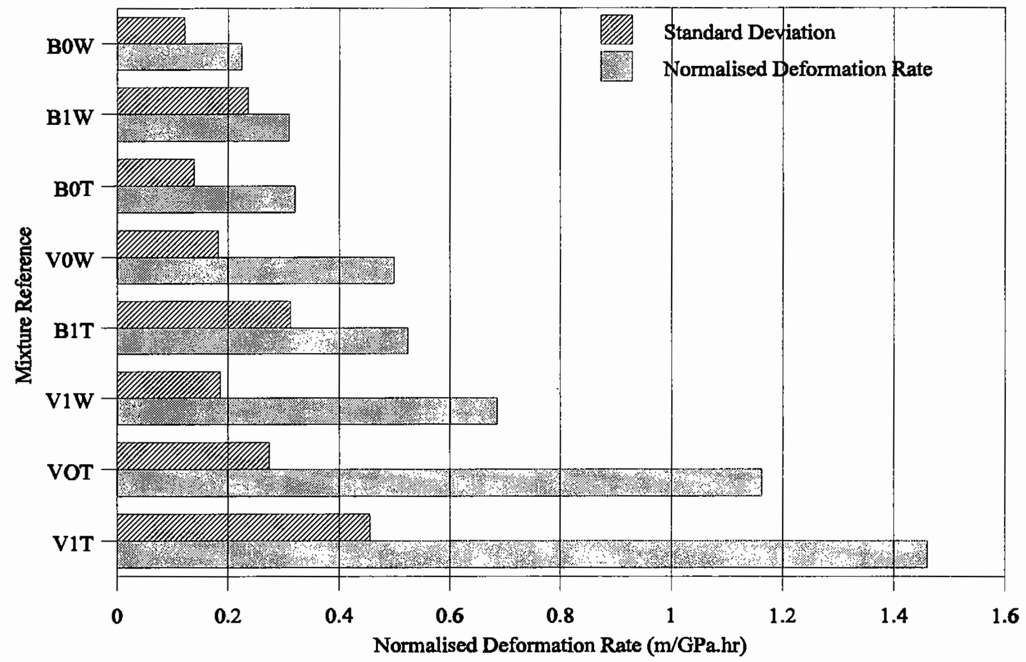


Figure 3.11 Comparison of Mixture Performance in the Wheeltracking Test

The ranking of mixture performance appears logical. The granite aggregate showed a lower deformation rate than the gravel when in combination with the same bitumen and for each combination of bitumen and aggregate a higher binder content produced a higher deformation rate. The bitumen type appears to be the most significant variable with the performance of the Valley being much poorer performance than the Boscan, which is less temperature susceptible.

The standard deviation associated with the mean deformation rate for each mixture is also shown in Table 3.5 and Figure 3.11. It is evident that the degree of scatter in the results is quite high. This is particularly significant where the absolute value of deformation rate is low, and would imply that a significant number of replicate tests would be needed if the results were to be reliable.

3.1.10 Summary

On the basis of these results it would appear that the wheeltracking test is able to rank effectively the rutting resistance of bituminous mixtures when using normalised rutting rate as the performance parameter, with bitumen type having the most significant effect on performance. However, from the observed degree of scatter it would appear that the repeatability of the test is low and a significant number of replicate tests would have to be performed to obtain a reliable determination of rutting rate.

For the better performing mixes the absolute values of final rut depth measured were small. It may, therefore, be beneficial to change the test conditions with the aim of producing increased rut depth in order to reduce the significance of error in measurement and enhance discrimination between mixtures. This could be achieved by increasing the test stress, increasing the test temperature or reducing the rate of tracking. Experience with the Georgia wheeltracker has shown that low levels of stress fail to distinguish between mixtures which show clearly different performance at higher stresses (107). However, the test stress of 950 kPa would constitute a high value and it may therefore be better to consider an increase in temperature or reduced loading speed. In the LCPC test temperatures of 50°C and 60°C are used for base and wearing course materials respectively.

No consistent effect of void content on performance could be discerned.

3.2 Uniaxial Compression Tests

3.2.1 Development of Uniaxial Tests

The use of a static unconfined uniaxial compression test, termed the creep test, for assessing the permanent deformation resistance of bituminous materials was developed in the 1970s by the Shell organisation (35). This test gained wide acceptance, principally due to ease of specimen preparation, simplicity of test procedure and low cost of test equipment. The only requirements for the test specimen were that it should be prismatic with flat and parallel ends normal to the axis of the specimen, while the test procedure comprised simply the application of a constant stress to the specimen for one hour at a constant temperature and measurement of the resultant deformation. The test equipment was, therefore, rather simple and early versions of the equipment generally applied the load as a dead weight via a mechanical lever arm (108).

Shell developed a rut prediction procedure based on the creep test, but it was found that the method under-predicted rut depths measured in trial pavements (109). This was attributed to the effects of dynamic loading producing higher deformations in the wheeltracking tests (86). An experimentally derived empirical correction factor which varied according to mixture type was, therefore, subsequently introduced into the creep analysis to account for these effects.

In 1983 Finn et al, in an assessment of the creep test and analysis procedures during investigations of pavement rutting (59), recommended that consideration be given to the development of a repeated load test. The effect of this form of loading in inducing plastic strains in the aggregate skeleton, which contribute significantly to permanent deformation, had already been recognised by Goetz et al (110) who had developed an early version of a rapid cycle repeated load test by 1957. Further concerns over the use of static loading have arisen more recently with evidence to suggest that the static test does not reflect the improved performance of binder modifiers which enhance the elastic recovery properties of a bituminous mixture whereas this can be demonstrated under repeated loading (111).

3.2.2 The Nottingham Asphalt Tester

Hitherto, the facility to carry out repeated loading tests had generally been limited to major research laboratories because of the cost and complexity of the equipment required. However, research carried out at the University of Nottingham during the 1980's into the application of pneumatic load and digital control systems led to the development of a simple, inexpensive modular test system capable of testing cylindrical specimens both in the indirect tensile mode and uniaxially, with either static or repeated loading (32,112).

In this system, known as the Nottingham Asphalt Tester (NAT), load is applied by a rolling diaphragm pneumatic actuator, the supply of compressed air to which is governed by a solenoid valve. The operation of this valve is controlled by an IBM PC compatible microcomputer via a digital to analogue converter. The load applied to a specimen and the resultant deformation are monitored by a strain-gauged load cell and LVDT's respectively. Outputs from these devices are acquired by the computer through an analogue-digital interface. The pneumatic actuator is capable of applying a load of up to 4.2kN and transient deformations as low as 1 micron can be recorded. The use of microcomputer control and data acquisition allows, through the use of suitable software, both considerable flexibility in the application of load and also the facility to acquire data continuously and automatically throughout a test.

The uniaxial testing configuration, shown in Figure 3.12, is used for assessment of permanent deformation, with the LVDT's mounted on the upper loading platen to monitor the strain which occurs over the whole length of the specimen. Huschek (113) has advocated this arrangement as it provides good "mechanical averaging" of the overall strain and avoids local effects which can affect on-sample measurements due to the heterogeneous nature of the material.

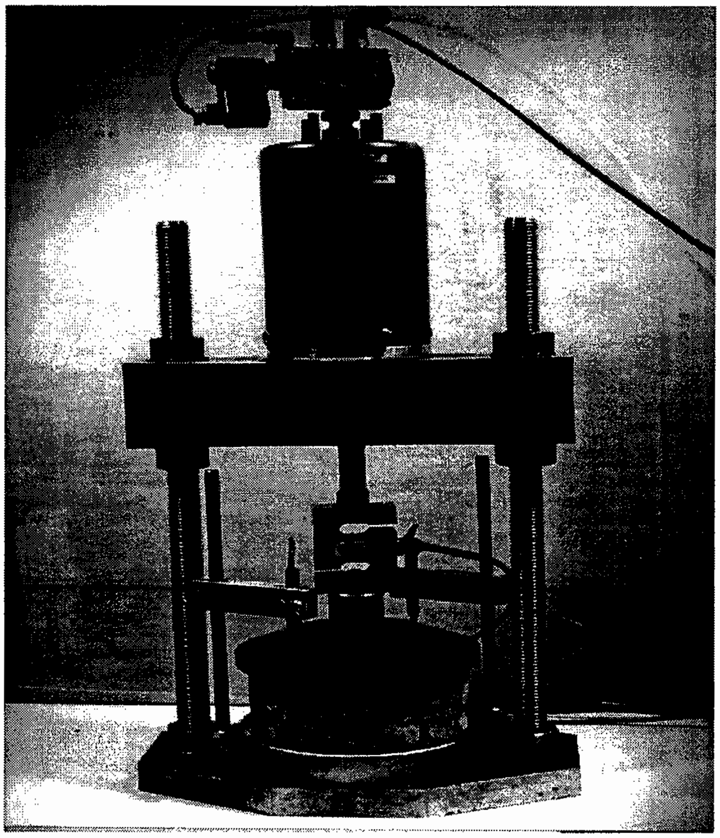


Figure 3.12 The Nottingham Asphalt Tester

3.2.3 Uniaxial Test Programme

The NAT was used to carry out a programme of unconfined uniaxial compression tests on the cores taken from the tracked slabs for the purposes of density determination (see Section 3.1.8.1). The aim of this testing, which did not form part of the research contract for UCB, was to permit a comparison of the performance of the mixtures in this form of test, under both static and repeated loading, with wheeltracking.

In the static, or creep, test a constant load was applied for a duration of one hour and the recovery of the specimen monitored for a period of 15 minutes after the removal of the load. The repeated load test, known as the repeated load axial (RLA) test, comprised 3600 cycles of a nominally square load pulse of one second duration followed by a no-load period of one second duration. Both tests were carried out at a temperature of 40°C with an applied axial stress of 100 kPa. All specimens were subjected to a pre-test conditioning regime consisting of the application of a static stress of 10 kPa for a duration of 10 minutes, the main purpose of which was to ensure that the loading platens were properly seated on the specimen prior to the commencement of deformation measurement. The results of a subsequent investigation into the effects and merits of this pre-test conditioning are presented in Chapter Six.

3.2.4 Results of Uniaxial Testing

A typical creep test data plot showing the accumulation of axial strain with increasing duration of load is presented in Figure 3.13. Figure 3.14 shows the RLA data, in terms of axial strain against number of load cycles, for the same mixture. In both cases the specimens tested covered similar ranges of void content.

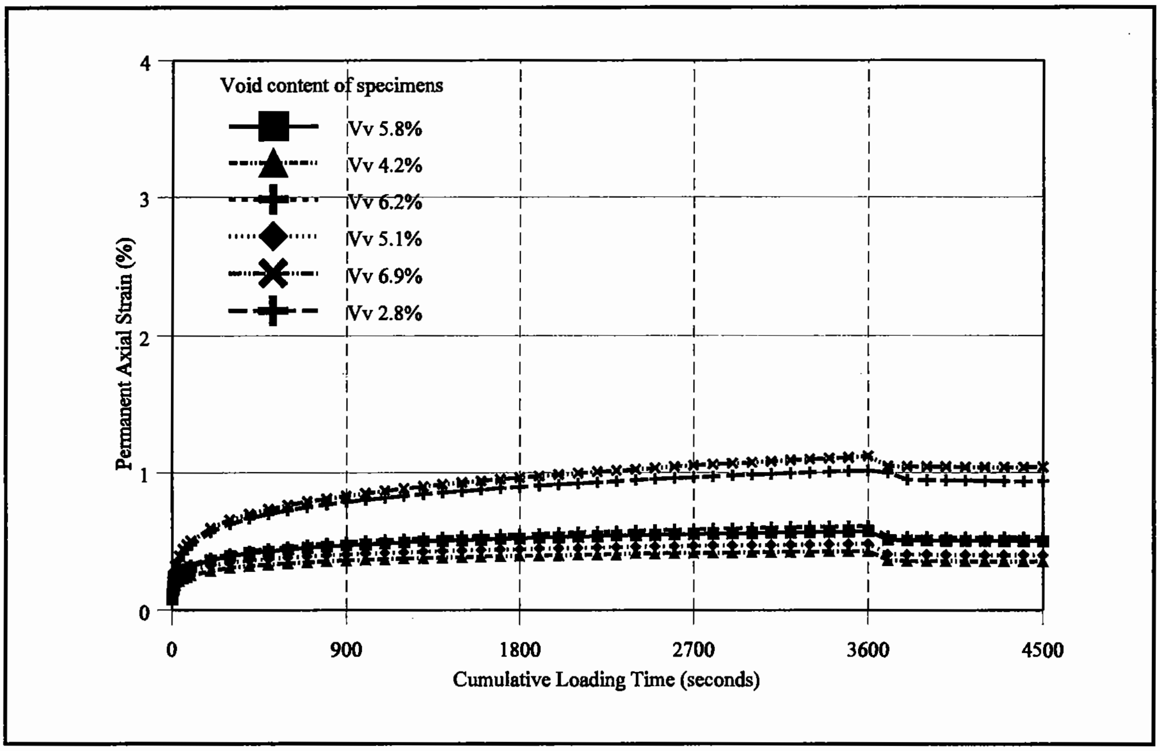


Figure 3.13 Typical Creep Test Data Plots : Mixture V1W

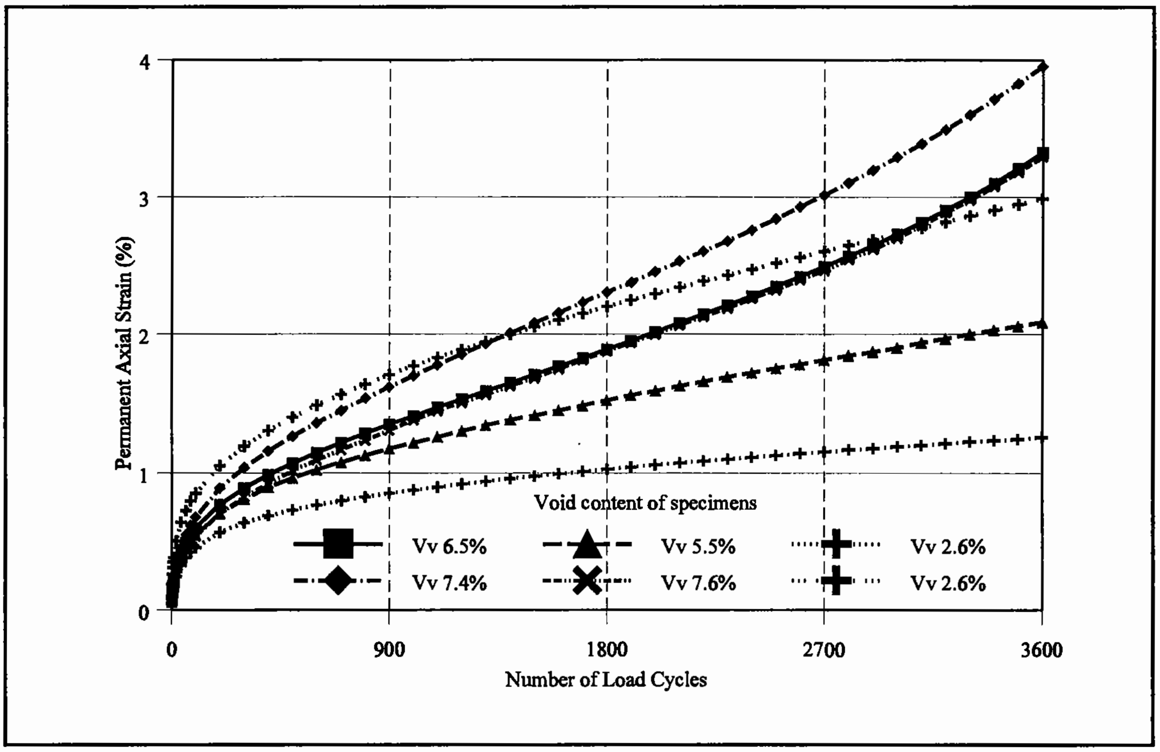


Figure 3.14 Typical RLA Test Data Plots : Mixture V1W

In order to be consistent with the analysis of the wheeltracking test data, mixture performance was quantified by determination of the rate of strain over the linear phase of the deformation response. For the majority of test results it proved satisfactory to calculate the slope of the curve by linear regression between 1800 and 3600 seconds of accumulated loading time in the case of creep data, and between 1800 and 3600 load cycles for RLA data. However, where failure of the specimen occurred during the test the linear phase generally did not pertain over this interval, as illustrated in Figure 3.15, and it was necessary to determine the position and extent of the linear portion of the curve by examination. In all cases the regression coefficient, R^2, was checked to be not less than 0.99 to ensure the linearity of the data included in the analysis. The comparison of mixture performance using this parameter is shown in Figures 3.16 and 3.17 for the creep and RLA tests respectively.

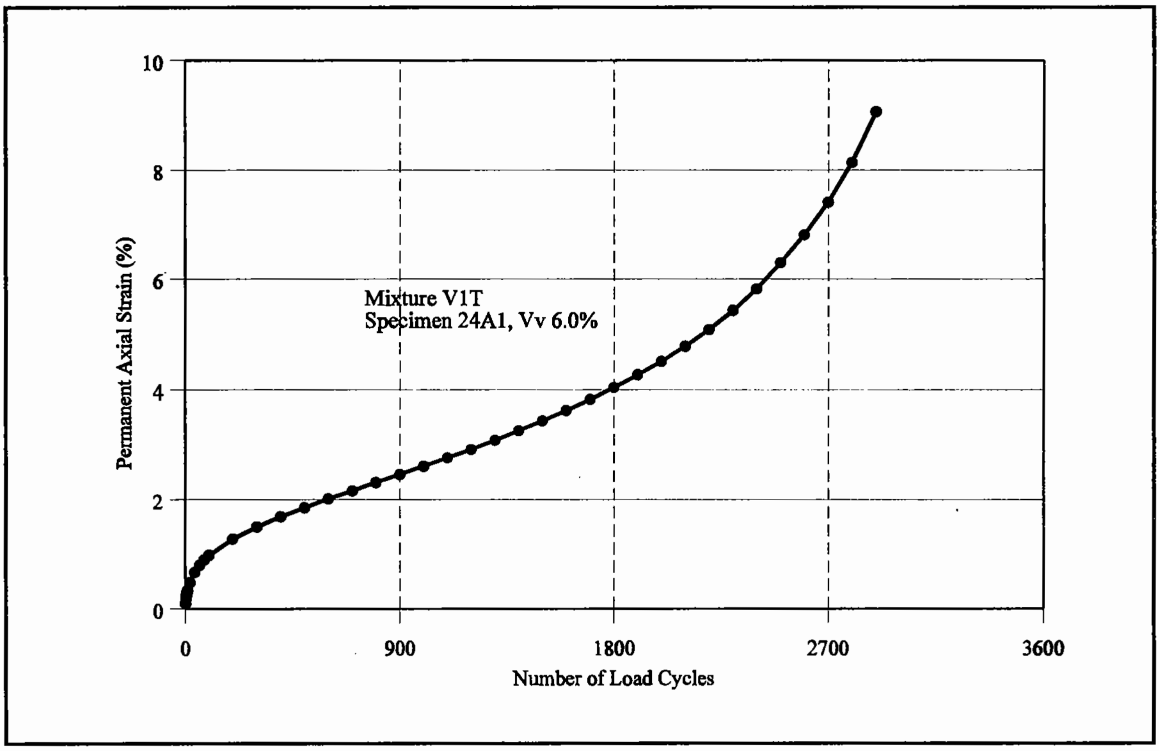


Figure 3.15 Data Plot for Specimen which Failed in the RLA Test

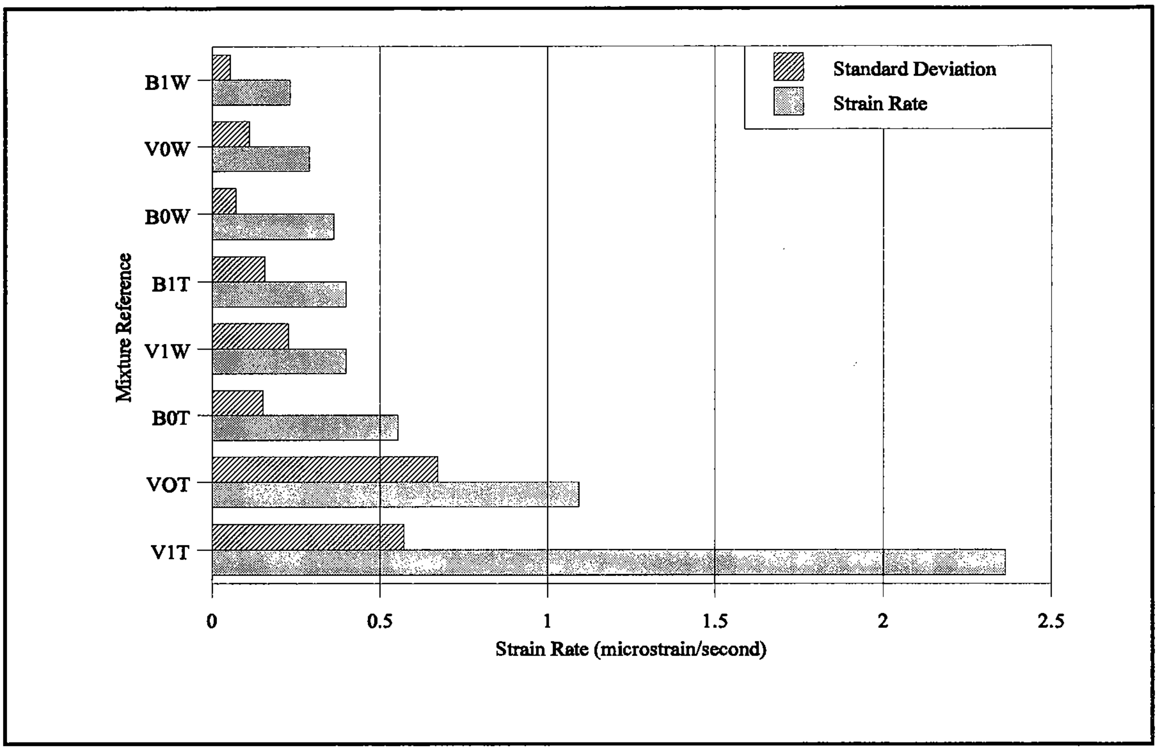


Figure 3.16 Comparison of Mixture Performance in the Creep Test

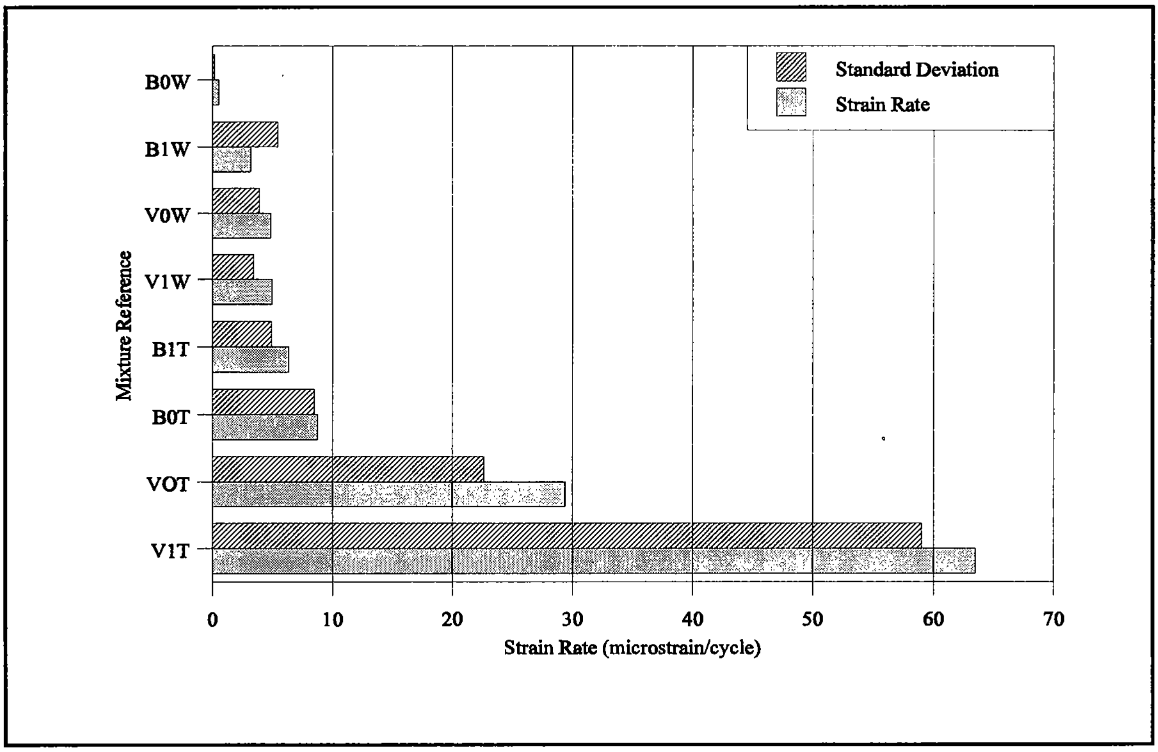


Figure 3.17 Comparison of Mixture Performance in the RLA Test

It is evident from these data that much higher strains were obtained under repeated loading than under creep, a finding which is consistent with those of other researchers (86,114,115).

The model to represent the development of permanent deformation in a bituminous mixture originally proposed by Shell (35) in conjunction with the use of the creep test attributed all permanent strain to viscous flow in the binder, with the consequence that for given temperature and stress conditions permanent strain was considered to be dependent only upon duration of loading. This is the mechanism examined by the creep test. However, as mentioned in section 3.2.1, it was found that use of the creep test underestimated deformation which occurred under simulated vehicle loading. From repeated load tests on both mixtures and dry aggregates, in which deformation was found to be strongly dependent upon the number of load applications, van de Loo (86) concluded that the purely viscous behaviour model was incomplete as it did not account for the effect of dynamic loading on aggregate contacts. In consequence Shell introduced correction factors for these dynamic effects into their creep analysis.

It would appear, therefore, that the two forms of loading emphasise different mechanisms of resistance to permanent deformation. Under static loading deformation may occur through viscous flow in the binder up to the point where sufficient strain is mobilised for the aggregate structure to "lock-up", whereas with repeated loading permanent deformation may continue to accumulate as instantaneous plastic strains are induced in the aggregate skeleton (110,116). This difference in response is clearly evident in Figures 3.13 and 3.14.

It is likely that the mobilisation of strains in the aggregate structure, which is random and will differ from specimen to specimen, in the repeated load test has contributed to the degree of scatter in the data which is rather higher than for the static test, as shown in Figures 3.16 and 3.17. The apparent poor repeatability is, however, compensated to an extent by the high magnitude of strain developed under repeated loading which should make the test more discriminating.

3.3 Comparison of Wheeltracking and Uniaxial Test Results

The rankings of the mixtures by performance, on the basis of strain rate, in the wheeltracking, creep and RLA tests are shown in Figure 3.18.

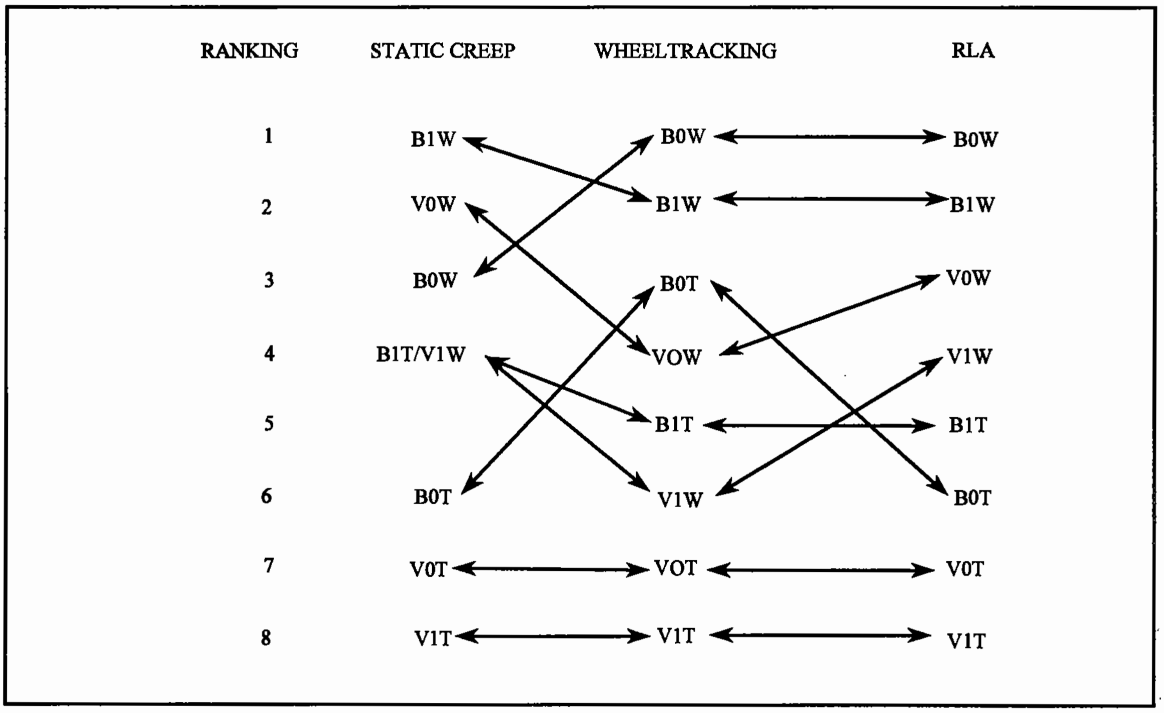


Figure 3.18 Ranking of Mixture Performance by Test Method

The ranking obtained from the RLA test appears sensible and consistent, with all the granite mixtures placed higher than the gravel mixtures, and within each aggregate type the less temperature susceptible Boscan binder ranked ahead of the Valley. Correspondence between the RLA and wheeltracking rankings is reasonable, though agreement between either test and the creep test is less clear. These findings are in line with those of Hopman et al (102) who failed to find any direct correlation between static creep and wheeltracking, while Valkering et al (111) have reported a correlation between a similar form of repeated load test and wheeltracking, though this was based on limited data.

3.4 Merits of the Test Methods

The appeal of wheeltracking tests due to the simulative nature of the loading has already been mentioned, however it has been suggested that for such tests to be truly representative the specimen would have to be large in relation to the size of the wheel (91,102). The manufacture of very large test specimens would obviously be a disadvantage for routine use because of the equipment and extra effort required for mixing and compaction, and also the dimensions of the test equipment. Smaller scale wheeltracking tests, while being simple in principle, may not be ideal in terms of simulation of field conditions. Specimen preparation is also a significant limitation for small scale wheeltracking tests as the specimens are still larger than those generally used in uniaxial tests, and as slabs are usually tested the choice of compaction equipment is limited. The rate of testing achievable, which is an important factor in the success of a test (23), is governed by the specimen preparation as well as the test procedures. Grimaux and Hiernaux (97) reported that with the LCPC test, in which two slabs may be tested simultaneously, it is possible to manufacture and test 4 slabs in a week, though with night work the rate can be increased to 2 slabs in 2 days. The testing time for a pair of slabs is about 9 hours (100).

While wheeltracking tests are simple in principle the equipment may be sophisticated and expensive. The cost of the LCPC wheeltracker and the necessary roller compactor has recently been put at $US 200 000, though by comparison the Georgia loaded wheel tester capable of testing three specimens simultaneously appears cheap at $US 35 000, including the cost of a compactor (103). The cost of a production version of the wheeltracking equipment used in this investigation would be approximately £11 000 ($US 17 500), including the control and acquisition system.

Uniaxial tests are not directly simulative of the field situation and are intended primarily as index tests for the ranking or classification of performance. However, they have the advantage over wheeltracking tests that results are produced in terms of fundamental properties. There is also greater scope for specimen preparation as specimens may be cored or moulded, and preparation and test procedures are acknowledged to be simpler than for wheeltracking (103).

The limitations of the static test in being unable to produce plastic strains in the aggregate structure (86,117), and the importance of these strains in the development of permanent deformation are well recognised (110,116) and have been demonstrated in the results presented above, as has the ability of the repeated load test to rank materials effectively.

The facility to perform repeated loading tests simply and at low cost provided by the development of the NAT - which currently costs around £20 000 ($US 32 000) and is capable of performing other tests - makes this form of test a prime candidate for use in mixture design and also, potentially, performance specification.

CHAPTER FOUR

VALIDATION OF THE REPEATED LOAD AXIAL TEST

4.1 Introduction

The correspondence between the TRL wheeltracking test and the RLA test demonstrated in the preceding Chapter lends support to the use of the latter in mixture design, where the primary aim is to compare and rank the performance of different mixtures. However, the opportunity to make a potentially more robust assessment of this test was provided by an extension to the SHRP contract research for UCB, which was commissioned with the aim of providing data on the deformation of bituminous mixtures for use in the validation of tests being considered for the Superpave system. This additional work for UCB entailed the construction and testing of a trial pavement in the University's Pavement Test Facility (PTF), which is a medium scale wheeltracking facility. On completion, cores and slabs were cut from the pavement and held in cold storage until time and funding were available under the LINK Bitutest project to make use of these specimens in a programme designed to provide a comparison between the results of both the RLA and TRL wheeltracking tests and the data obtained from the PTF.

The following sections describe the construction, instrumentation and testing of the trial pavement and present the results of this testing together with those from the subsequent wheeltracking and RLA tests. Additional data from the construction and testing of the trial pavement are included in Appendix B to this thesis.

4.2 Selection of Materials for the Trial Pavement

It was elected to test four bituminous mixtures since this was the number of separate sections which could reasonably be accommodated in a single installation in the PTF. As for the programme for the evaluation of the wheeltracking test described in Chapter Three, the mixtures were made up from the combinations of two aggregates and two binders, with both the aggregates and the binders being selected to give a range in expected performance. However, because the quantities of hot-mixed material required for the construction of the test sections were larger than could be prepared in the University laboratories, it was not feasible to use materials supplied from the SHRP MRL. The mixtures had, therefore, to be obtained from commercial suppliers and this restricted the range of materials available for use.

Since the principal objective of the exercise was to provide data from simulative testing for the validation of other test systems by UCB, the bituminous material was placed and tested on a typical flexible pavement foundation.

4.2.1 Aggregate for Bituminous Mixtures

A local granite quarry was chosen as one source of bituminous material, with the expectation that its crushed aggregate would give good resistance to permanent deformation. The aggregate for the material intended to give poorer performance was obtained from a supplier prepared to provide a flint gravel mixture with aggregate fractions sourced from three separate pits. It was considered prudent to use a standard aggregate grading rather than a "one-off" formulation which could be difficult to achieve, particularly since the materials were to be obtained from two separate suppliers. A 20mm DBM basecourse grading conforming to BS 4987(21) was used, therefore, this being similar to the asphaltic concrete grading used in the evaluation of the wheeltracking test.

4.2.2 Bitumen

Obtaining conventional bitumens with widely differing properties proved difficult, since paving grade binders in the UK all conform to a common specification (118). Although different grades of bitumen are readily available the requirements of the specification are such that the permissible variation in temperature susceptibility is small, even between grades.

Therefore, in order to achieve a more radical difference in properties than could be obtained by simply using different grades of binder, a conventional 100 penetration grade bitumen was used and altered in two of the four mixtures by the addition of a polymer modifier. The modifier chosen was a styrene-butadiene-rubber (SBR) latex that had been assessed in earlier work at the University (119,120) and which it was known would change the temperature susceptibility of the binder. Trials in South Africa (121) have also shown that SBR modification can reduce the temperature susceptibility and improve the deformation resistance of bituminous materials. Furthermore, this modifier is easy to use as it requires no special blending and may simply be added to the mixing box at the batching plant.

The modifier was added at a concentration of 7% by mass of binder which, being slightly higher than that used commercially, was intended to ensure adequate distribution throughout the material. A target binder content of 5.0% by mass of mixture was selected. This is the value specified in BS 4987 for the chosen basecourse grading with gravel aggregate, but marginally higher than the target value of 4.7% for use with crushed rock, though still within the tolerance range of ± 0.6%. Although the bituminous materials were obtained from two separate plants, both used bitumen from the same supplier.

4.2.3 Sub-base

The granular sub-base was obtained from the quarry which was to supply the crushed rock DBM basecourse. The material chosen conformed to the specification for Type 1 (24), which is that normally used in flexible construction on trunk road and motorway in the UK.

4.2.4 Subgrade

The existing subgrade used for previous experiments in the PTF was retained. This material is Keuper Marl, a local silty clay, which had originally been obtained in the form of unfired bricks.

4.3 The Pavement Test Facility

4.3.1 The Loading System

The PTF is a medium scale testing facility in which trial pavements can be constructed and subjected to simulated vehicle loading under controlled conditions. The loading system is a linear tracking, servo-controlled, hydraulic apparatus which is shown diagrammatically in Figure 4.1. This system was designed to be capable of applying a load of up to 10 kN through a driven wheel at a speed of up to 14 km/hr (122). Either unidirectional or bi-directional tracking can be employed and the loading frame also has a traversing facility to simulate lateral wander. The loading wheel is fitted with a treaded pneumatic tyre which can be inflated to give a maximum contact pressure of around 650 kPa. Pavement temperature can be cycled or maintained constant anywhere with the range 15°C to 30°C.

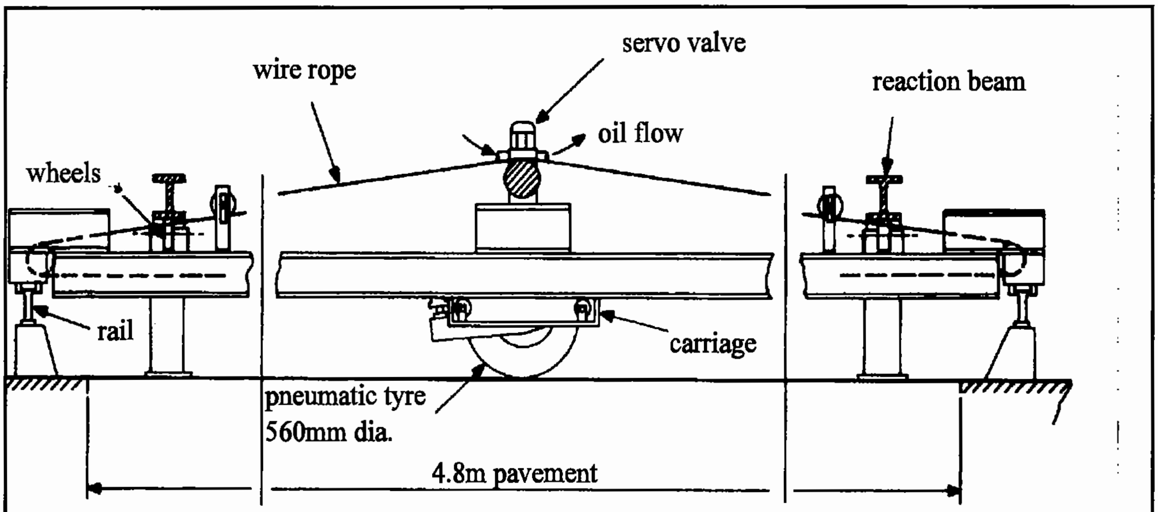


Figure 4.1 Pavement Test Facility Loading System

4.3.2 Pavement Structure

Trial pavement sections are constructed within a concrete pit which measures approximately 4.8m in length, 2.4m in width and 1.3m in depth. Since construction and instrumentation of trial pavements is both an expensive and lengthy process, it was not feasible to carry out the whole operation separately for each of the bituminous mixtures to be tested. The four mixtures were, therefore, laid in the four quadrants of the pit in sections with plan dimensions of 2.4m x 1.2m. This approach had the advantage of requiring the pavement foundation to be constructed only once, which increased the probability of achieving a uniform supporting structure beneath each of the bituminous mixtures. The layout of the trial pavement is represented schematically in Figure 4.2, which also indicates the reference codes adopted for each section.

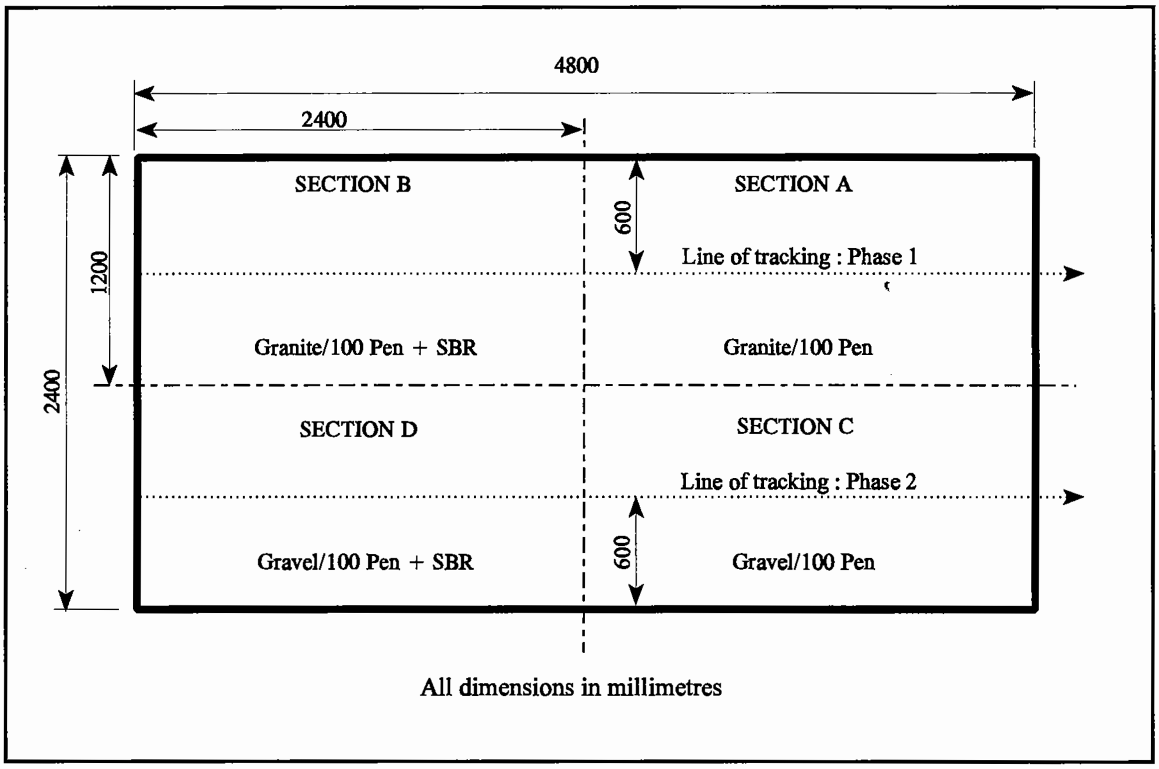


Figure 4.2 Plan Layout of Trial Pavement Sections

The nominal pavement structure consisted of 100mm bituminous surfacing over 200mm of granular sub-base placed and compacted on the silty clay subgrade. The thickness of 100mm for the bituminous layer was chosen to allow the 20mm nominal size material to be placed in two lifts, which was necessary for the installation of instrumentation within the bituminous material. This thickness of surfacing would also guard against failure through fatigue cracking, essential in a deformation test, as well as providing adequate depth of material to accommodate deep ruts.

4.3.3 Test Conditions

The conditions pertaining throughout testing were as detailed below:

Pavement temperature	:	30°C
Wheel load	:	10 kN
Wheel speed	:	4 km/hr
Tyre pressure	:	620 kPa
Mode of loading	:	Unidirectional
Lateral wander	:	Nil

Unidirectional loading was chosen as being most representative of actual trafficking, though lateral wander was not permitted in order to simplify the interpretation of results. Tracking was conducted in two phases, with two sections in line along the length of the pavement tested in each phase. The lines and direction of tracking are indicated on Figure 4.2.

The testing was carried out at a temperature of 30°C, the maximum which could be maintained, to accelerate the accumulation of permanent deformation.

4.4 Instrumentation

4.4.1 Objective

The purpose of this test programme was to evaluate the development of permanent deformation in four bituminous mixtures subjected to identical environmental conditions and loading regimes. However, since the mixtures were laid on a flexible pavement foundation it was possible that any rutting apparent at the surface may in part be due to permanent deformation in the pavement foundation, ie structural rutting. Simple measurements of surface rutting could not, therefore, be solely relied upon as the basis for comparing the performance of the different mixtures. For this reason, instrumentation was installed through the depth of the pavement in order to monitor the deformations both within and below the bituminous material.

4.4.2 Strain Measurement

Inductance strain coils were used to monitor the development of permanent strain throughout the pavement structure. These coils comprise copper windings around discs of insulating material. The coils operate in pairs, usually in either coplanar or coaxial arrangement, and are electromagnetically coupled so that, with careful calibration, their exact separation can be determined. Considerable experience has been gained in the installation and use of these coils in numerous experiments in the PTF (122). They have been found to be robust, stable and reliable and the absence of any mechanical linkage between the coils minimises restraint on the material over the gauge length where strains are measured.

4.4.3 Arrangement of Instrumentation

The majority of the coils, all 25mm in diameter, were placed coaxially in vertical stacks through the depth of the structure. However, coils were also placed in coplanar arrangement at mid-depth in the bituminous layer to monitor strains both in, and normal to the direction of trafficking. Two stacks of coils were placed on the line of trafficking in each of the sections. The arrangement of coils in each stack is shown in Figure 4.3.

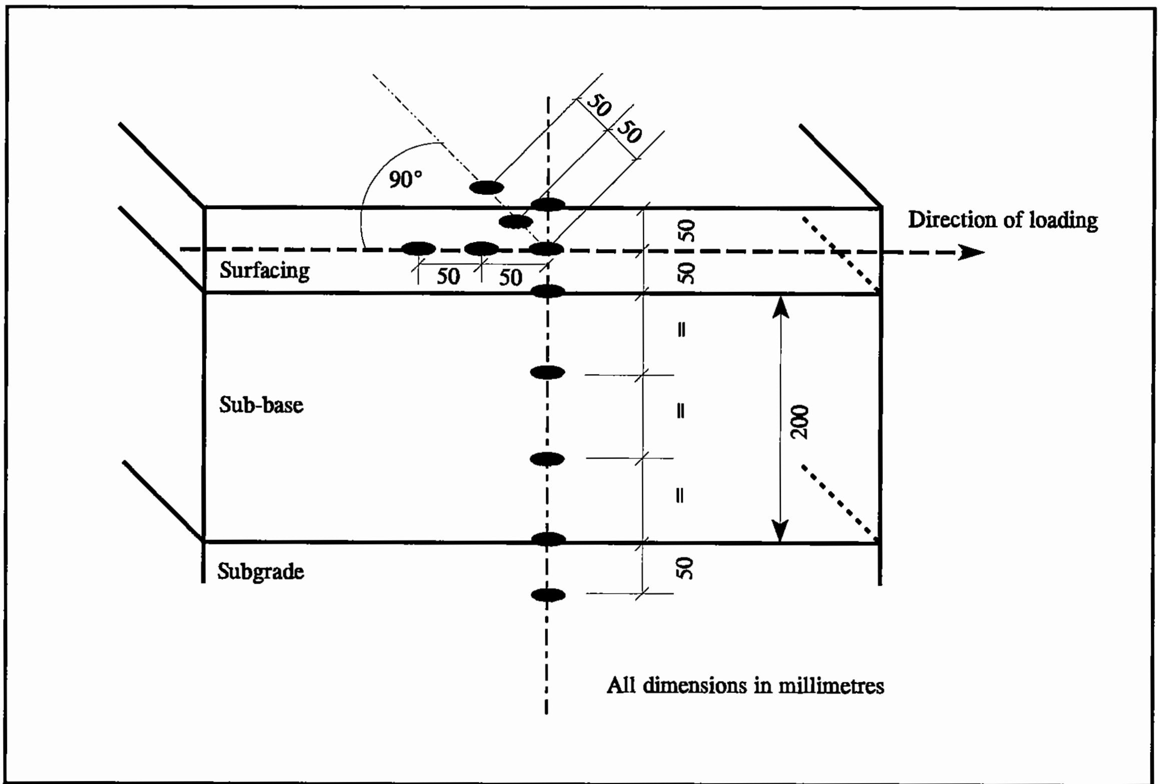


Figure 4.3 Configuration of Strain Coil Stack

In one of the stacks in each section, the uppermost coil was recessed into the pavement surface and fixed in position with an epoxy adhesive. The top coil of the other stack was not embedded in the construction but was positioned on the surface of the pavement only while trafficking was suspended in order to take measurements of permanent strain.

4.5 Construction of the Pavement Foundation

The principal objective in the construction of the pavement foundation was to produce as stiff a structure as possible in order both to minimise the accumulation of permanent deformation within the foundation itself and to prevent fatigue failure in the overlying bituminous material. Therefore, after trimming to level and applying light compaction, the clay subgrade was allowed to dry out to a moisture content of 11 - 12% at the surface, though care was taken to prevent fissuring of the material. In situ CBR tests gave values ranging from 14% to 26%, which would indicate a strong subgrade.

In addition to being stiff the foundation had also to be uniform if a fair and reliable comparison of the performance of the different bituminous mixtures was to be made. In order to gauge the uniformity of the subgrade testing was carried out with a Clegg Hammer (123), which is a simple impact device developed primarily for use on unbound granular bases, since this provided a rapid means of assessment. The results indicated little variation in response between the four sections of the trial pavement.

The granular sub-base was placed in two lifts, each approximately 100mm thick after compaction with a vibrating pedestrian roller. Attempts to measure the density of the compacted sub-base using a nuclear density meter (NDM) in backscatter mode proved unsuccessful due to the rugosity of the surface. Determinations of moisture content made with the same instrument were similarly unreliable. The Clegg Hammer was again used, on the final compacted surface, to assess uniformity and this showed relatively low variation between the four sections.

The data from the testing carried out on the subgrade and sub-base, which overall indicated a stiff and uniform foundation, are presented in Appendix B.

The finished thickness of the sub-base was assessed by taking spot heights on the formation prior to placing and then on the surface of the sub-base after final compaction. The thicknesses determined on the spot grid are presented on Figure 4.4. This shows that 78% of the values were within 6mm of the target thickness of 200mm and 95% were within the surface level tolerance range of +10mm / -30mm permitted in the DOT's specification governing principal road construction (24). The only points outside this range were along the extreme edge of the PTF in sections A and C (Figure 4.4), beyond the limit of travel of the loading wheel.

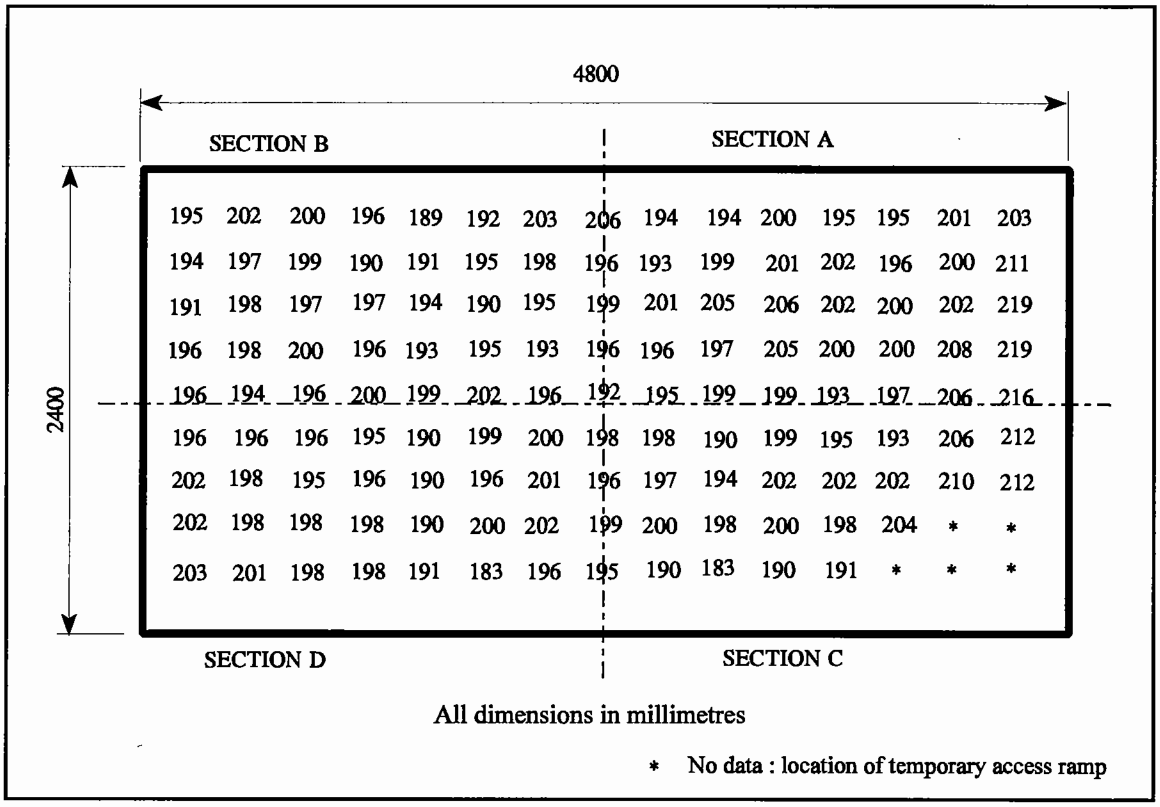


Figure 4.4 Sub-base Thicknesses

4.6 Production and Placing of the Bituminous Surfacing

4.6.1 Mixing

Approximately two tonnes of basecourse were required for each of the four trial pavement sections, and this material was produced in batch mix plants at the source of supply. The two batches from each source, one modified and one unmodified, were obtained on consecutive days to minimise variation in the material. When producing the SBR modified mixture, the required amount of SBR was calculated once the batch mass of both aggregate and binder had been recorded at the weigh hopper. The SBR latex was then added at the top of the mixing box, approximately thirty seconds after the start of mixing. Total mixing time was approximately two minutes, at a temperature of 150°C.

4.6.2 Transportation

The mixed material was discharged direct from the batch plant into a hot-mix storage hopper, which was mounted on a rigid bed truck. The storage hopper was heated by thermostatically controlled propane gas burners capable of maintaining the mixture temperature for several hours. This was essential since one of the quarries was almost two hours' journey time from the University and the placing and compaction procedures in the PTF were slow due to use of hand laying techniques and installation of instrumentation.

4.6.3 Placing, Compaction and Instrumentation

On arrival at the PTF, the bituminous material was discharged from the storage hopper, by auger, into barrows. Strain coils on the surface of the sub-base were fixed in position with a small amount of fine bituminous material before the first layer was spread and levelled by rake. A vibrating pedestrian roller was used to compact the first lift to a thickness of approximately 50mm. Strain coils were then placed on top of the first lift and again fixed in position with fine material, before the second lift was placed and compacted.

The number of roller passes on the materials was not standardised since it was recognised that each mixture would probably require a different compactive effort to achieve the same density. On the first section which was placed a NDM was used in an attempt to determine whether consistent compaction had been achieved. However, attempts at further compaction after taking the NDM readings proved ineffective so, for the remaining three sections, the material was merely rolled until a satisfactory finish had been achieved. All the bituminous material was placed at a temperature of approximately 150°C.

The thicknesses of surfacing determined on the spot grid are shown on Figure 4.5. The mean thicknesses for sections A, B, C and D were found to be 102, 95, 103 and 98mm respectively. These values were all within acceptable range of the target thickness of 100mm, which was a nominal value selected for the reasons stated in Section 4.3.2.

There was some variation in individual measurements both within and between sections, which exceeded the permitted tolerance for the surface level of basecourse given in the DOT specification (24). This reflects the level control achievable with hand laying of the material. This variation would not, however, adversely affect the validity of the experiment since the rutting propensity of a bituminous mixture is dependent upon its composition rather than its thickness.

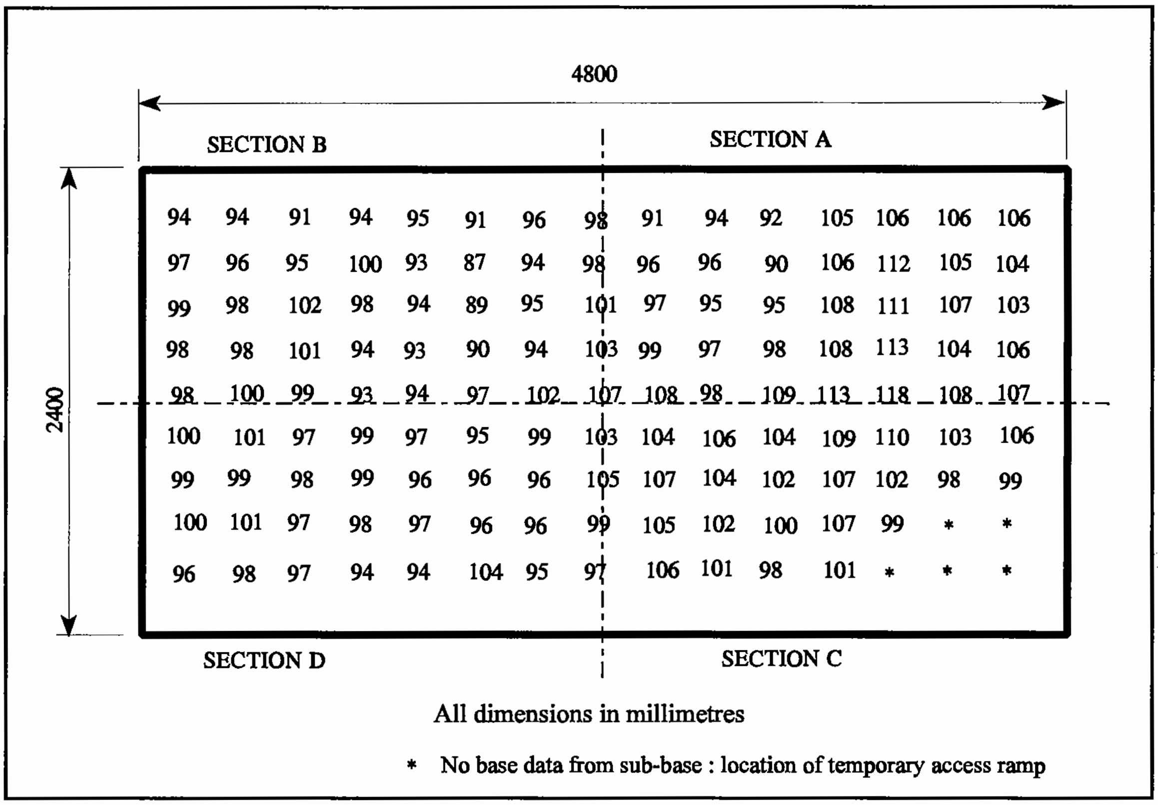


Figure 4.5 Thicknesses of Bituminous Surfacing

4.6.4 Sampling and Testing of Bituminous Material

Samples of the loose material were obtained from each of the four sections at the time of placing, and submitted to a commercial laboratory for analysis to determine aggregate grading and binder content. The resulting gradings are presented in Figure 4.6 which shows that the conventional gravel mixture was fine of the target grading, while the other three mixtures all lay inside the specification envelope. The binder contents are presented in Table 4.1, though the values quoted for the mixtures containing the SBR modifier may be unreliable as difficulty in extracting the binder was reported for these materials.

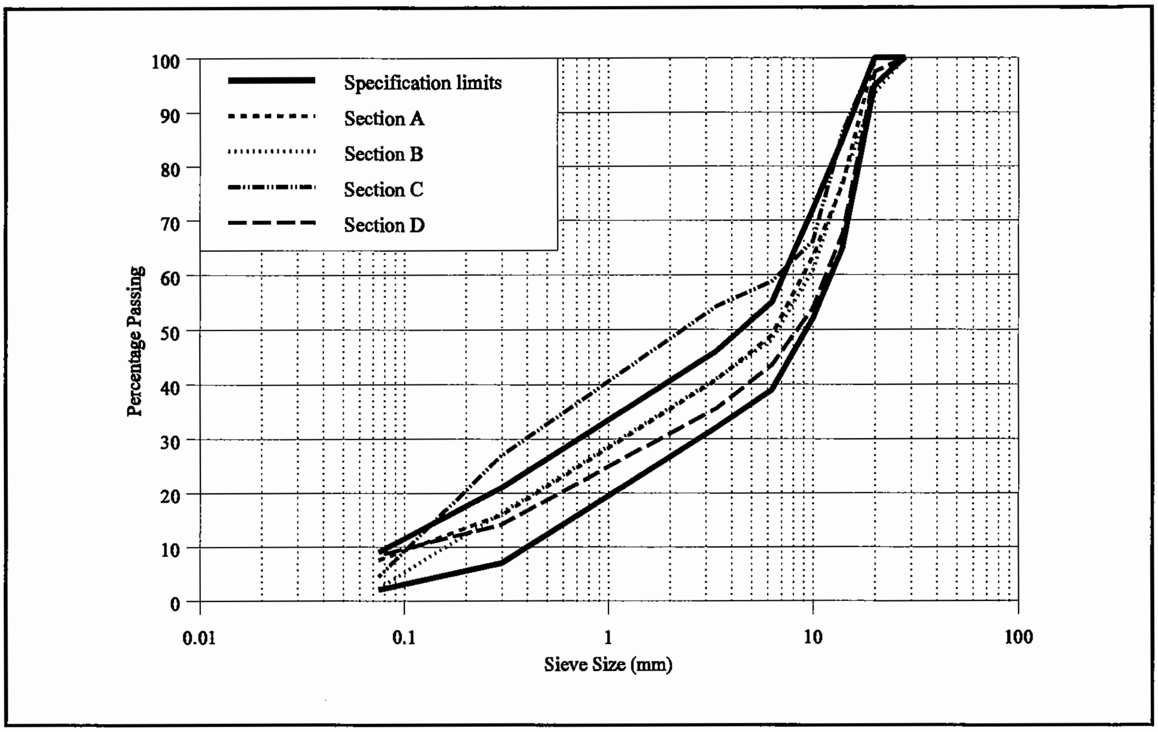


Figure 4.6 Aggregate Gradings Determined from Mixture Analysis

Table 4.1 Binder Contents Determined from Mixture Analysis

Section Reference	Mixture (aggregate/binder)	Binder content by mass of mixture (%)
A	Granite/100 Pen	4.5
B	Granite/100 Pen + SBR	4.9
C	Gravel/100 Pen	5.2
D	Gravel/100 Pen + SBR	4.5

4.7 Testing of the Trial Pavement

4.7.1 Test Sequence

The pavement was constructed with the sections comprising the bituminous mixtures with aggregate from the same source in line along the length of the pit (Figure 4.2) to permit simultaneous testing of these materials. The trafficking of the two granite sections was completed before the loading frame was repositioned for testing of the two gravel sections. A total of 50 000 wheel passes was applied in each case.

4.7.2 Data Collection

The transverse topographic profile was recorded periodically throughout the test at three locations within each section, using the profilometer shown in Figure 4.7. This device consists of a small diameter wheel mounted on a bogey which is tracked across the pavement surface. Vertical movement of the wheel is monitored by an LVDT, the output from which was directed through an X-Y plotter to produce a graphical trace of the surface profile.

Trafficking was suspended at intervals, which were determined by the observed development of deformation, to permit profilometer traces to be obtained and permanent strains within the pavement structure to be evaluated using the inductance strain coils.

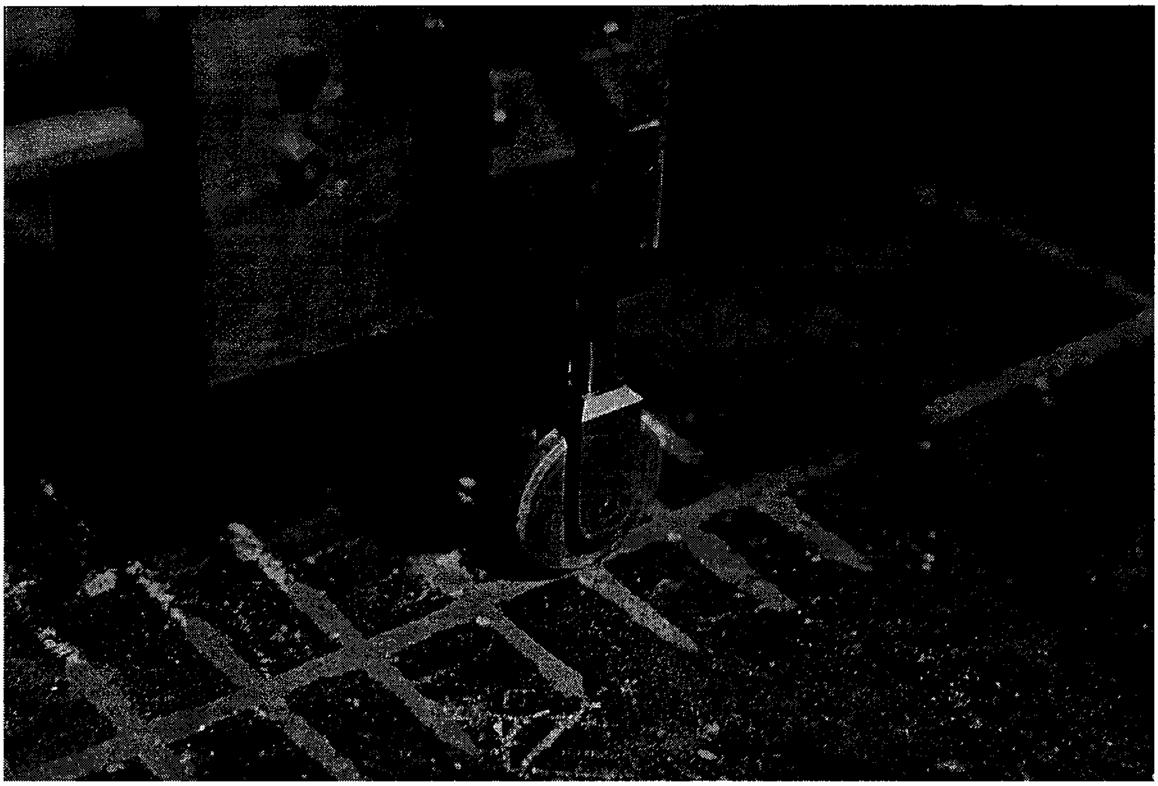


Figure 4.7 Profilometer used for the Recording of Rut Profiles

4.8 Results from Testing of the Trial Pavement

4.8.1 Strain Profiles

Figures 4.8 to 4.11 show the development and distribution of permanent vertical strain throughout the pavement structure, as determined by one of the two inductance strain coil stacks in each section. Similar data from the other coil stacks are presented in Appendix B. Compressive (negative) strains recorded in the upper sub-base (see Figure 4.3) after 50 000 passes in each of the sections were consistently in the range 2.0 to 2.5%, which would represent a contribution of approximately 1.5mm to the apparent surface deformation. Strains in the mid and lower sub-base and subgrade were, in all cases, very small and constant indicating little or no permanent deformation in the lower foundation. It would seem reasonable, therefore, to compare performance of the bituminous material on the basis of measured surface ruts, since the contribution to rutting from the pavement foundation was both small and similar in each section.

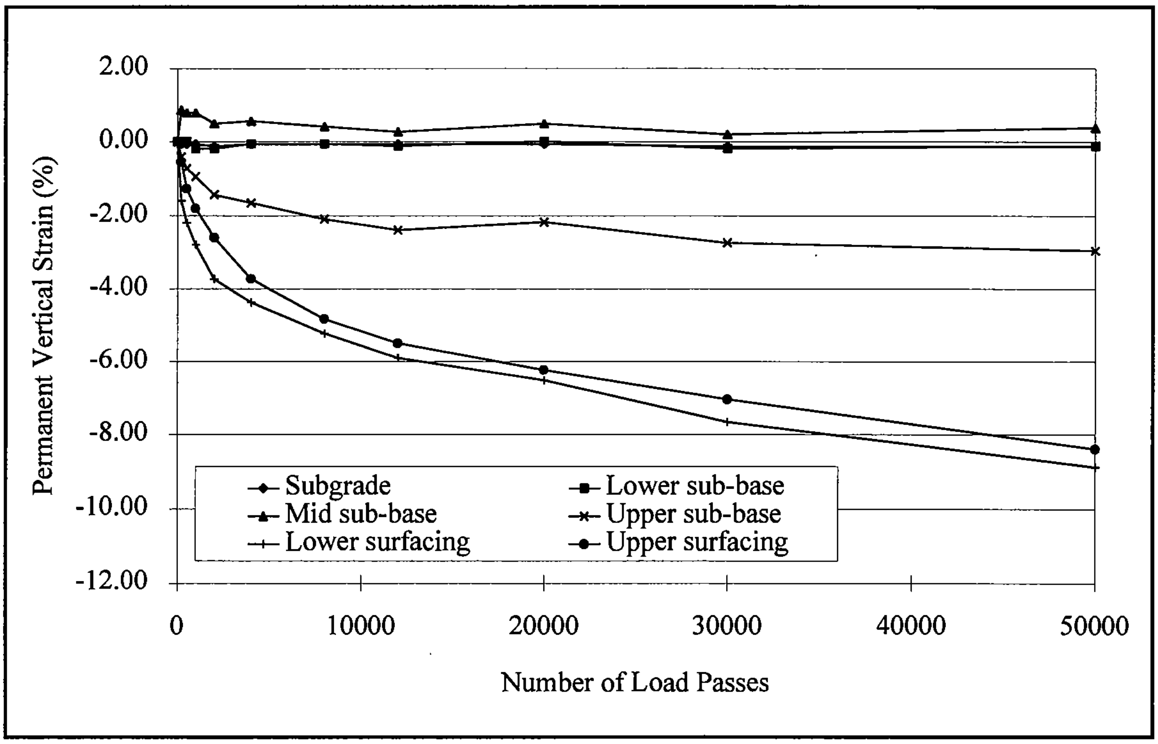


Figure 4.8 Development and Distribution of Permanent Vertical Strain : Section A Coil Stack 4

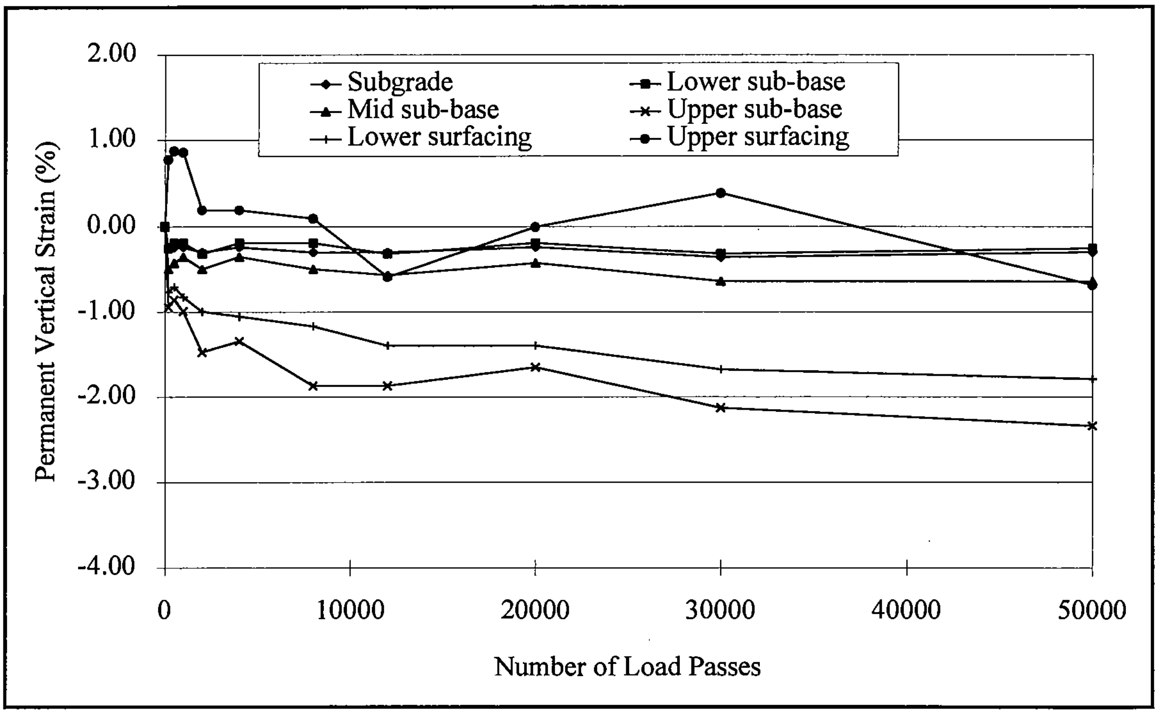


Figure 4.9 Development and Distribution of Permanent Vertical Strain : Section B Coil Stack 1

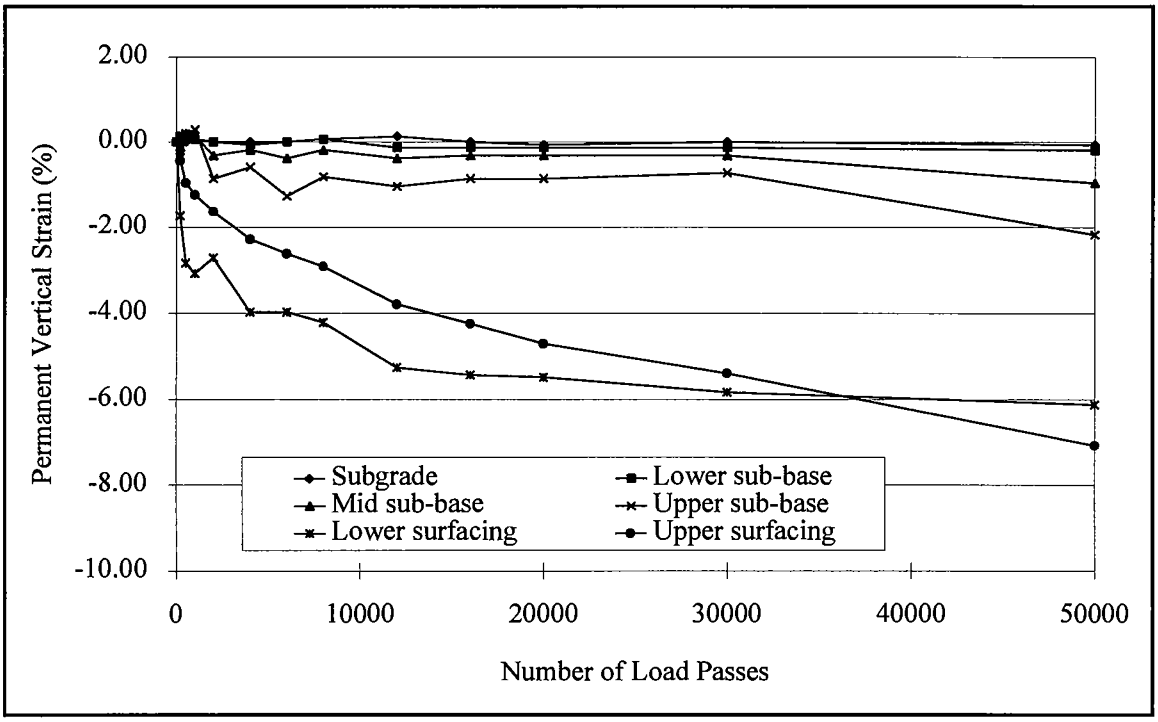


Figure 4.10 Development and Distribution of Permanent Vertical Strain : Section C Coil Stack 8

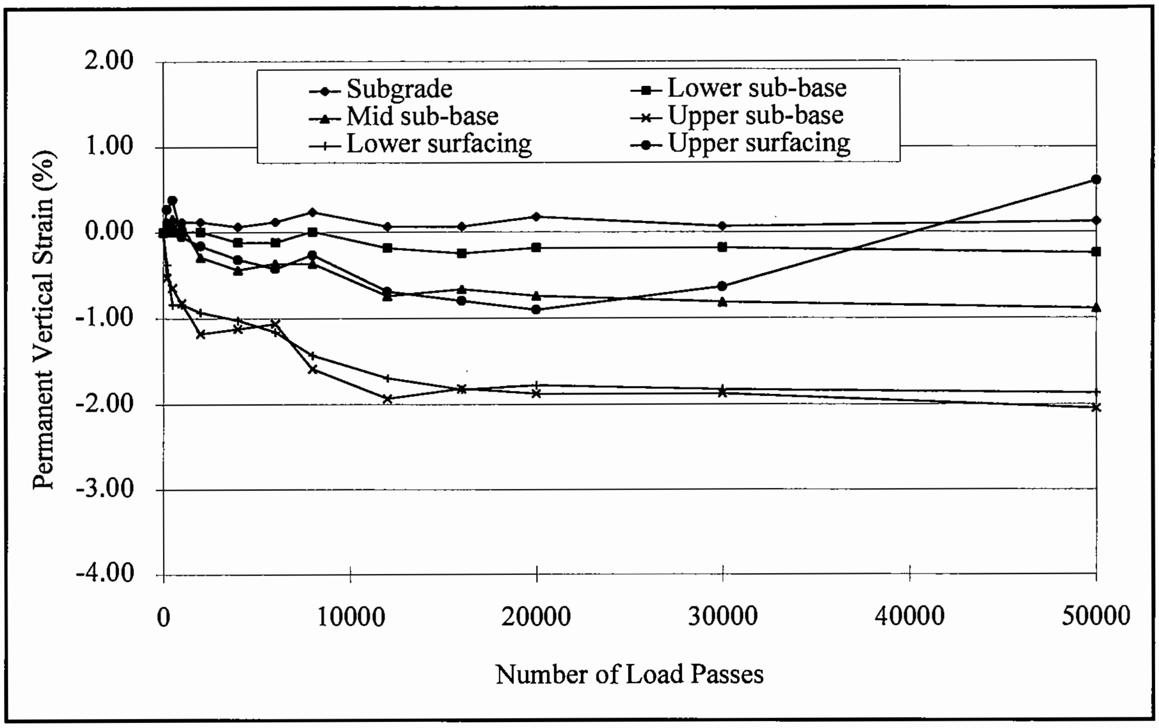


Figure 4.11 Development and Distribution of Permanent Vertical Strain : Section D Coil Stack 5

The development of both vertical and lateral permanent strains within the bituminous material was also monitored with the strain coils. A plot showing the accumulation and distribution of strain in the surfacing in section A is presented in Figure 4.12. It can be seen that the lateral strains were tensile and of the same order as the compressive vertical strains, indicating significant outward displacement of the material. Figure 4.13 shows a slice saw-cut from the pavement after testing at the location of the coil stack in section A. The positions of the coils, painted white for clarification, which were originally placed in coplanar arrangement at mid-depth and normal to the direction of trafficking, confirm and demonstrate the extent of the upward and outward displacement of the material.

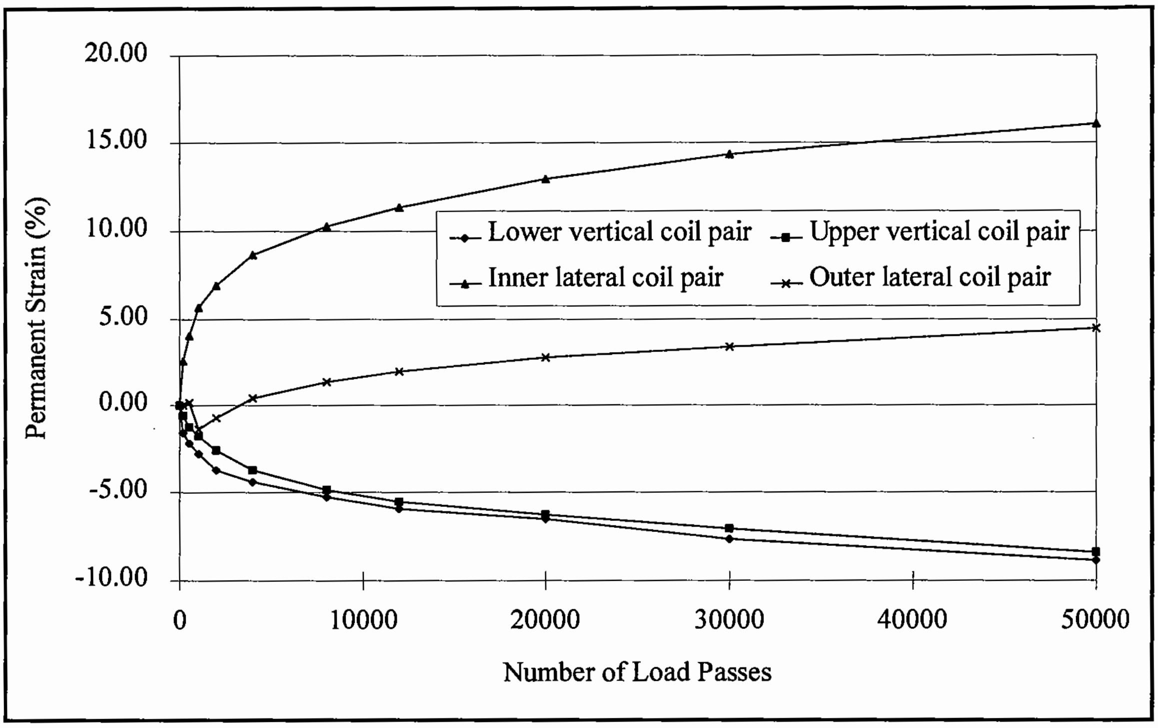


Figure 4.12 Development and Distribution of Strain within the Bituminous Surfacing Section A, Coil Stack 4

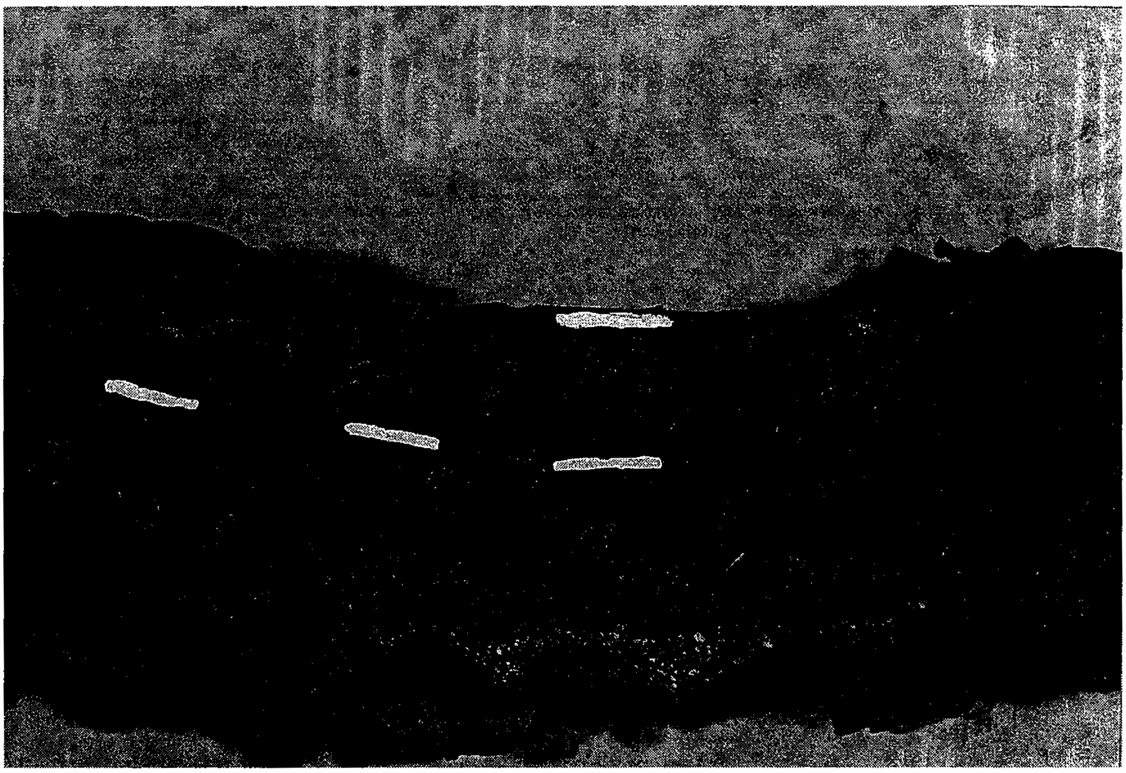


Figure 4.13 Slice Cut at Coil Stack Location in Section A after Testing

The profiles of permanent strain within the bituminous material for the other sections, which are included in Appendix B, generally showed a similarity in magnitude between vertical and lateral strains. The vertical compressive strain was, however, occasionally larger and its distribution between the upper and lower parts of the bituminous layer was not consistent, possibly indicating the occurrence of a degree of densification. Also, it should be noted that displacements of the coils which cause the coaxial or coplanar arrangement to be significantly disturbed, as in Figure 4.13, may render the calibration of the pair, and hence the determination of strain, invalid.

4.8.2 Surface Rut Profiles

A topographic rut profile, obtained using the profilometer, showing the development of permanent deformation with increasing number of load passes in section A is presented in Figure 4.14. In order to facilitate analysis and interpretation of this data the three surface profiles from each section were digitised to translate the graphical trace from the profilometer into data in the form of X-Y co-ordinates. This enabled the initial trace to be established as the datum and the subsequent development of the ruts to be charted and measured simply by subtraction of successive traces. The effect of this treatment of the data is illustrated in Figure 4.15 which is the digitised form of the topographic profile shown in Figure 4.14. Figures 4.16 to 4.18 show one digitised profile from each of sections B, C and D and the remaining profiles for all four sections are included in Appendix B.

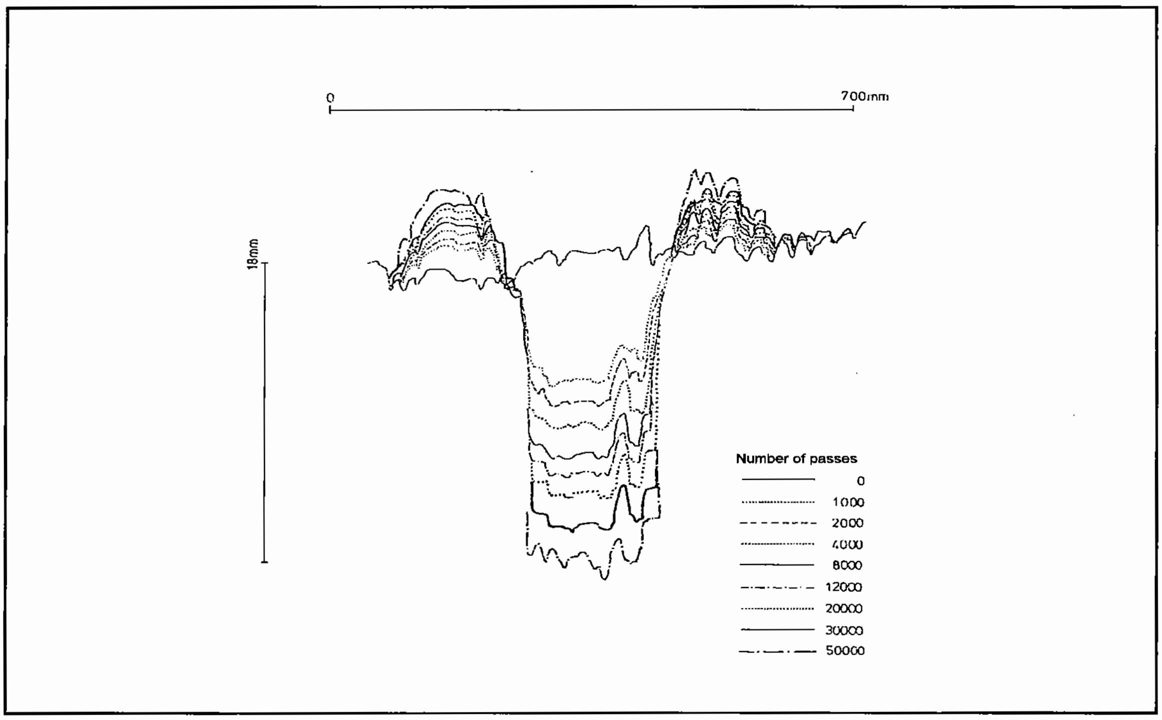


Figure 4.14 Topographic Rut Profile : Section A, Profile 2

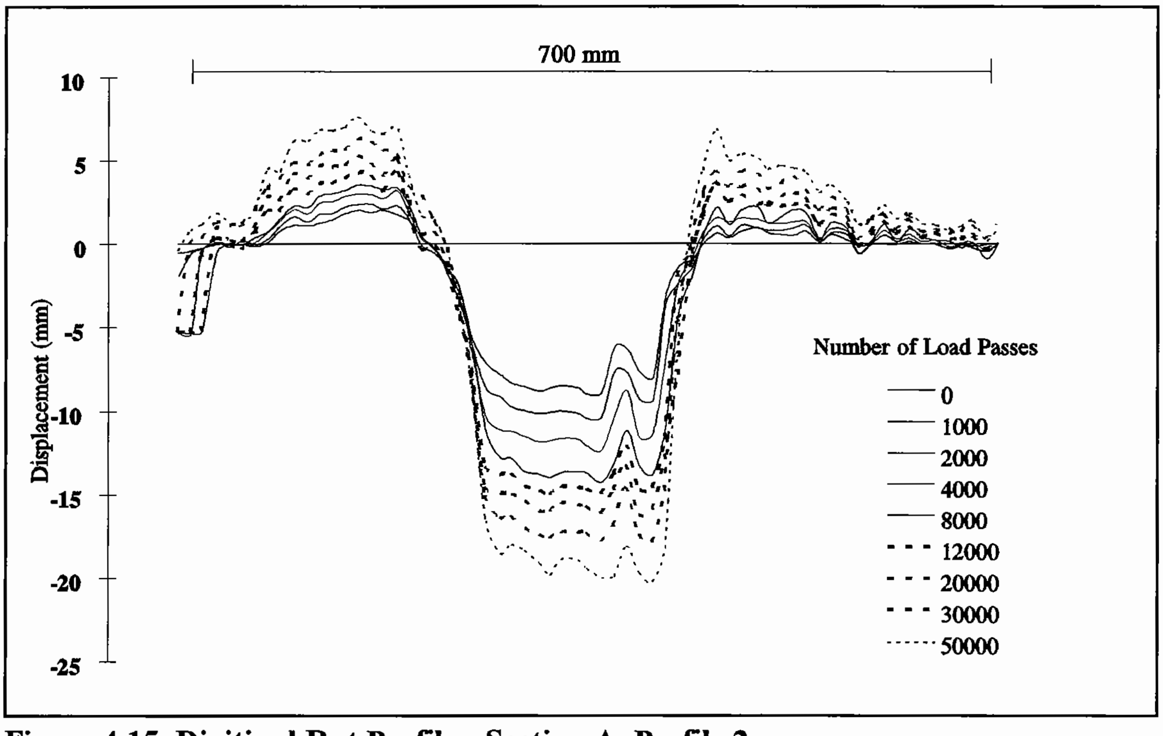


Figure 4.15 Digitised Rut Profile : Section A, Profile 2

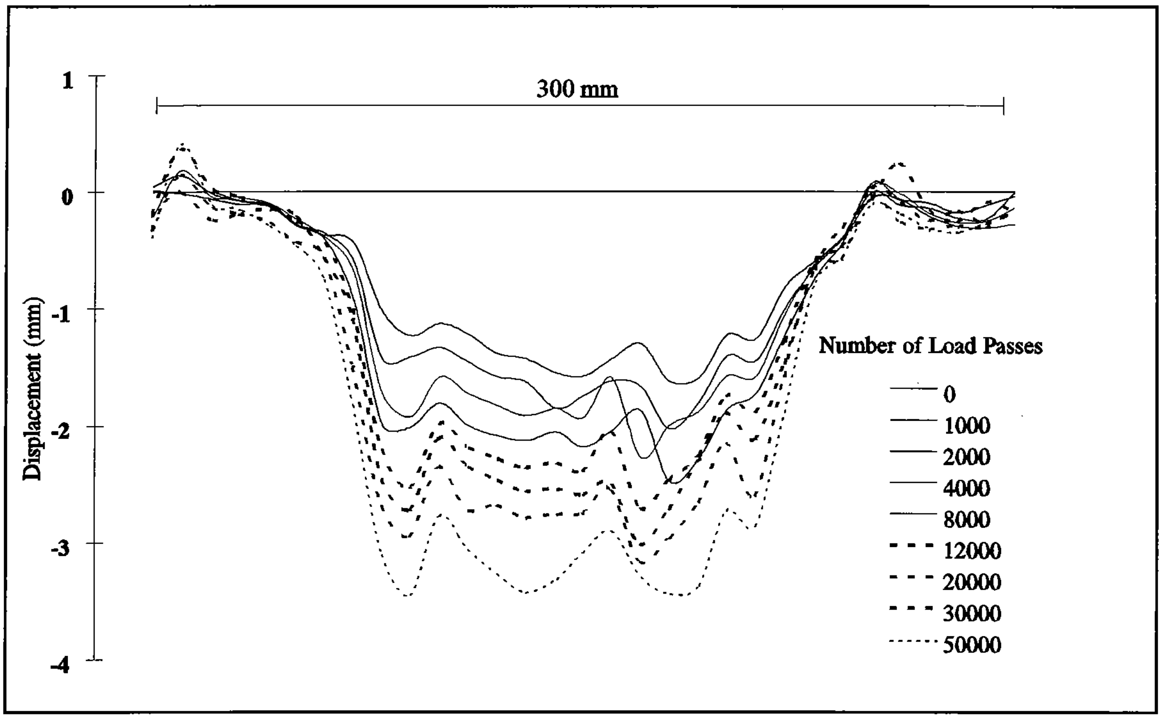


Figure 4.16 Digitised Rut Profile : Section B, Profile 2

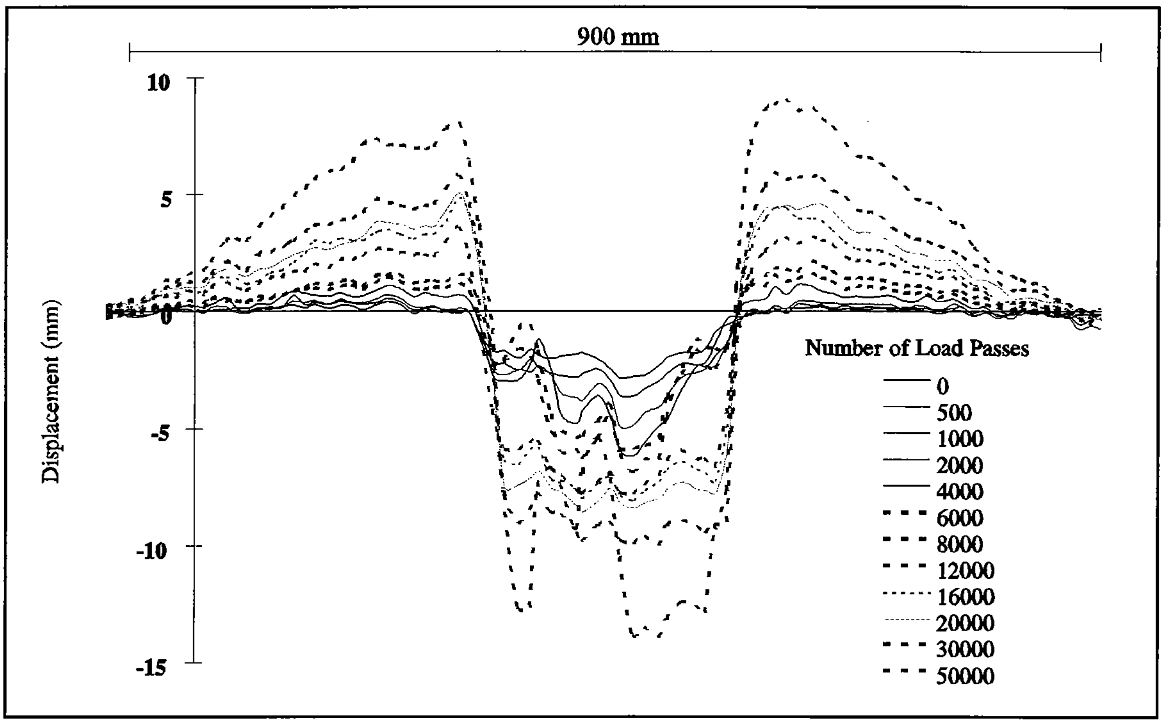


Figure 4.17 Digitised Rut Profile : Section C, Profile 2

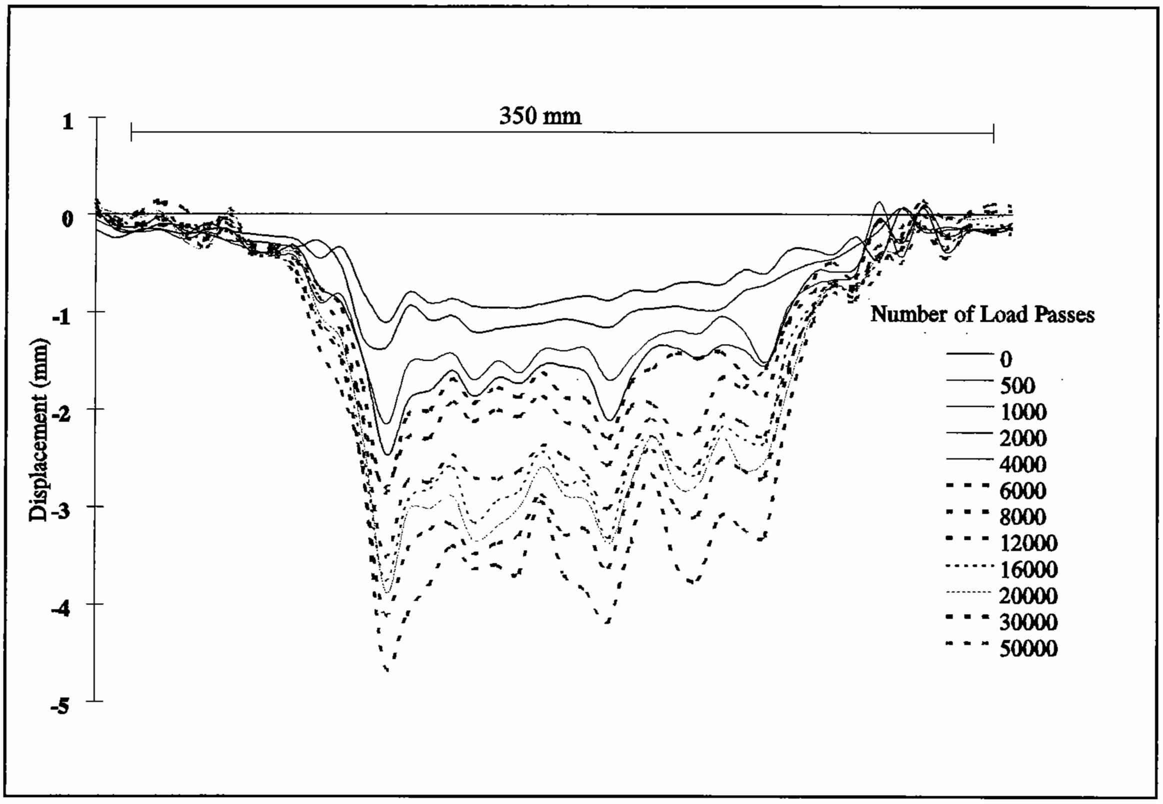


Figure 4.18 Digitised Rut Profile : Section D, Profile 3

These profiles clearly show the superior performance of sections B and D containing the mixtures with the modified binders and further illustrate the extent of lateral and upward displacement of the material at the edge of the rut in both the gravel and granite mixtures with conventional binder. The apparent decrease in rut depth at 50 000 passes in the section C profile (Figure 4.17) was caused by the formation of the tyre tread pattern in the material, as can be seen in Figure 4.19. Compositional analysis of the basecourse supplied for this section had shown it to have a grading which was fine of the target envelope and also the highest binder content. These factors may have contributed to the migration of binder and fine material to the surface under trafficking, in which the pattern of the tyre became imprinted.

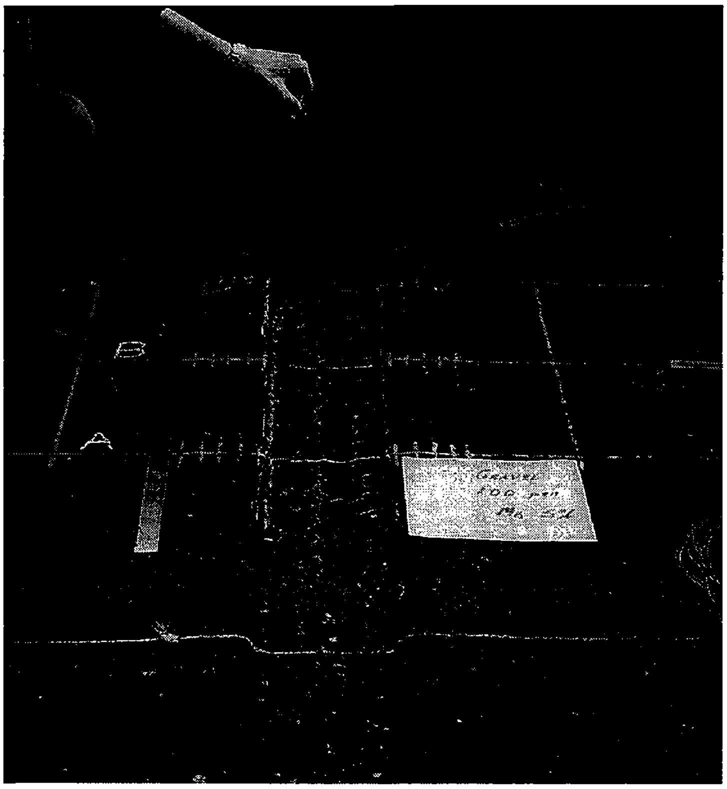


Figure 4.19 Formation of Tyre Tread Pattern in Surfacing, Section C

Rut depth, which was taken as the vertical distance from shoulder to trough, was obtained from the digitised data by subtracting the averaged vertical ordinate of the bottom of the rut from the mean vertical ordinate of the uppermost points of the left and right shoulders. The final values of rut depth which were derived from this analysis are presented in Table 4.2, while Figures 4.20 to 4.23 show the increase in rut depth with trafficking which was determined at the three profile measurement locations in each section.

Table 4.2 Final Rut Depths

Section	Rut Depth at 50 000 Passes (mm)			
	Profile 1	Profile 2	Profile 3	Mean
A	20.9	20.9	20.3	20.7
B	5.2	3.4	4.0	4.2
C	18.9	22.1	32.9	24.6
D	4.3	5.2	3.7	4.4

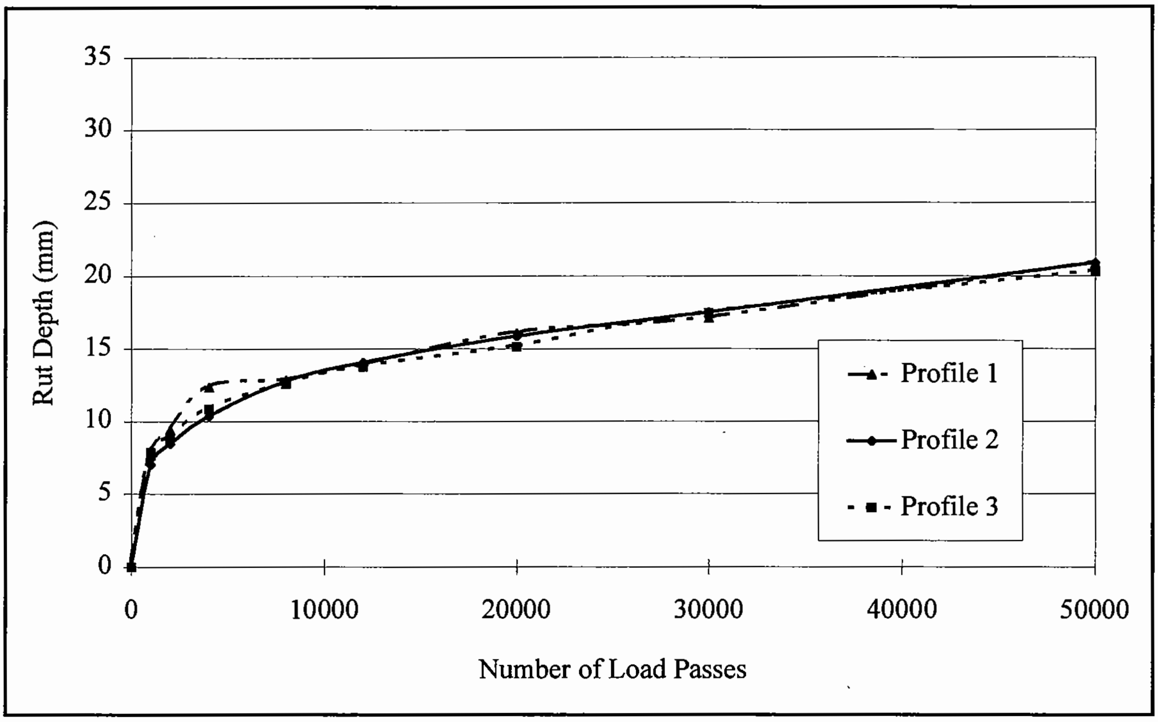


Figure 4.20 Rut Development : Section A

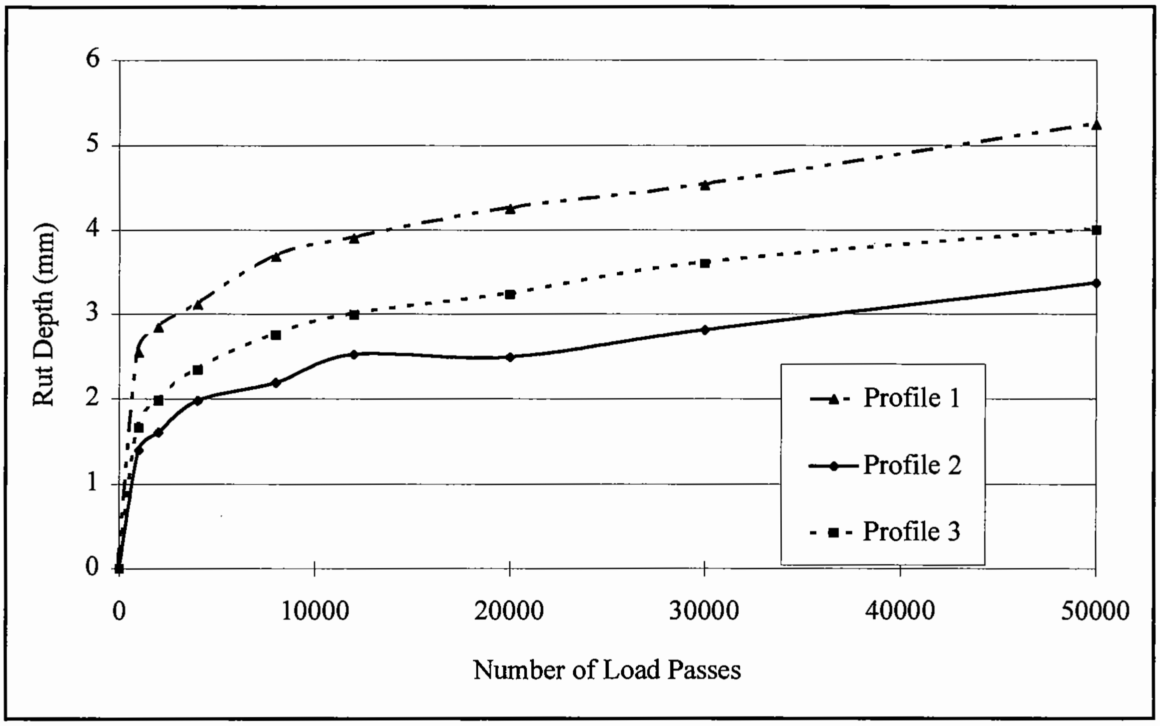


Figure 4.21 Rut Development : Section B

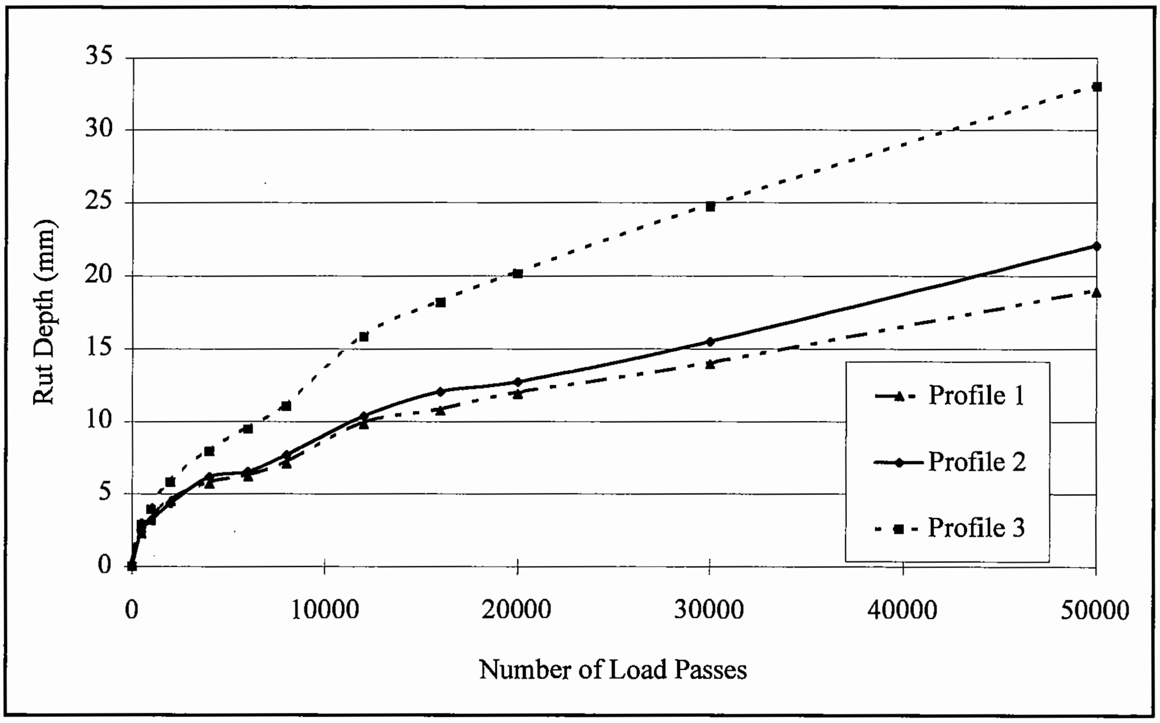


Figure 4.22 Rut Development : Section C

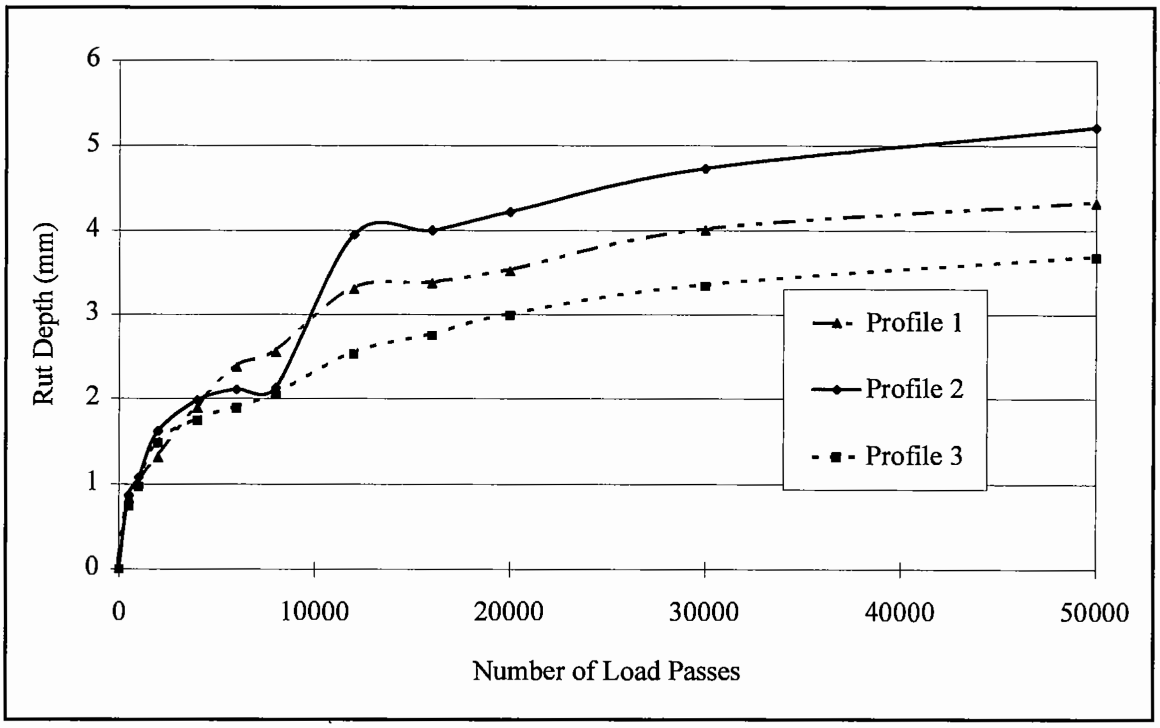


Figure 4.23 Rut Development : Section D

Figure 4.23 shows that none of the deformation/load cycles curves for section D had established a definite linear phase by the end of the test, and the same is true for at least one of those from section B (Figure 4.21). For this reason, and given the limited number of data points available for regression analysis, it was not feasible to calculate a rate of deformation from this data in the same way as for the wheeltracking test (see Chapter Three). A parameter termed mean deformation rate was, therefore, introduced. Mean deformation rate is obtained by taking the average of the values of deformation rate calculated over each measurement interval throughout the test. A mathematical definition is given below and the calculated values are given in Table 4.3.

$$\text{Mean deformation rate} = \frac{\sum \frac{\Delta\epsilon}{\Delta n}}{N} \qquad (4.1)$$

where N is the number of increments at which deformation is recorded

$\Delta\epsilon = (\epsilon_{i+1} - \epsilon_i)$, and ϵ_i is the deformation recorded at increment i

$\Delta n = (n_{i+1} - n_i)$, and n_i is the number of load cycles elapsed at increment i

Table 4.3 Mean Deformation Rates

Section	Mean Deformation Rate (mm per 1000 passes)			
	Profile 1	Profile 2	Profile 3	Mean
A	1.5	1.4	1.4	1.4
B	0.4	0.2	0.3	0.3
C	1.0	1.1	1.5	1.2
D	0.3	0.4	0.3	0.3

These results show not only the considerable improvement in performance afforded by the use of the SBR modifier, but also that the binder has had a more significant influence on resistance to deformation than aggregate type. The difference between the modified and unmodified bitumen was clear, but there was little difference between the performance of the two aggregate types when combined with the same binder. This was particularly evident in the modified mixtures where the performance of the granite and gravel aggregates was very close. For the unmodified materials, the greatest rut depths were recorded in the gravel mixture though this gave a lower mean deformation rate. This may possibly be due to the difference in intervals at which rut profiles were recorded for the gravel and granite mixtures, as the value of mean deformation rate is "weighted" to the part of the test where data collection rate is highest. For the gravel mixtures three more measurements of rut depth were made than for the granite materials. However, as these additional measurements were made after 500, 6000 and 16000 passes they all fell during the first half of the test where any weighting effect might be expected to increase the value of mean deformation rate.

4.9 Wheeltracking Tests

On completion of the testing of the trial pavement, slabs of the bituminous material from each section were cut from areas unaffected by the tracking. Three slabs of sufficient size for wheeltrack testing were obtained from sections A, B and D but only two from section C due to the extent of disruption in the material caused by the heaving associated with the rut. These slabs were trimmed to plan dimensions of 404 × 280mm to suit the dimensions of the test moulds and once placed in the test mould the underside of each slab, which had been formed against the granular sub-base, was levelled with casting plaster which also served to fix the slab firmly in the mould.

The wheeltracking tests were carried out at a temperature of 40°C under an applied wheel load of 790 N, which corresponds to a contact pressure of 930 kPa, and run for a duration of 15 000 load passes. Permanent deformation, determined as for the wheeltracking tests described in Chapter Three, was recorded at every 60th pass and the final values are presented in Table 4.4, together with mean deformation rates, which were calculated as for the PTF data. The test data for the specimens from each section are plotted in Figures 4.24 to 4.27.

Table 4.4 Wheeltracking Test Data

Section	Permanent Deformation at 15000 Passes (mm)				Mean Deformation Rate (mm per 1000 passes)			
	Test 1	Test 2	Test 3	Mean	Test 1	Test 2	Test 3	Mean
A	5.5	6.4	6.2	6.0	0.3	0.4	0.3	0.3
B	2.2	1.8	2.1	2.0	0.1	0.1	0.1	0.1
C	14.0	N/A	-	14.0	0.8	1.7	-	1.3
D	3.0	3.1	3.3	3.1	0.2	0.2	0.2	0.2

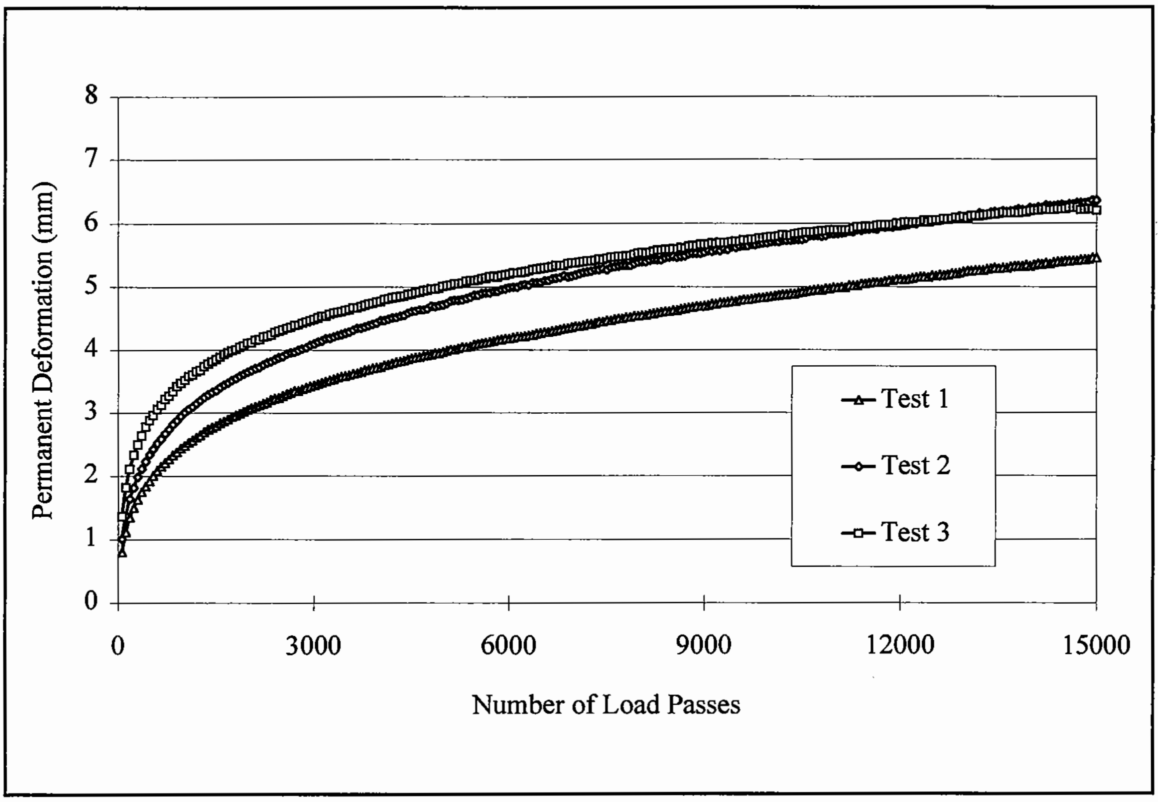


Figure 4.24 Wheeltracking Test Data Plots : Section A

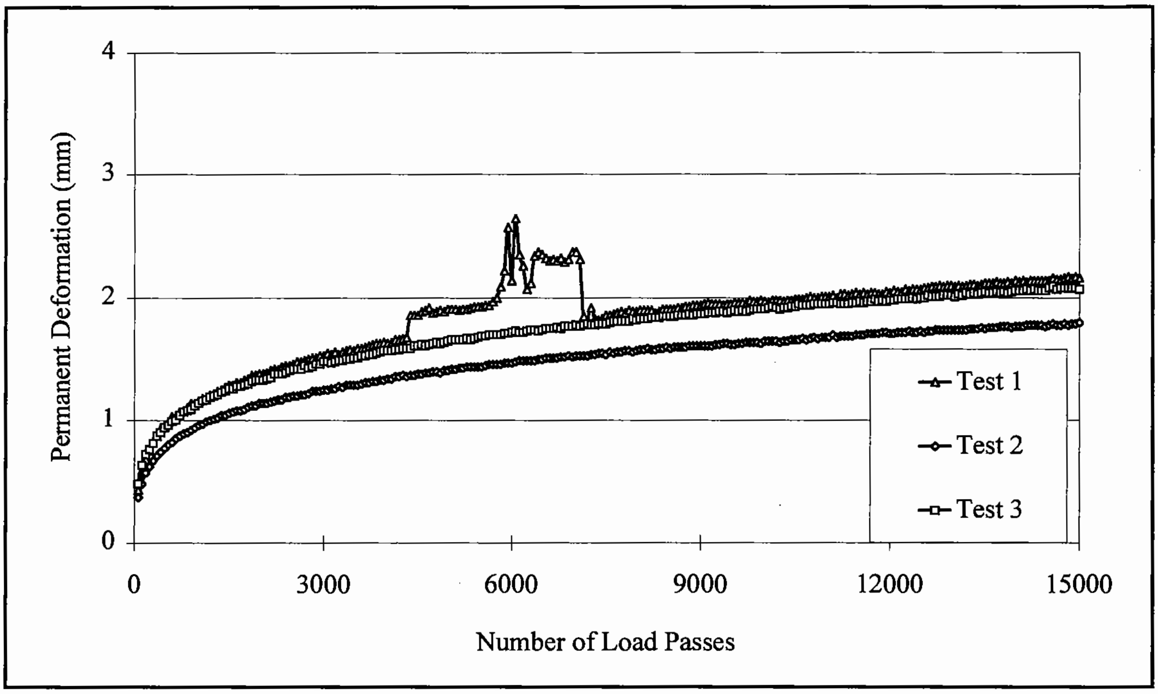


Figure 4.25 Wheeltracking Test Data Plots : Section B

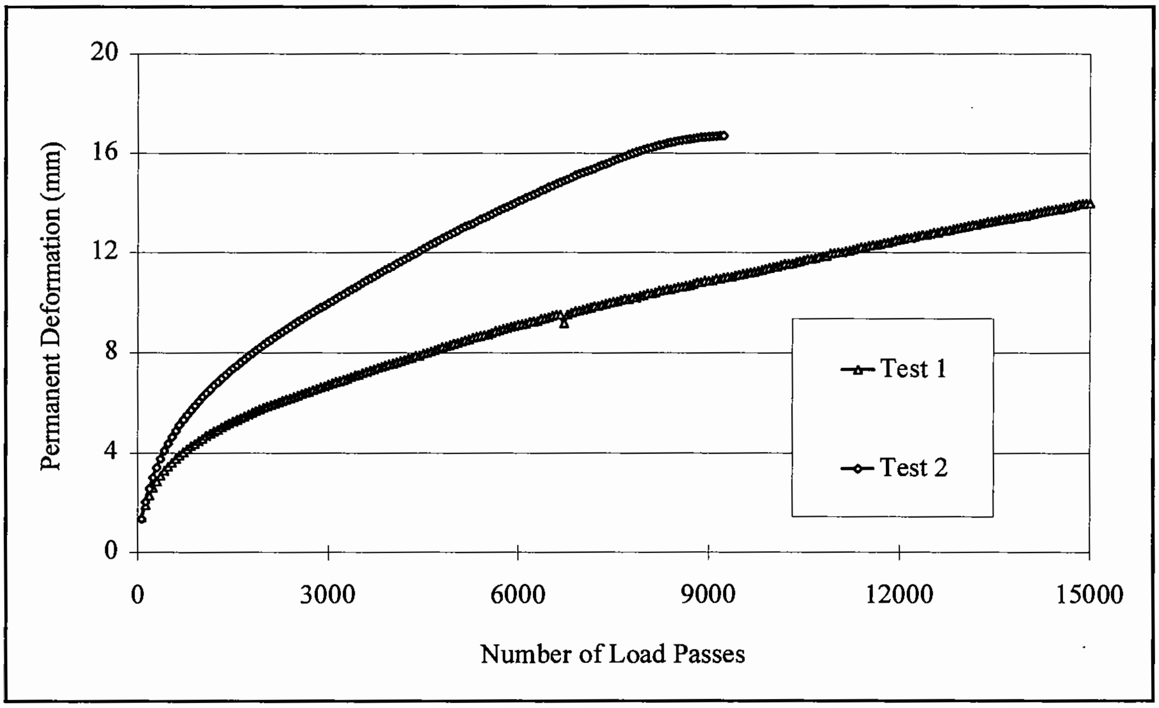


Figure 4.26 Wheeltracking Test Data Plots : Section C

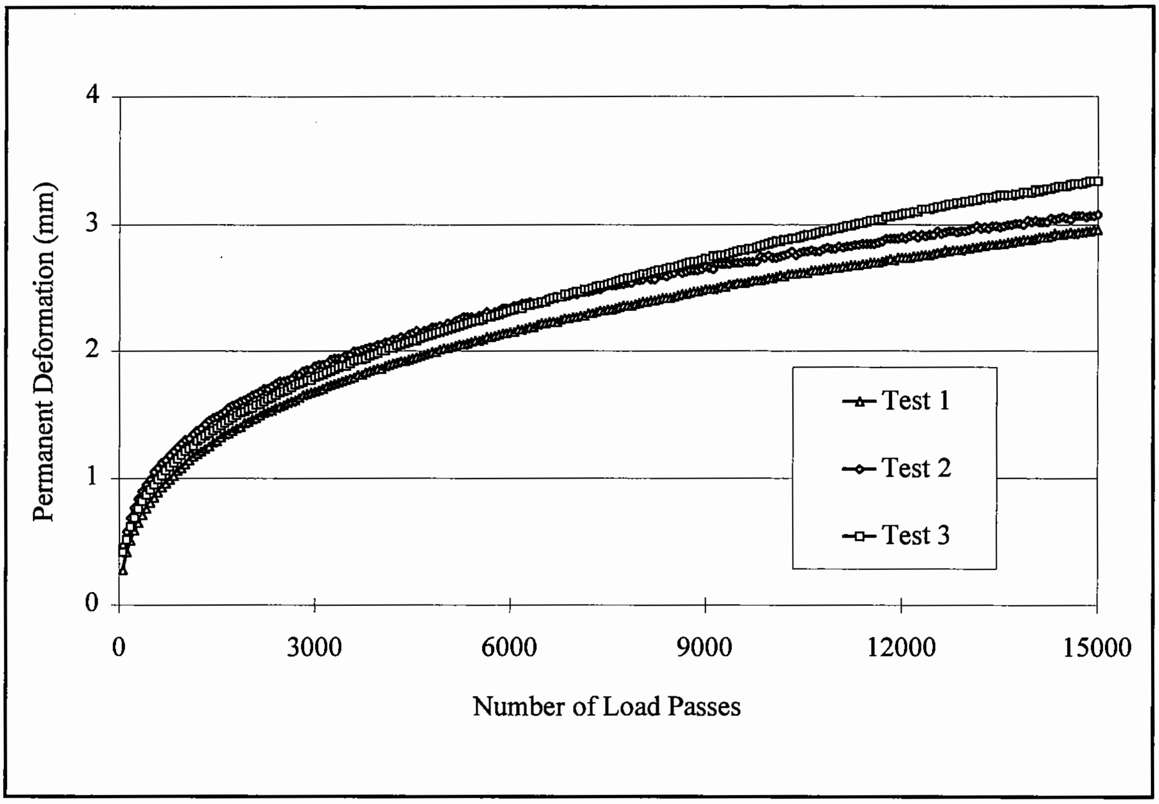


Figure 4.27 Wheeltracking Test Data Plots : Section D

These results clearly demonstrate the superior performance of the mixtures with the modified binder, but they also appear to show a greater influence of the aggregate type than was apparent in the results from the PTF. The modified mixture with granite aggregate was consistently better than the modified gravel mixture, though the most notable feature was the very poor performance of the material in section C, the unmodified mixture with gravel aggregate.

4.10 Repeated Load Axial Tests

RLA tests were carried out on 100mm diameter cores which were cut both direct from the trial pavement and, with the exception of the unmodified gravel, from the wheeltracked slabs after testing. Only cores taken from the trial pavement were tested for the material from section C, since the rutting in the wheeltracked slabs was so severe that no cores could be obtained.

The test conditions were as follows:

Test temperature	:	40°C
Compressive stress	:	100 kPa
Load pulse frequency	:	0.5 Hz
Load pulse duration	:	1 second
Test duration	:	10 000 cycles
Pre-test conditioning	:	10 minutes @ 10 kPa (static)

Prior to testing, the density of each core was determined with the core sealed during immersion for volume measurement. The ranges and average values of void content which were calculated, based on theoretical maximum densities determined by the Rice method, are given in Table 4.5.

Table 4.5 Air Void Contents of Repeated Load Axial Test Specimens

Section	Air Void Content (%)		
	Range	Mean	Standard Deviation
A	2.2 - 8.6	5.3	2.1
B	5.1 - 14.3	9.1	2.8
C	6.4 - 10.1	7.8	1.0
D	3.6 - 10.0	5.8	1.5

There are clearly differences in void content between the materials, which it is assumed reflect the variation in compacted state between the sections as laid in the PTF. This does not, however, affect the analysis as the purpose of this testing was not to compare directly the performance of the mixtures but to compare performance in the RLA test with that in the PTF. Furthermore, the RLA test plots, Figures 4.28 - 4.31, indicate that there was no consistent effect of air void content on performance for the materials from any of the sections.

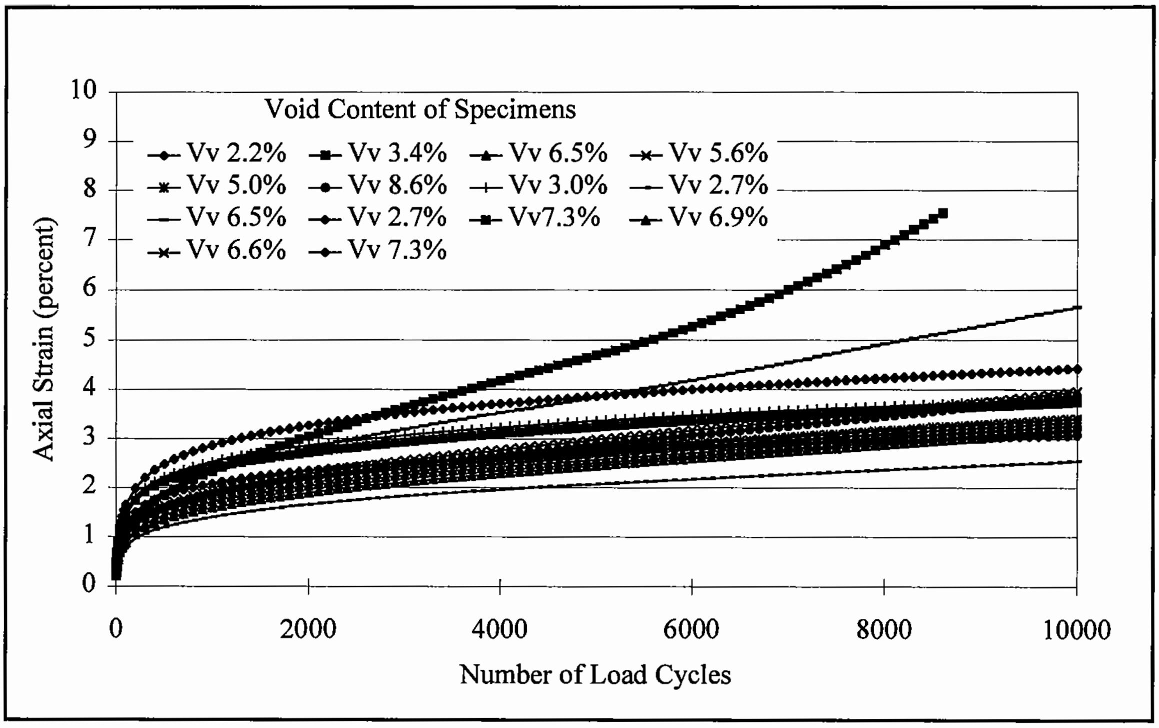


Figure 4.28 Data Plots for RLA Tests at 40°C : Section A

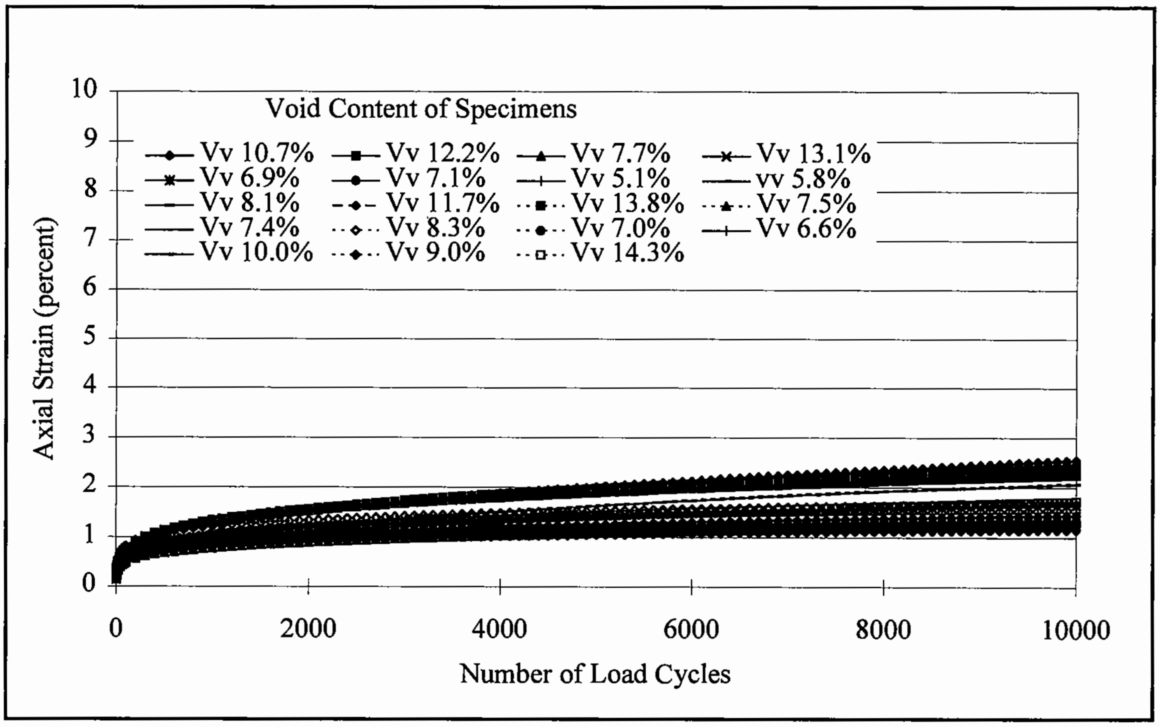


Figure 4.29 Data Plots for RLA Tests at 40°C : Section B

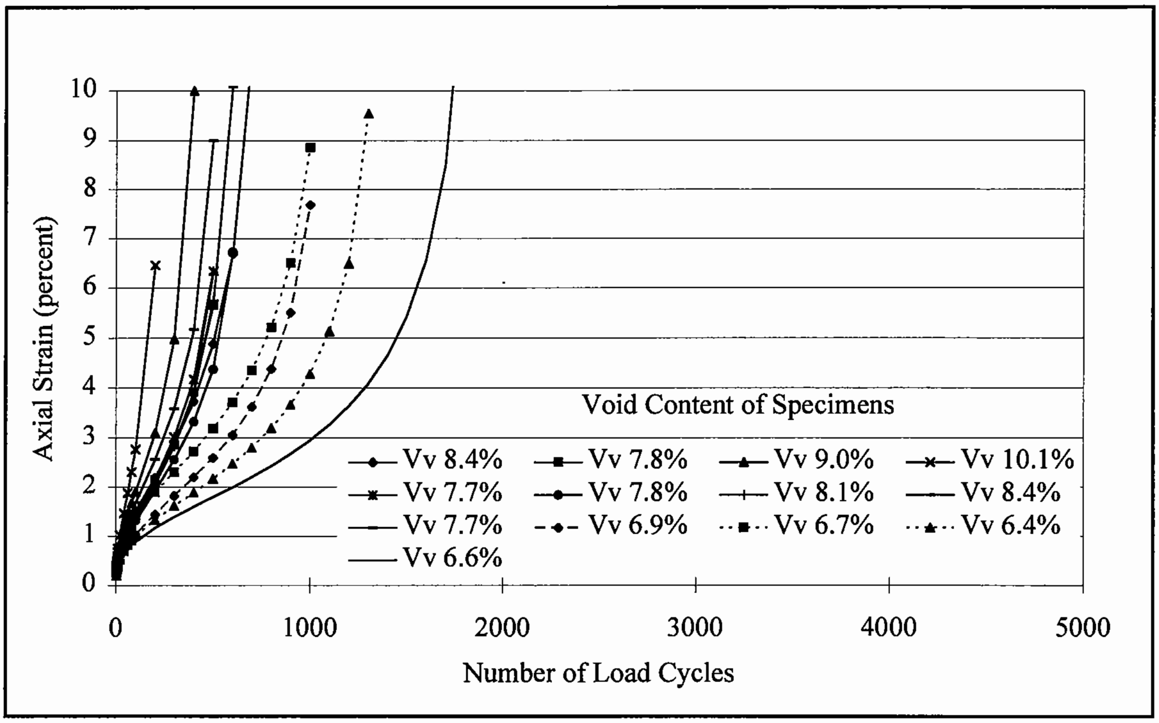


Figure 4.30 Data Plots for RLA Tests at 40°C : Section C

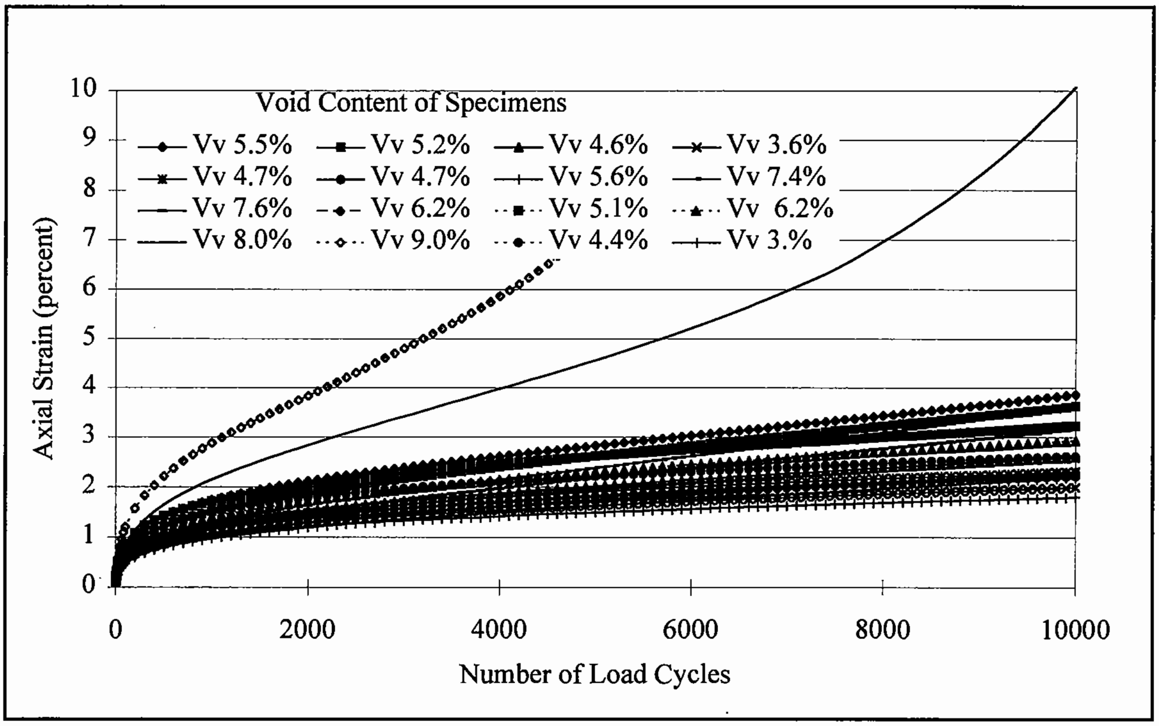


Figure 4.31 Data Plots for RLA Tests at 40°C : Section D

The average values of ultimate strain and mean strain rate, which was calculated in the same manner as mean deformation rate for the PTF and wheeltracking data, for the tests on material from each section are presented in Table 4.6. These data show the strain rates for the material from section D to be rather better than those for the material from section A, though the two materials are similar in terms of ultimate strain.

Table 4.6 Results of Repeated Load Axial Tests at 40°C (Mean Values)

Section	Ultimate Strain (%)	Mean Strain Rate (microstrain/cycle)
A	3.7	20
B	1.6	9
C	Fail	178
D	3.5	13

Both the plots and the tabulated data show a difference in the performance of the two aggregate types when in combination with the same binder, which is consistent with the findings from the wheeltracking tests but tends to conflict with the PTF data. It is clear, however, that in the RLA tests though the modified mixture has outperformed the unmodified mixture with the same aggregate, in both cases the materials with granite aggregate have shown better resistance to deformation than those containing gravel.

As for the wheeltracking tests, the unmodified gravel mixture (section C) performed very poorly.

4.11 Comparison of Test Results

Table 4.7 shows the ranking of the materials as determined by their performance in each of the tests on the basis of the average value of ultimate rut depth or strain, as appropriate, while Table 4.8 shows the rankings determined on the basis of the average value of mean deformation or strain rate.

Table 4.7 Ranking of Mixture Performance by Ultimate Deformation/Strain

Rank	Test System		
	Wheeltrack	PTF	RLA
1	B	B	B
2	D	D	D
3	A	A	A
4	C	C	C

Table 4.8 Ranking of Mixture Performance by Mean Deformation/Strain Rate

Rank	Test System		
	Wheeltrack	PTF	RLA
1	B	B, D	B
2	D	-	D
3	A	C	A
4	C	A	C

The wheeltracking and RLA tests have given the same rankings for the mixtures by both ultimate deformation/strain and deformation/strain rate, which supports the correspondence between these tests which was reported in Chapter Three. The PTF, however, while in agreement with the wheeltracker and RLA test on the basis of ultimate deformation/strain produced a different ranking when based on mean deformation/strain rate.

Such rankings do not, however, give any indication of the relative performance of the materials. In order, therefore, to provide a better comparison of performance in each of the tests a simple index was used. The index for the material from each section was obtained by dividing the mean test result for section B by the mean result for the section. Thus the material from section B always has an index of unity.

The result for section B was taken as the reference value for each test method since this material (granite aggregate with SBR modified binder) was shown to be the most deformation resistant in every case.

The performance indices based on ultimate deformation or strain and mean deformation or strain rate are given in Tables 4.9 and 4.10, respectively, and are presented graphically in Figures 4.32 and 4.33.

Table 4.9 Mixture Performance Indices Based on Ultimate Deformation/Strain

Mixture	Test System		
	Wheeltrack	PTF	RLA
A	0.33	0.20	0.43
B	1.00	1.00	1.00
C	0.14	0.17	N/A
D	0.65	0.95	0.45

Table 4.10 Mixture Performance Indices Based on Mean Deformation/Strain Rate

Mixture	Test System		
	Wheeltrack	PTF	RLA
A	0.32	0.23	0.43
B	1.00	1.00	1.00
C	0.08	0.27	0.05
D	0.58	0.94	0.69

Table 4.9 shows the PTF performance index for material from section D to be marginally lower than that from section B though the mean deformation rates reported in Table 4.3 are the same. This is because the values presented in Table 4.3 have been rounded to one decimal place.

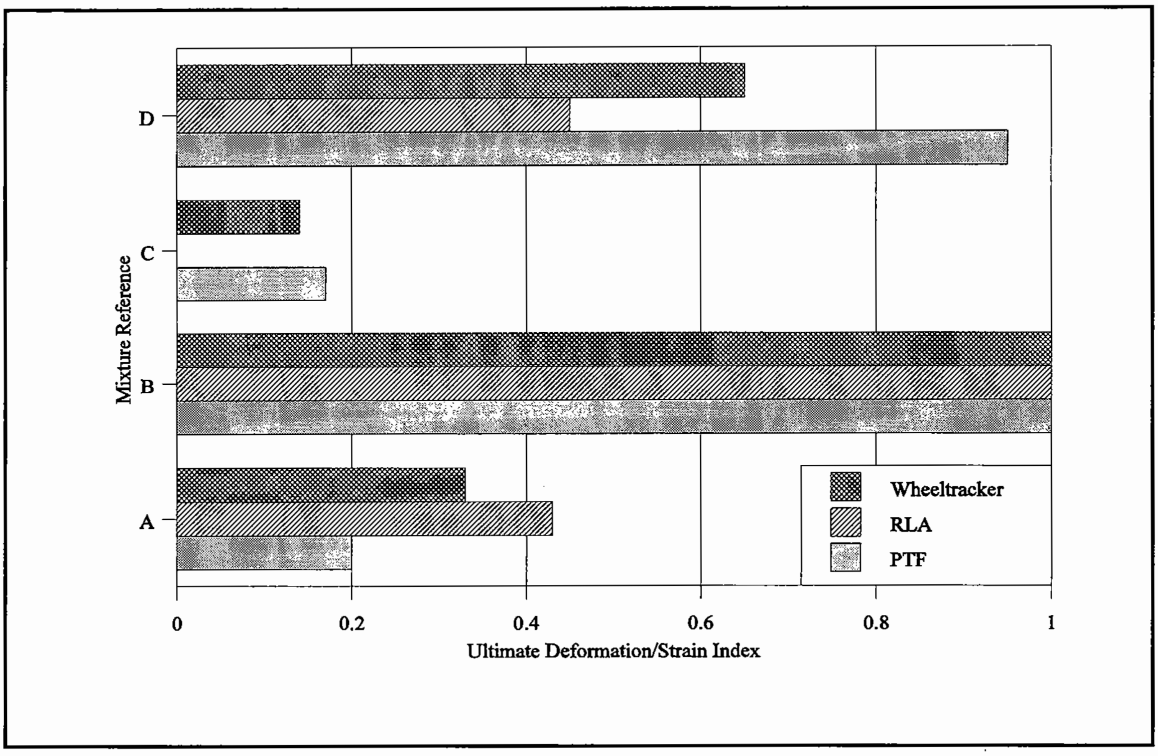


Figure 4.32 Mixture Performance Indices Based on Ultimate Deformation/Strain

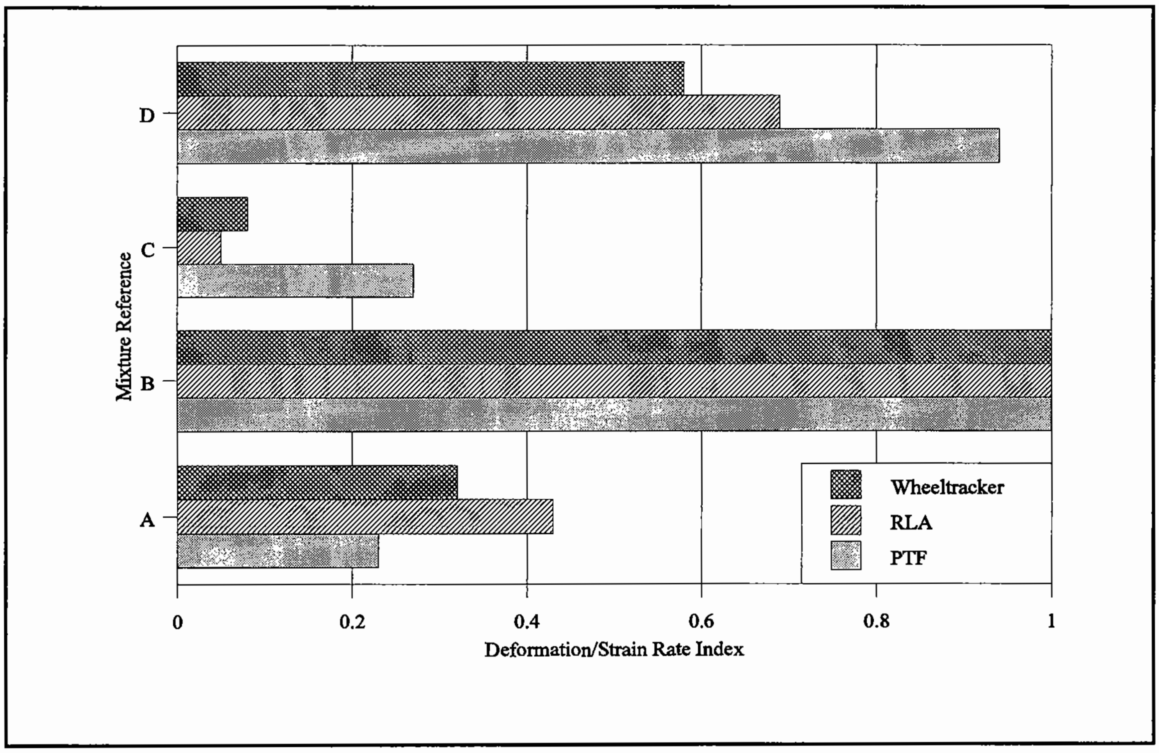


Figure 4.33 Mixture Performance Indices Based on Mean Deformation/Strain Rate

This analysis shows that in comparison with the PTF, both the RLA and wheeltracking tests underrated the performance of the gravel mixtures and overrated the unmodified granite mixture. This suggests that these latter tests have not reflected the influence of the binder to the extent which was apparent in the PTF trial. However, in order to accelerate the tests and obtain significant strains in the specimens, both the RLA and wheeltracking tests had been carried out at 40°C, which was significantly higher than the temperature of 30°C used in the PTF. To assess whether this increased temperature, and consequent reduction in binder stiffness, was sufficient to account for this observed diminution of the effect of the binder, a further series of RLA tests was performed at 30°C. These tests were carried out only on specimens from section B, as this was used as the standard material for calculation of performance indices, and from section C, which had shown the worst performance in the RLA and wheeltracking tests. In view of the early failure of specimens from section C in the RLA tests at 40°C, the duration of the test was reduced to 3600 load cycles for the programme at 30°C. Table 4.11 presents both the averaged test results and calculated RLA test performance indices, while the data plots are shown in Figures 4.34 and 4.35. The values of mean strain rate given in Table 4.11 are not directly comparable with those presented in Table 4.6 because the duration of the test, ie the number of strain recording increments, is a factor in the calculation of this parameter.

Table 4.11 Results of Repeated Load Axial Tests at 30°C and Performance Indices

Section	RLA Test Results		RLA Test Performance Indices	
	Ultimate Strain (%)	Mean Strain Rate (microstrain/cycle)	Ultimate Strain	Mean Strain Rate
B	1.1	14	1.00	1.00
C	6.1	64	0.17	0.22

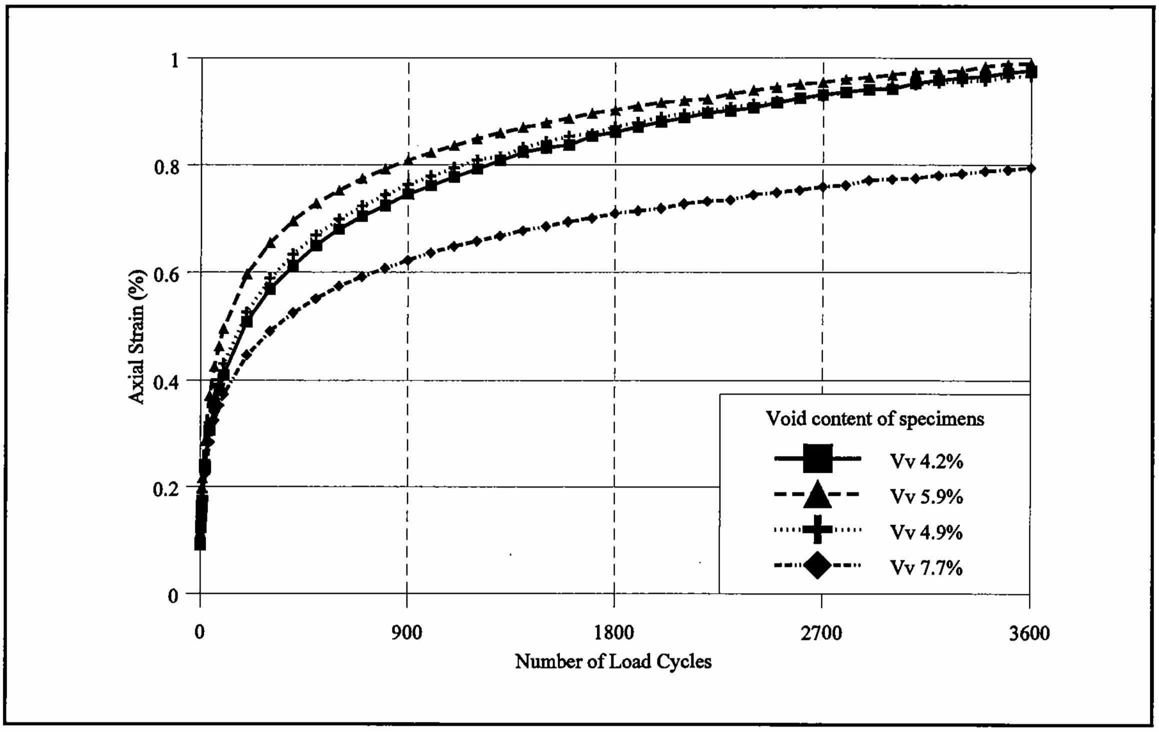


Figure 4.34 Data Plots for RLA Tests at 30°C : Section B

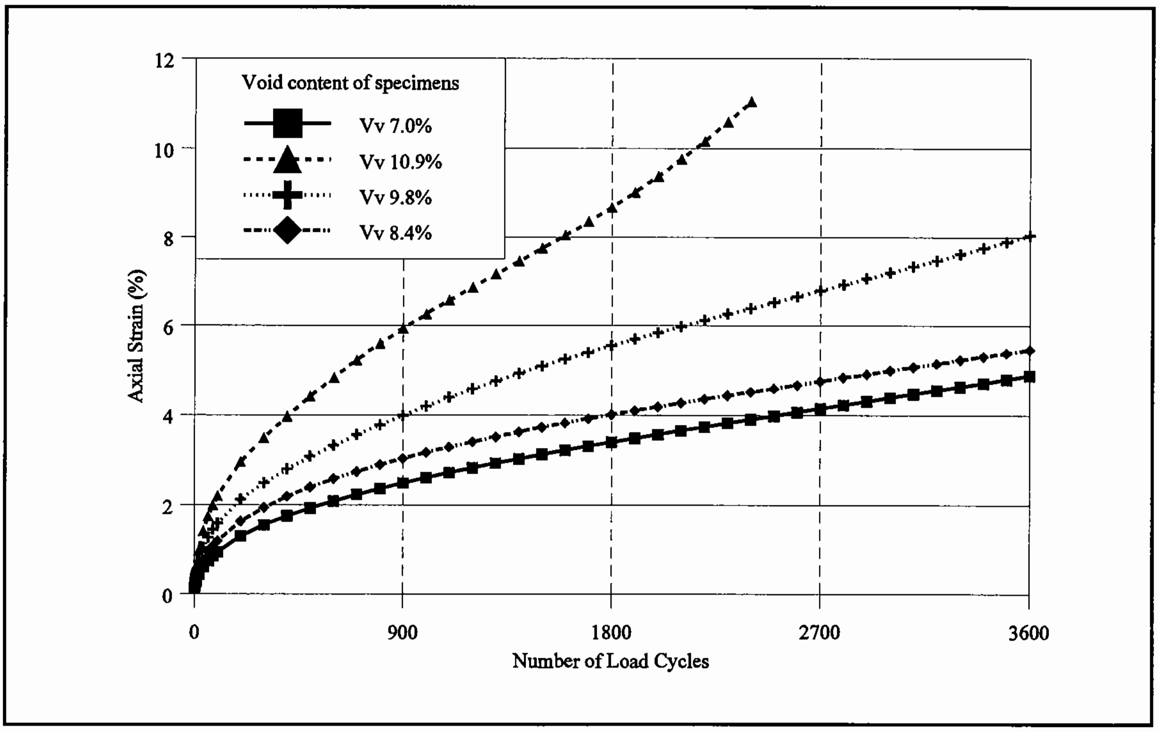


Figure 4.35 Data Plots for RLA Tests at 30°C : Section C

The performance indices calculated for the unmodified gravel mixture from these tests, 0.17 and 0.22 for ultimate strain and mean strain rate respectively, are much closer to the corresponding values for the PTF trials, 0.17 and 0.27, than those which were obtained from the RLA tests at 40°C, no result and 0.05, which serves to illustrate the importance of binder stiffness in resistance to permanent deformation. Such improvement in correspondence may not have resulted for the modified gravel mixture because of the reduction in temperature susceptibility caused by the addition of the modifier.

4.12 Confined Repeated Load Axial Tests

An element of bituminous material subjected to vehicle loading in a pavement is confined by the surrounding material. By carrying out plate loading tests with a 4 inch (102mm) diameter plate on 2 inch (51mm) thick slabs of a bituminous concrete Goetz et al (110) established that increasing the plan dimensions of the specimen up to the point where the ratio of loaded area to unloaded area reached a value of around 1:10 produced an increase in compressive strength. Further increase in specimen size was found to have little effect. This gives an indication of the extent of the influence of surrounding material. Small scale wheeltracking tests will in some way simulate the effect of confinement which occurs in the pavement, though criticisms of such tests in which the size of the specimen is small in comparison to the size of the wheel have been reported in Chapter Three, while larger scale facilities such as the PTF should be representative in this respect.

Triaxial testing on bituminous mixtures has shown the application of a confining pressure to have a significant positive effect on the permanent deformation performance of axially loaded specimens (110,114,124). Results of repeated load (sinusoidal) triaxial tests on DBM's and rolled asphalts reported by Brown and Cooper (20) suggest that the use of confinement is more beneficial for continuously graded materials, which rely on the aggregate interlock load transfer mechanism, than for gap graded materials which rely on the mortar mechanism. Recent work in France by LCPC (125) on gussasphalt (mastic asphalt), porous asphalt and asphaltic concrete also indicates that confinement is very significant for mixture types whose resistance to deformation comes from the aggregate skeleton but less so, if at all, for those whose deformation resistance is based on the stiffness of the binder/filler mortar.

The use of a triaxial cell necessarily makes test equipment more expensive and test procedures more complex. However, it was considered that the facility to confine specimens during testing was important if an axial test was to be widely accepted and used, particularly as triaxial testing is being advocated by the French for adoption in European standards (125). Therefore, an assessment was made of a simple system which had been proposed (126) in which the specimen could be confined through the application of a partial vacuum, with the specimen sealed using only a latex membrane, i.e. without the need for a triaxial cell.

In this system the specimen is evacuated through a porous stone disc, of the type used in triaxial testing on soils, positioned centrally in the plan area of the lower platen and fitted flush with the surface of that platen, as shown in Figure 4.36. The porous stone is ported, through the body of the platen, to a vacuum pump. The latex sealing membrane, also of the type used in triaxial testing, is fitted over the specimen and both upper and lower platens, and secured using rubber 'O' rings which locate in a groove machined around the circumference of each platen in the plane of the surface of the platen. The NAT configured for confined RLA testing using this technique is shown in Figure 4.37.

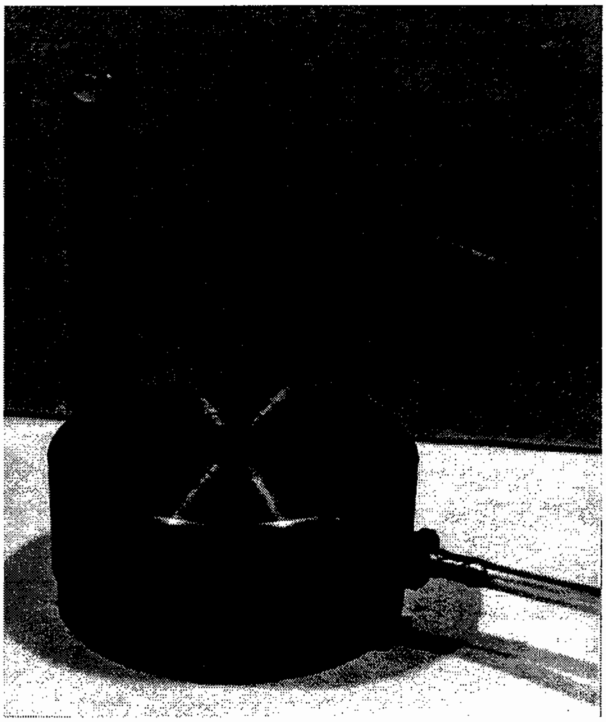


Figure 4.36 Lower Platen used for RLA Tests with Confinement by Vacuum

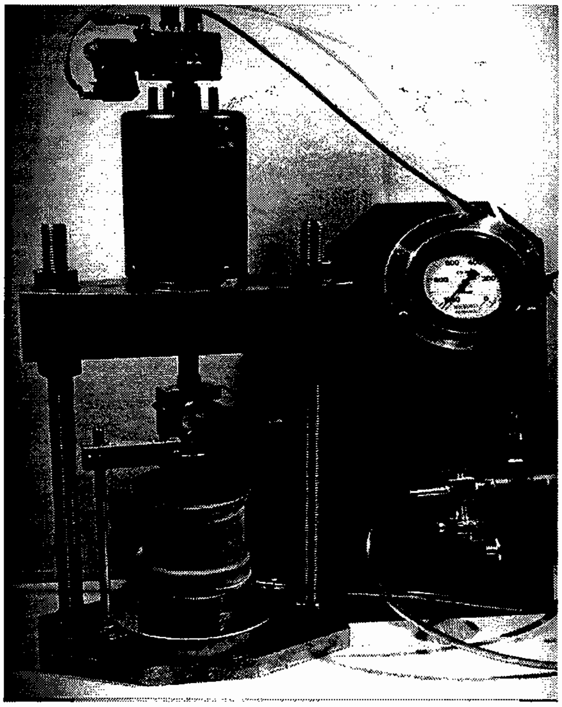


Figure 4.37 The NAT Configured for Confined RLA Testing

Preliminary trials had demonstrated that a partial vacuum equivalent to a confining pressure of 70 kPa (700 mbar) would produce real effects, and so this was the value used in a further series of tests carried out on cores from the PTF. Other than the application of confinement, test conditions were as for the original programme of RLA testing, with a test temperature of 40°C and duration of 10 000 load cycles. The average values of test results and mixture performance indices for the confined RLA tests are given Table 4.12, and the test data plots are shown in Figures 4.38 to 4.41.

Table 4.12 Confined Repeated Load Axial Test Results and Performance Indices

Section	RLA Test Results		RLA Test Performance Indices	
	Ultimate Strain (%)	Mean Strain Rate (microstrain/cycle)	Ultimate Strain	Mean Strain Rate
A	0.7	5	0.59	0.56
B	0.4	3	1.00	1.00
C	0.7	5	0.59	0.56
D	0.6	4	0.65	0.80

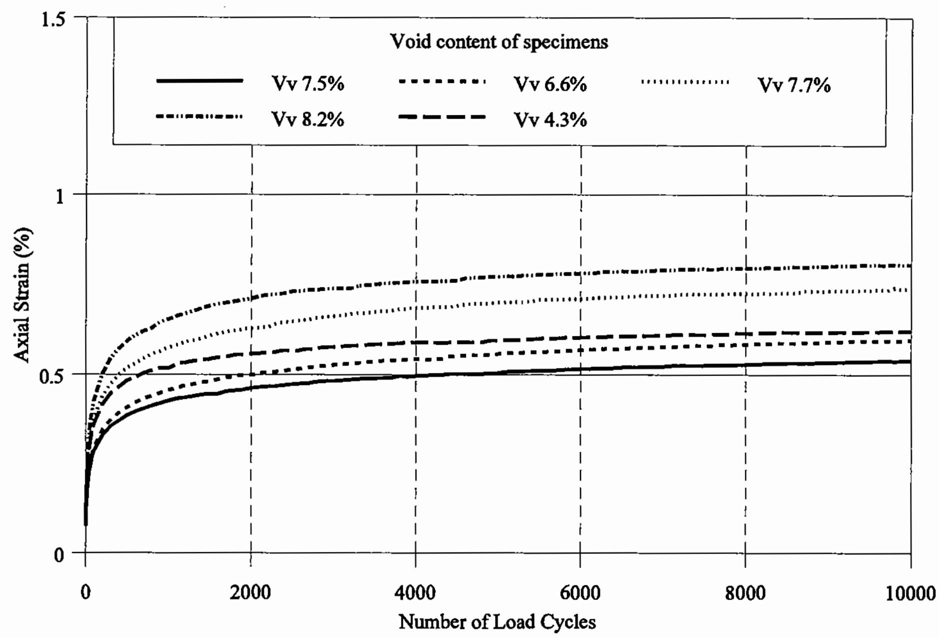


Figure 4.38 Data Plots for Confined RLA Tests at 40°C : Section A

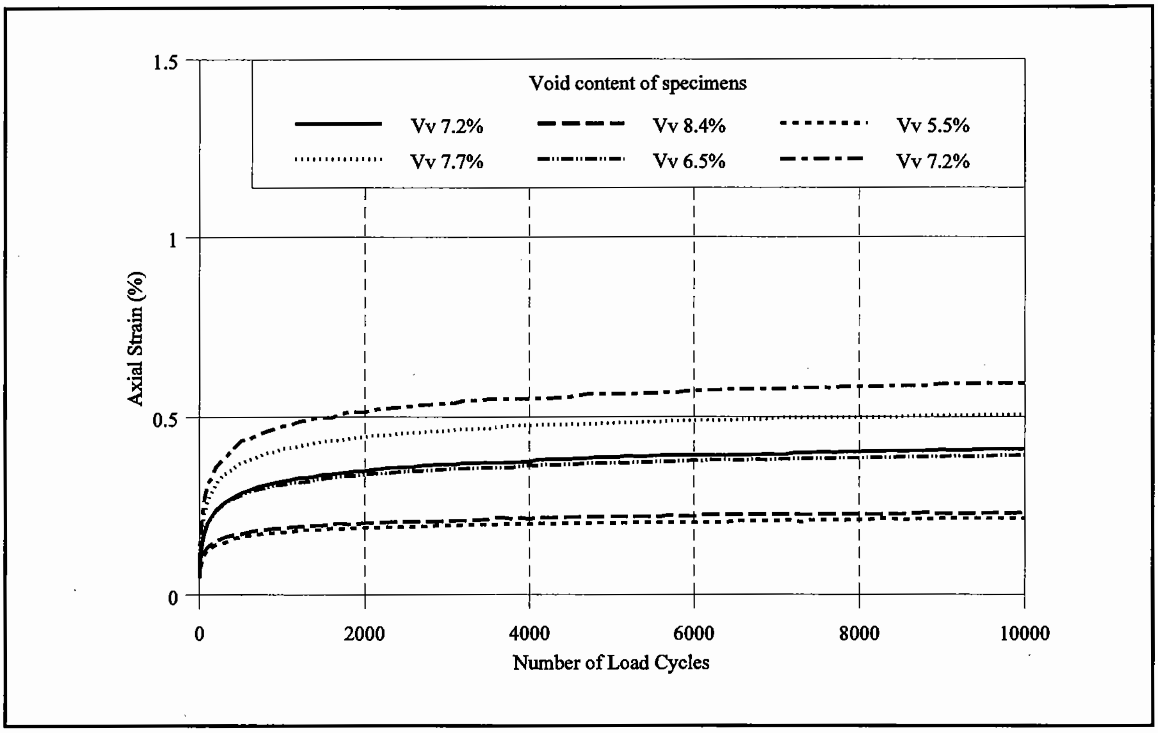


Figure 4.39 Data Plots for Confined RLA Tests at 40°C : Section B

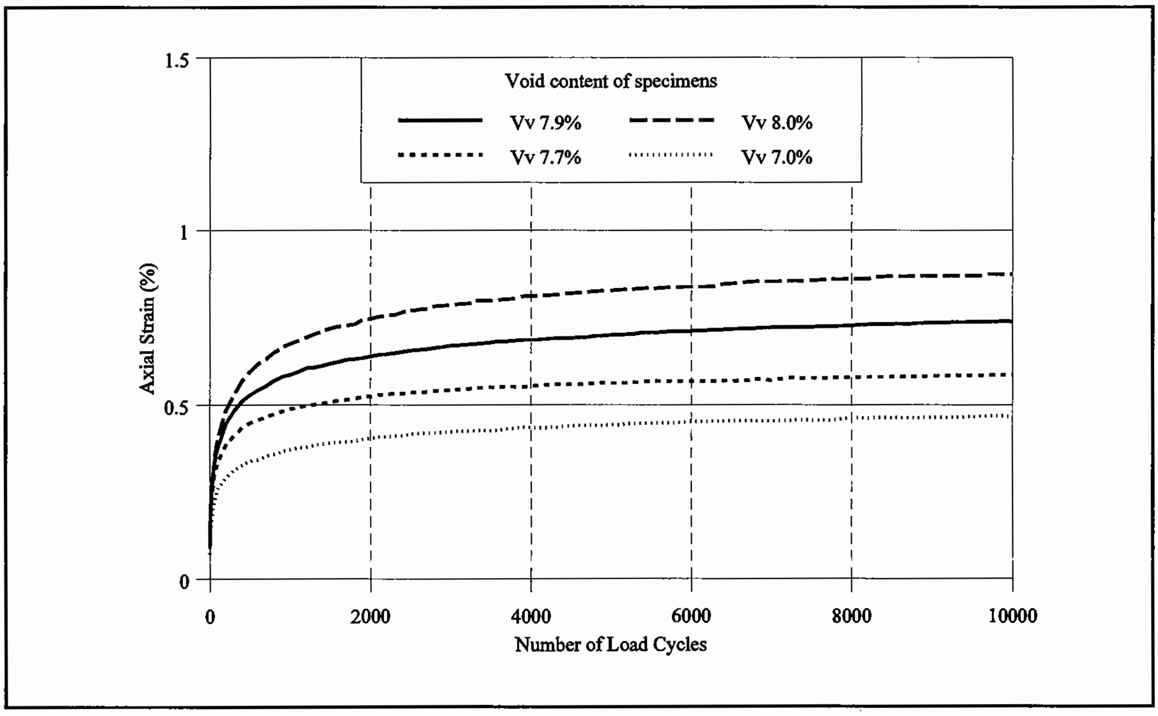


Figure 4.40 Data Plots for Confined RLA Tests at 40°C : Section C

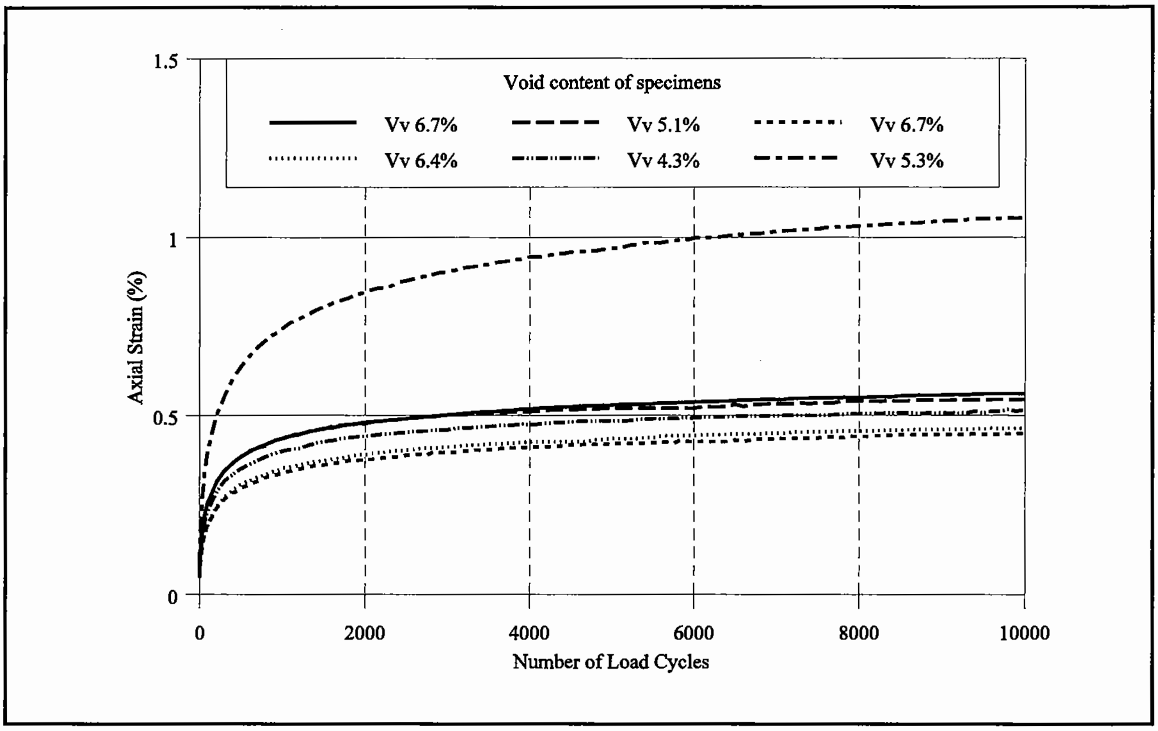


Figure 4.41 Data Plots for Confined RLA Tests at 40°C : Section D

The test data plots show that for all four mixtures the application of confining stress has reduced the magnitude of permanent strain below that which occurred in the unconfined tests at 40°C (Figures 4.28 - 4.31). The effect of confinement has been particularly significant for the unmodified gravel mixture the performance of which, in terms of the performance indices, is indistinguishable from that of the unmodified granite. In comparison to its performance in the PTF the unmodified gravel mixture is considerably overrated in the confined test whereas it was significantly underrated in the unconfined test at 40°C, in which all specimens failed rapidly.

4.13 Discussion

The performance of the mixtures tested in the trial pavement was dominated by binder type rather than aggregate type. It is not, however, possible to ascertain from this data whether the superior performance of the SBR modified mixtures was due to greater binder stiffness resulting from reduced temperature susceptibility, enhanced strain recovery properties, a combination of these two or some other mechanism.

The ranking of the mixtures on the basis of ultimate deformation obtained from the TRL wheeltracking test matched that from the PTF, but this test appeared to reflect a greater influence of aggregate type on resistance to deformation. Though the modified mixtures still outperformed the unmodified mixtures in the wheeltracker, the granite aggregate was clearly superior to the gravel when in combination with the same binder, as demonstrated by the mixture performance indices (Tables 4.9 and 4.10, Figures 4.32 and 4.33).

Similar results were obtained from the unconfined RLA tests at 40°C, with the effect of aggregate type being apparent and particularly poor performance from the unmodified gravel mixture. However, further unconfined tests at 30°C showed a considerable improvement in performance of this material both in terms of the magnitude of recorded strains, even though the tests were shorter, and in relation to the performance of the SBR modified granite mixture. The effect of reducing the test temperature from 40°C to 30°C would have been to increase the stiffness of the unmodified binder and, as can be seen from Table 4.13 which includes the performance indices from the PTF and RLA tests under all conditions, has resulted in values of the performance indices close to those from the PTF.

Table 4.13 Mixture Performance Indices for PTF and RLA Tests under all Conditions

Section	Ultimate Deformation/Strain Index				Mean Deformation/Strain Rate Index			
	PTF	RLA @ 40°C	RLA @ 30°C	Confined RLA @ 40°C	PTF	RLA @ 40°C	RLA @ 30°C	Confined RLA @ 40°C
A	0.20	0.43	-	0.59	0.23	0.43	-	0.56
B	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
C	0.17	N/A	0.17	0.59	0.27	0.05	0.22	0.56
D	0.95	0.45	-	0.65	0.94	0.69	-	0.80

Despite this improved correspondence when testing at the same temperature as used in the PTF, the binder stiffness in the unmodified gravel mixture at 30°C would have been significantly lower in the RLA tests than in the trial pavement due to differences in loading time. The duration of loading in the RLA test is 1 second while, using the expression given below (10), the loading time at a depth of 50mm in the pavement with a wheel velocity of 4 km/hr is estimated as approximately 0.2 seconds. The effect of loading time on binder stiffness is discussed further in Chapter Eight.

$$\log t = 0.5\times10 - 4 \times h - 0.2 - 0.94 \log v \qquad (4.2)$$

where t is loading time (seconds)
h is depth (mm)
v is vehicle speed (km/hr)

The application of confinement also had a significant effect on the performance of the mixtures, with the magnitude of recorded strains being lower in each case than for the unconfined tests at the same temperature. The modified mixtures still fared best but in terms of the performance indices, shown graphically in Figures 4.42 and 4.43, the unmodified materials showed improvements over their performance in both the unconfined test and, perhaps more significantly, the PTF.

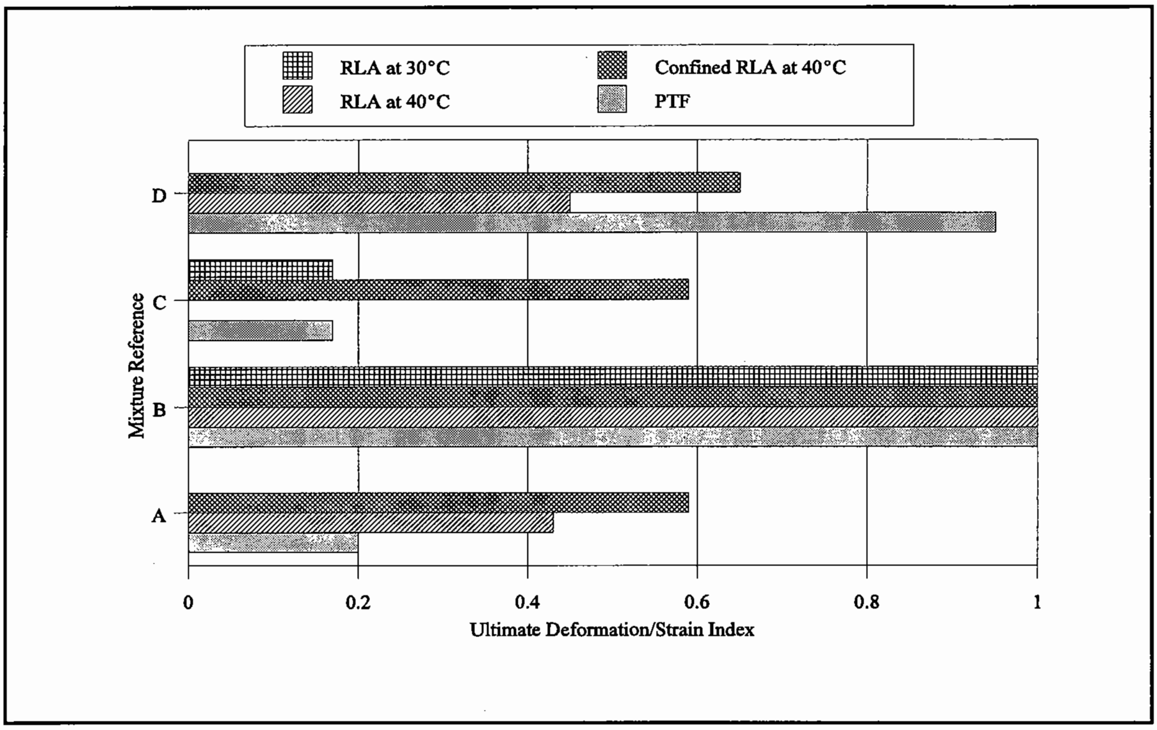


Figure 4.42 Performance Indices Based on Ultimate Deformation/Strain for PTF and RLA under all Conditions

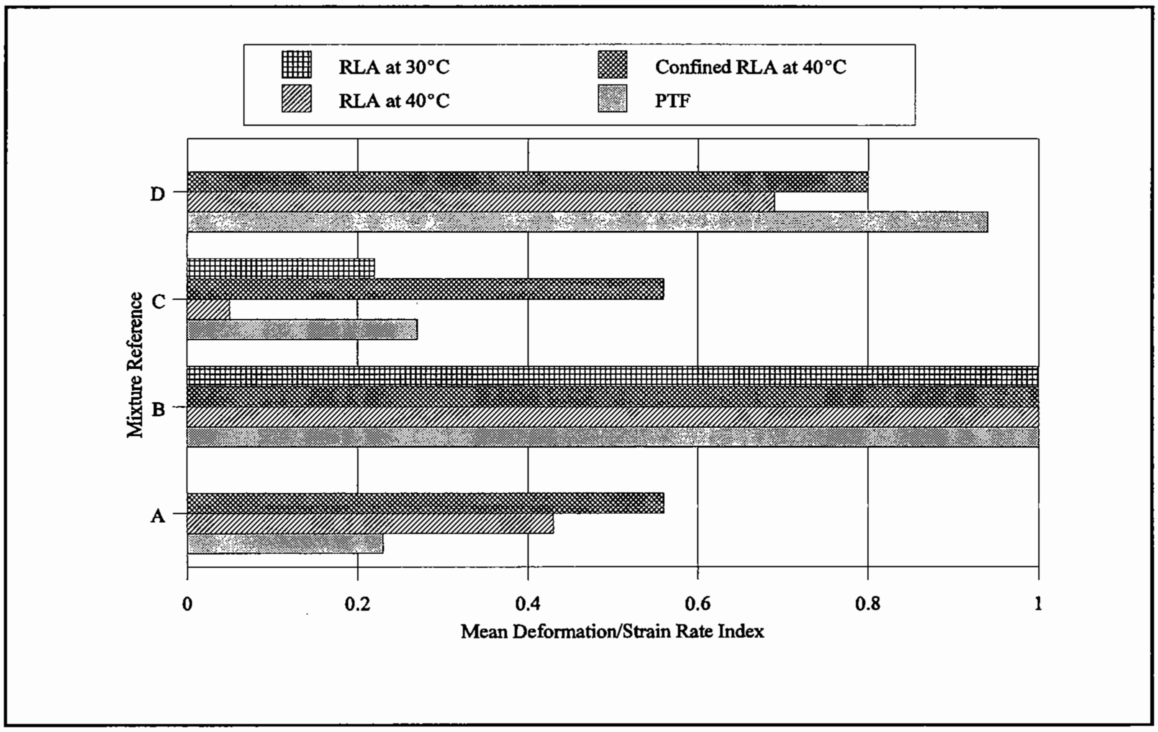


Figure 4.43 Performance Indices Based on Mean Deformation/Strain Rate for PTF and RLA under all Conditions

Uniaxial and triaxial tests carried out by Monismith and Tayebali (114) on an asphaltic concrete with Watsonville granite aggregate, as used in the work described in Chapter Three, demonstrated that higher strains occur under repeated loading than under creep when testing without confinement. This is consistent with the findings presented in Chapter Three and is due to plastic strains being induced in the continuous aggregate skeleton by repeated loading. However, their data shows that with a confining stress of approximately 200 kPa the strains ultimately achieved under creep and repeated loading are the same, though there was little replicate testing. At a confining pressure of approximately 100 kPa the repeated load strains were generally slightly higher than the creep strains. These results indicate that the effect of confinement is to restrict plastic strains in the aggregate structure so that, where the confining stress is high, only the component of permanent deformation due to viscous flow in the binder will develop, up to the point where "lock-up" occurs in the aggregate structure. This is consistent with the observations that confinement is most important for mixtures which rely on an aggregate skeleton for resistance to deformation (20,125), and may explain why the application of confinement had the greatest effect on the unmodified gravel mixture which, with smooth rounded aggregate, would likely have been most susceptible to plastic strains in the aggregate structure.

Interestingly, Monismith and Tayebali's data also shows that in confined tests where the ultimate strain due to creep and repeated loading is the same, the rate of strain is initially higher in the creep test. This may be explained by the fact that under creep conditions the binder stiffness will be low due to the long load duration and the viscous component of deformation will therefore develop more quickly.

4.14 Conclusions

This exercise has shown that the RLA test is able to discriminate effectively between different mixtures and the results reflected the superiority of the modified mixtures which was clearly evident in the testing of the trial pavement. However, mixture performance was significantly influenced by the test conditions. Moreover, in comparison with their relative performance in the PTF, as indicated by the mixture performance indices, the effect of changes in test conditions appeared to vary between the mixtures. The unconfined test at

40 °C showed up greater differences in the performance of the aggregates than was apparent in the PTF, with the performance of the unmodified gravel mixture being particularly poor. However, this material showed the greatest improvement in performance from the application of confinement and also benefited from a reduction in the temperature of the unconfined test. The improvement due to confinement was such that the performance of the unmodified gravel mixture was equal to that of the unmodified granite, which is more in line with the observations from the PTF.

This data suggests that the effect of RLA test conditions which are likely to restrict plastic strains in the aggregate structure, such as the application of confinement or reduced temperature which would give higher binder stiffness, may be of proportionately greater benefit for mixtures with poor aggregate. This would apply principally to mixtures which rely on the aggregate interlock mechanism for resistance to permanent deformation.

CHAPTER FIVE
THE ROLES OF AGGREGATE AND BINDER IN RESISTANCE TO PERMANENT DEFORMATION

5.1 Introduction

The findings of the work described in Chapter Four, which showed the dominance of the binder in the performance of the mixtures in the trial pavement and the varying degree of influence of test conditions on the different mixtures in the RLA test, highlighted the need for a more fundamental investigation of the roles of aggregate and binder in resistance to permanent deformation. Two separate test programmes were, therefore, devised and carried out. The first was designed to provide a qualitative assessment of the effects of binder stiffness and mode of loading, static or repeated, from tests on a simple aggregate structure. In an attempt to isolate the effects of binder and aggregate, tests were performed on unbound specimens which necessitated the use of triaxial apparatus for this investigation. A description of the equipment used is given in Appendix C. The second test programme was intended to assess the effect of binder stiffness on different aggregate structures and this was carried out in the NAT using only repeated loading without confinement.

5.2 Triaxial Test Programme

5.2.1 Selection of Materials

Aggregates : Single-sized rather than graded aggregates were used to minimise the number of components, and potential variables, in the system in order that the effect of the introduction of a bituminous binder could be directly assessed. The intention was to use two types of aggregate; one smooth and rounded, the other rugous and angular, in the expectation of obtaining as wide a variation in performance as possible. Nominally spherical ceramic pebbles, manufactured for application in chemical process catalysation, were used as the "poor" aggregate. A crushed gritstone was selected as the "good" aggregate. Information obtained from the suppliers indicated the water absorption of both aggregates to be 0.4%

All aggregates were washed and then dried and sieved prior to use. The ceramic pebbles were supplied as nominally 6mm in diameter and these were screened on a 6.3mm sieve to remove any undersize, resulting in a very uniform material. The gritstone was initially sieved to obtain material passing 10mm and retained on a 6.3mm aperture. However, the particle size of this material was observed to be generally much larger than that of the ceramic pebbles and the absence of an intermediate sieve in this range prevented closer control of the particle size. It was, therefore, decided to use the gritstone fraction which passed the 6.3mm sieve but was retained on the 4.75mm mesh. This produced a very uniform material with a particle size more similar to that of the ceramic pebbles. Samples of both aggregates are shown in Figure 5.1.

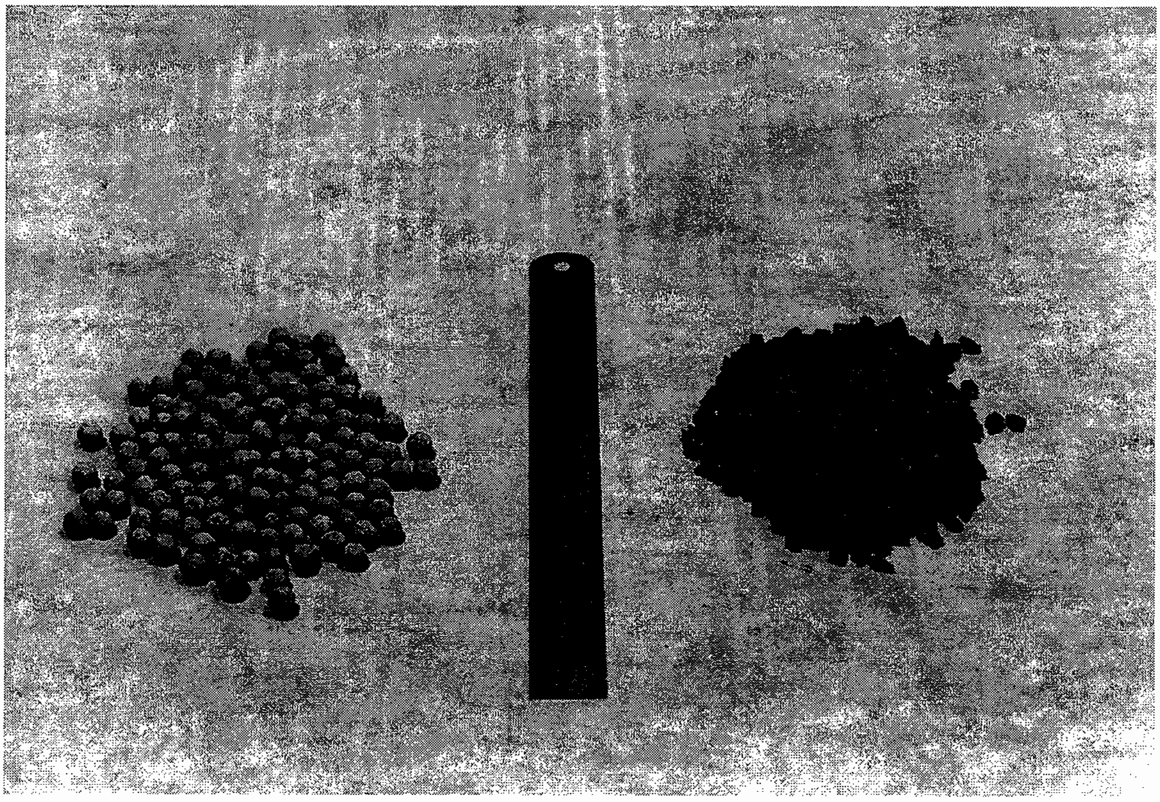


Figure 5.1 Ceramic (left) and Gritstone (right) Aggregates

Binder : The triaxial testing apparatus used for this test programme was housed in an air conditioned laboratory which enabled a constant temperature of 22 °C to be maintained in the test environment. There was, however, no facility to vary the temperature of the specimen so different grades of binder were used to allow the effect of different binder stiffnesses to be evaluated. The binders were a 50 Pen and a 200 Pen refined from the same, carefully selected, Venezuelan crude source and these were shown by rheometric tests to have the same temperature-stiffness relationship, as demonstrated in Figure 5.2. Substitution of the 200 Pen for the 50 Pen would, therefore, be equivalent to a temperature increase of about 15°C.

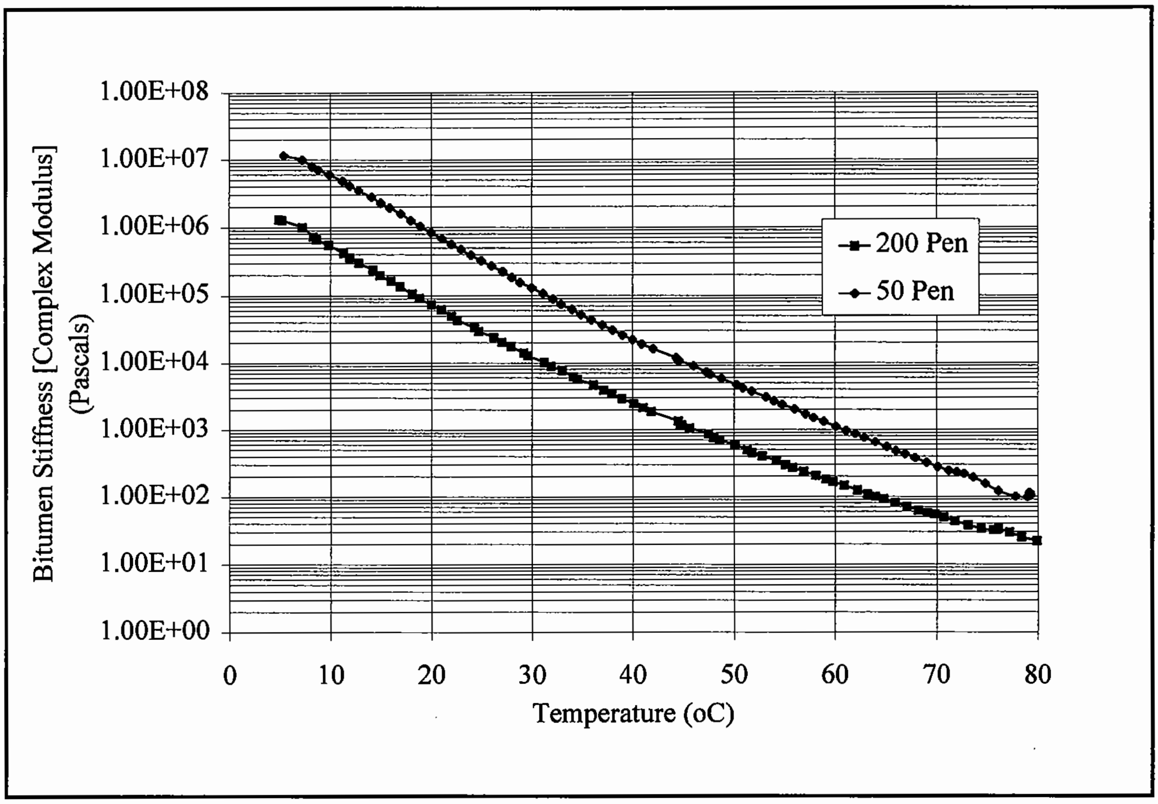


Figure 5.2 Temperature - Log Stiffness Relationships for 50 and 200 Pen Bitumens Determined from Rheometric Testing at 0.5 Hz

5.2.2 Specimen Manufacture

Unbound Specimens : All specimens for the triaxial testing were 75mm in diameter. Because the unbound specimens had no cohesion, the material had to be placed and compacted inside an impermeable membrane, within a steel split mould, in order that a confining partial vacuum could be applied to the specimen before the mould was released. Attempts to compact the unbound material with a PRD hammer proved unsuccessful as this caused the aggregate particles to rupture the membrane making application of the partial vacuum impossible. For this reason, the unbound specimens were prepared by placing the mould on a vibrating table while the material was added and compacted under a small surcharge.

Bound Specimens : Trial mixing of the ceramic pebbles with bitumen revealed that they would not coat adequately or retain cohesion on cooling without the addition of limestone filler to the mixture. Experimentation showed that the minimum viable filler content was 4% by mass of mixture. No such problems were encountered with the gritstone but 4% filler was added to these mixtures to ensure consistency.

Trials were also carried out to establish the binder contents to be used in the test programme since this was one of the variables to be considered. The ceramic pebbles were found to be relatively intolerant of variations in binder content, not being able to retain much more than the minimum required for coating without drainage. This situation was, however, improved by the addition of the limestone filler and it proved possible to produce satisfactory mixtures at binder contents of 1.5% and 3.0% by mass of mixture, which gave an acceptable range for this variable. These values are much lower than the 8% by mass of mixture reportedly used by Lee and Dass (127) in mixtures of bitumen and 6mm glass beads during investigations into methods of characterising packing structures for micromechanical modelling of granular materials. The gritstone was much more tolerant of variations in the amount of bitumen present in the mixture and the binder contents for this material were determined to give the same volume of binder as in the ceramic pebble mixtures. This gave low and high target binder contents of 1.4% and 2.7% by mass of mixture, respectively. Mixing was carried out by hand since the amount of material required for each specimen was small.

Use of a vibrating table for compaction of the bound specimens was not feasible because of the level of compactive effort required. The potential for differences in the deformation resistance in the bound and unbound specimens due to the use of different methods of compaction (see Chapter Seven) was minimal because the use of single-sized materials gave a very simple aggregate structure, particularly in the case of the nominally spherical ceramic aggregate. However, in an effort to make the method of compaction of the bound specimens as close to that of the unbound specimens as could reasonably be achieved, a frame was constructed from which a PRD hammer was suspended on a pivoted arm, as shown in Figure 5.3. This ensured that the hammer, which would impart a vibrating action, remained vertical during compaction and allowed its mass to be counterbalanced to reduce the effective surcharge. All specimens were compacted in specially manufactured split moulds, illustrated in Figure 5.4, which could be jacked apart to release the specimen without damage. Care had to be taken to ensure that the specimen ends were flat and normal to the major axis as these mixtures would not withstand trimming after compaction and cooling. A typical gritstone specimen is shown in Figure 5.5.

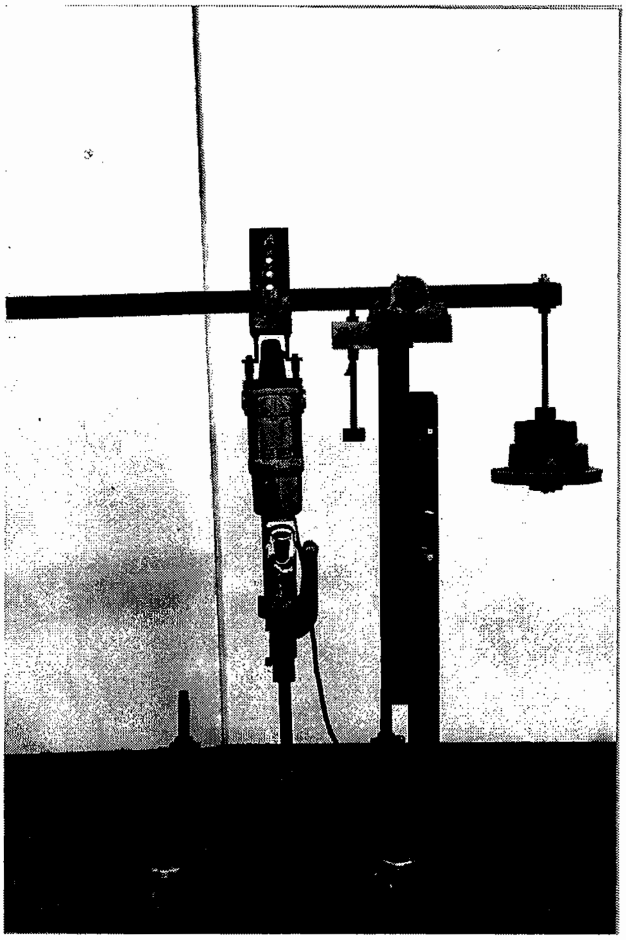


Figure 5.3 PRD Hammer Mounted on Frame for Compaction of Bound Specimens

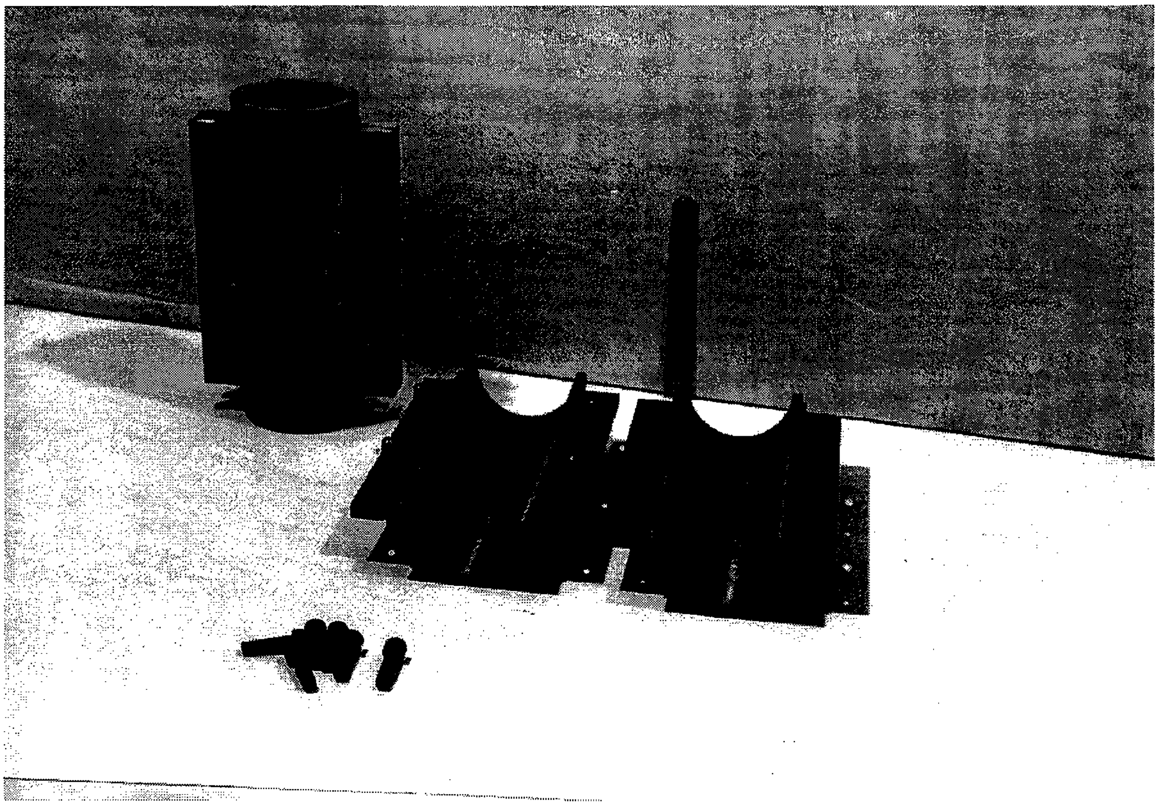


Figure 5.4 Split Moulds

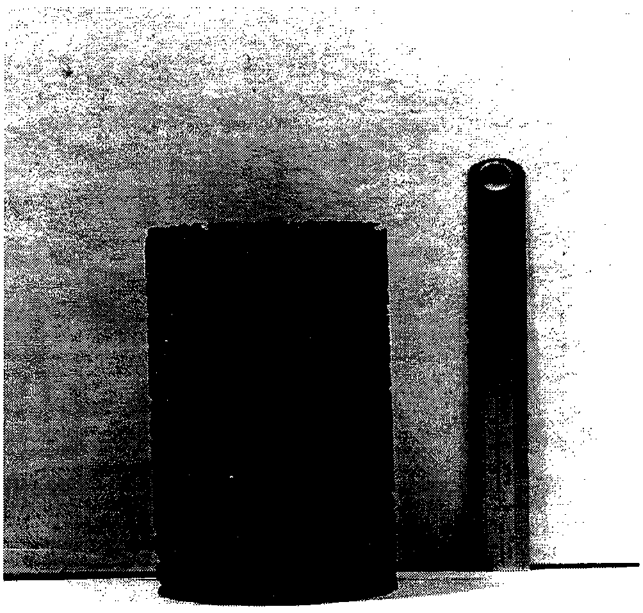


Figure 5.5 Bound Gritstone Specimen

Equiviscous temperatures used in the mixing and compaction of the bound specimens were determined from initial binder properties. A principal concern in the manufacture of specimens was to obtain similar values of VMA in the bound and unbound specimens for both aggregate types. The mean values of VMA and their standard deviations presented in Table 5.1 show that this was effectively achieved and indicate that the overall structure of the aggregate was similar in all specimens. Determination of specimen volume for the calculation of VMA was by mensuration since this was the only feasible means for the unbound specimens. Also, the high permeability of the bound specimens meant that they would have to be sealed for measurement by immersion but the rugosity of the surface would probably have caused significant error if an adhesive film or tape had been used, and the use of wax was not viable because of the difficulties of removing it satisfactorily from specimens to be subjected to mechanical testing. Maximum theoretical densities of the bound specimens were determined by the Rice method.

Table 5.1 Volumetric Data

Specimen Group	VMA (%)	
	Mean	Standard deviation
Unbound ceramic	42.1	0.15
Unbound gritstone	41.6	0.09
Bound ceramic	41.2	0.3
Bound gritstone	42.3	1.13

5.2.3 Test Programme

In order to establish the loading regime to be used throughout the test programme, a preliminary series of tests was carried out on the unbound ceramic pebbles since these were likely to be the weakest of the materials to be tested. Confinement was applied by pressurising the triaxial cell with compressed air to give a static confining pressure of 70kPa, which was approximately equal to that applied by partial vacuum during preparation of the unbound specimens. Ramped load tests on the ceramic pebbles showed the axial stress required to cause failure under this confinement to be approximately 250 kPa. Further tests in which a square load pulse of one second duration was applied repeatedly with intervals of one second between pulses, as used in the RLA test, showed that an applied axial stress, or deviator stress, of 230 kPa (92% of the static failure stress) was required to produce significant permanent deformation in the specimen without causing failure. The test conditions used throughout the programme are given in Table 5.2.

Table 5.2 Test Conditions for Triaxial Test Programme

	Static Test	Repeated Load Test
Temperature	22°C	22°C
Confining Stress	70 kPa (static)	70 kPa (static)
Deviator Stress	230 kPa	230 kPa
Mode/Duration of Loading	Constant load applied for 3600 seconds followed by no-load recovery period	Square load pulse, 0.5 Hz, applied for 3601 cycles

No pre-test conditioning load was applied since the cell pressure should have been adequate to ensure that the load platens were correctly seated on the specimen. The variables evaluated in the test programme are summarised below:

Aggregate type	:	Ceramic pebbles, gritstone
Binder content	:	None, low, high
Binder grade	:	50 Pen, 200 Pen
Mode of loading	:	Static, repeated

At least three replicate specimens were manufactured and tested for each test condition. Static tests were not carried out on the bound specimens with 200 Pen binder.

Permanent deformation was monitored by means of a pair of LVDT's mounted off the sample in a configuration similar in principle to that used in the RLA test. In the case of the repeated load tests on the bound specimens, the LVDT signals were recorded at the end of the one second "load-off" period to allow maximum recovery to take place before making the measurement. This is illustrated schematically in Figure 5.6. The state of the specimen when strains were recorded was, therefore, different to that in the static test where measurements were made while load was applied, except during the recovery period at the end of the test, and this should be borne in mind in analysis of the data plots.

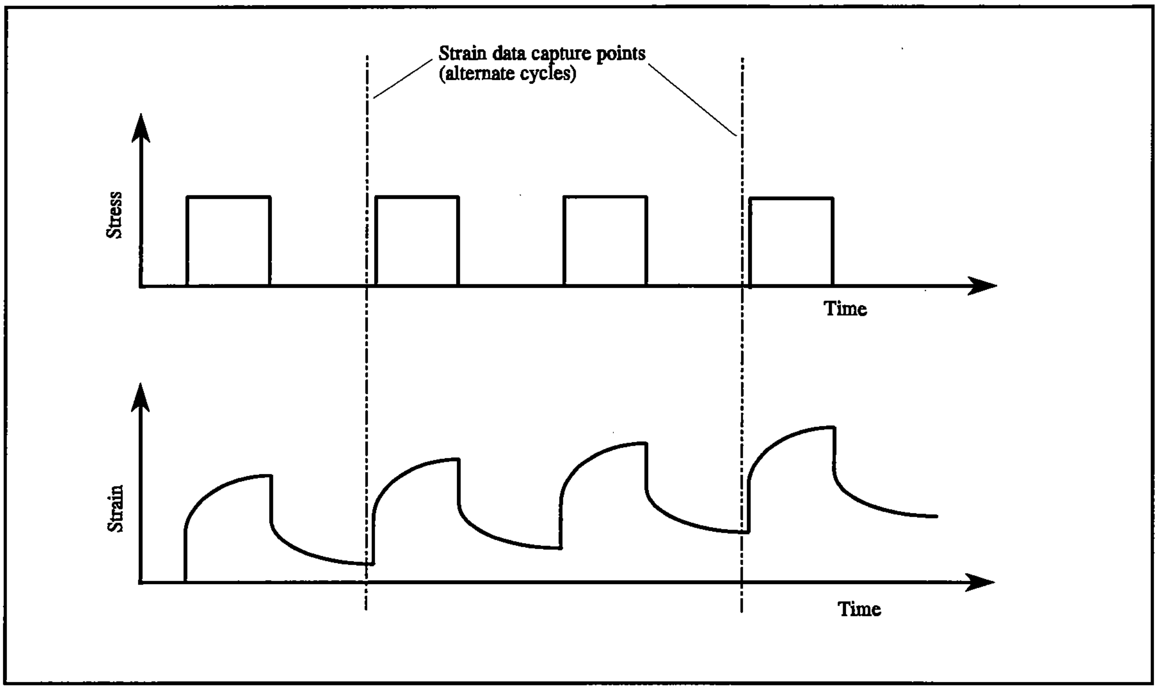


Figure 5.6 Regime for Recording of Deformation in Repeated Load Tests

5.2.4 Results of Tests on Unbound Specimens

The data obtained from the static load tests on the unbound ceramic pebbles are presented graphically in Figure 5.7 while Figure 5.8 shows the data from the repeated load tests on that material. In the static tests the material strained instantly on the application of load to a level which remained constant until the load was removed, whereupon there was some instantaneous recovery though a degree of permanent, plastic strain remained. In the repeated load tests, however, permanent strain accumulated throughout the test, indicating the continued development of plastic deformation in the aggregate under this form of loading. The magnitude of the permanent strain in the specimens at the end of the repeated load tests was generally greater than that in the static tests after removal of the load. Similar behaviour is shown in Figures 5.9 and 5.10 for the tests on the unbound gritstone though the magnitude of the recorded strains was lower, reflecting the superior performance of the coarse, angular aggregate.

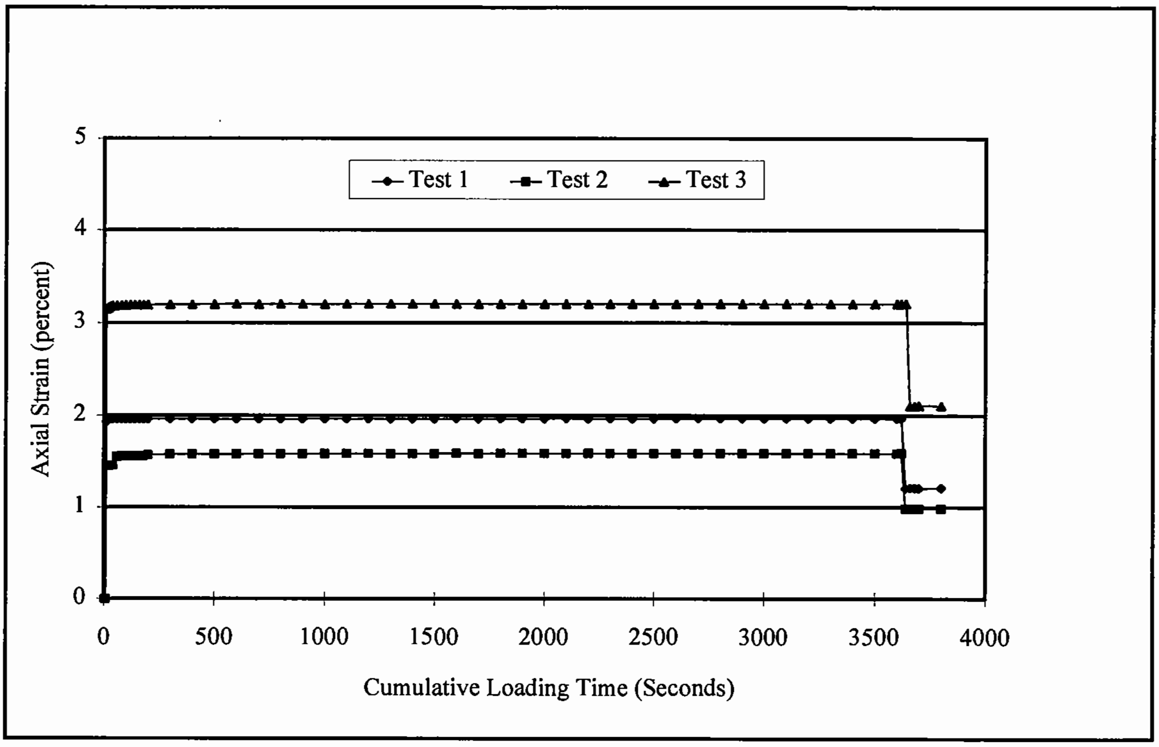


Figure 5.7 Unbound Ceramic Aggregate : Static Test Plots

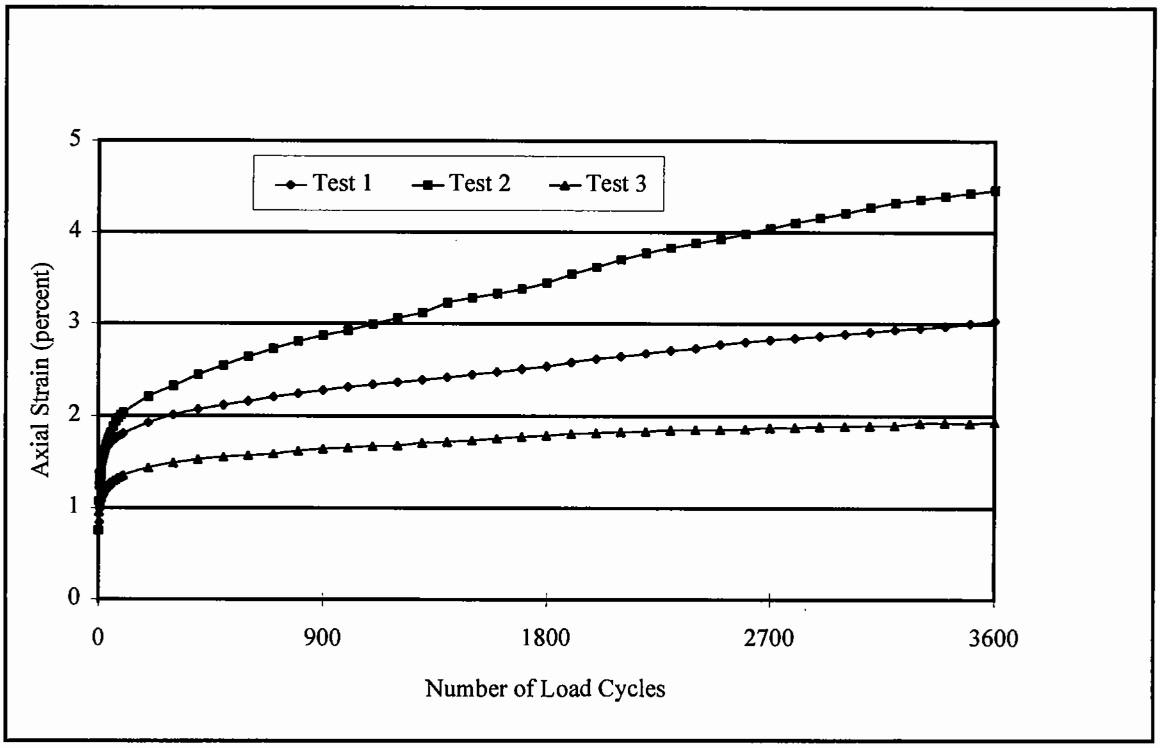


Figure 5.8 Unbound Ceramic Aggregate : Repeated Load Test Plots

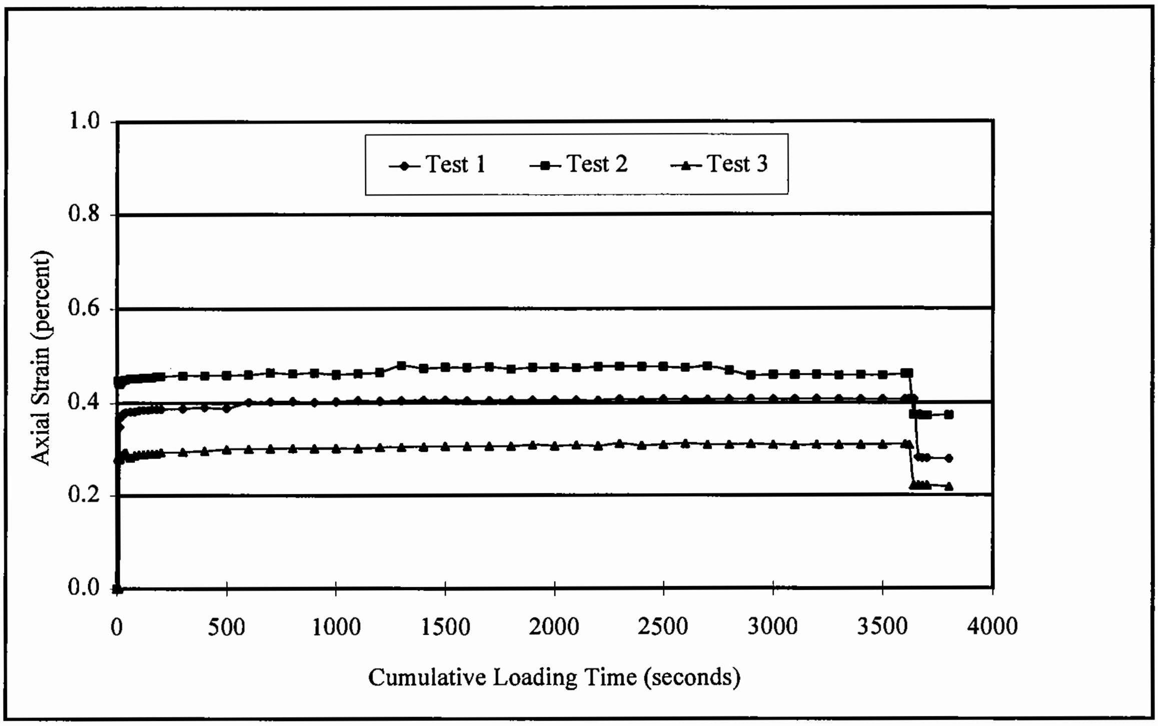


Figure 5.9 Unbound Gritstone : Static Test Plots

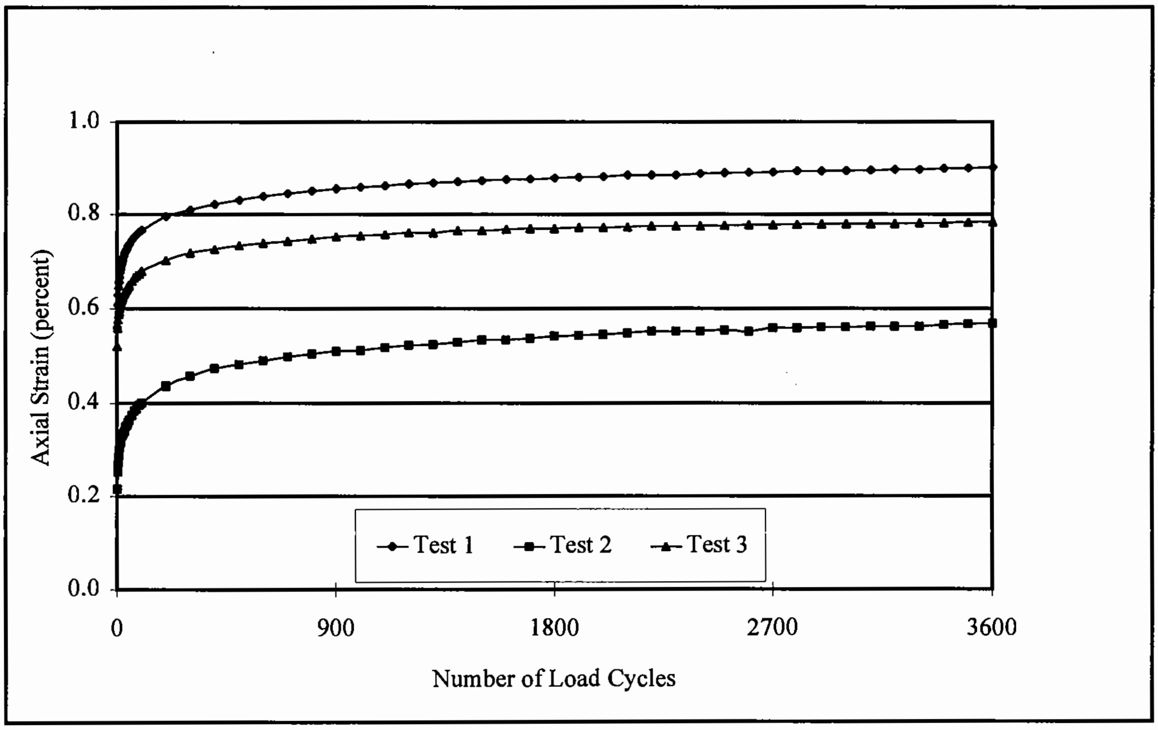


Figure 5.10 Unbound Gritstone : Repeated Load Test Plots

5.2.5 Results of Tests on Bound Specimens

Ceramic Pebble Mixtures : The data plots for the repeated load tests on bound ceramic aggregate specimens, Figures 5.11 to 5.14, show an accumulation of permanent strain throughout the test, as was observed in the unbound specimens. However, though there is some variability in the data, particularly for the 200 Pen high binder content specimens, the results indicate that only the 50 Pen high binder content condition gave clearly and consistently better performance than the unbound material under repeated loading. The specimens with the 200 Pen binder gave generally worse results (Figures 5.11 and 5.12) than the unbound material, possibly indicating that the effect of this soft binder was to lubricate rather than restrain the aggregate particles. The effect of the 50 Pen binder at low content (Figure 5.13) is less clear but the addition of a higher amount of this stiff bitumen improved the resistance of the ceramic aggregate to deformation under repeated loading (Figure 5.14).

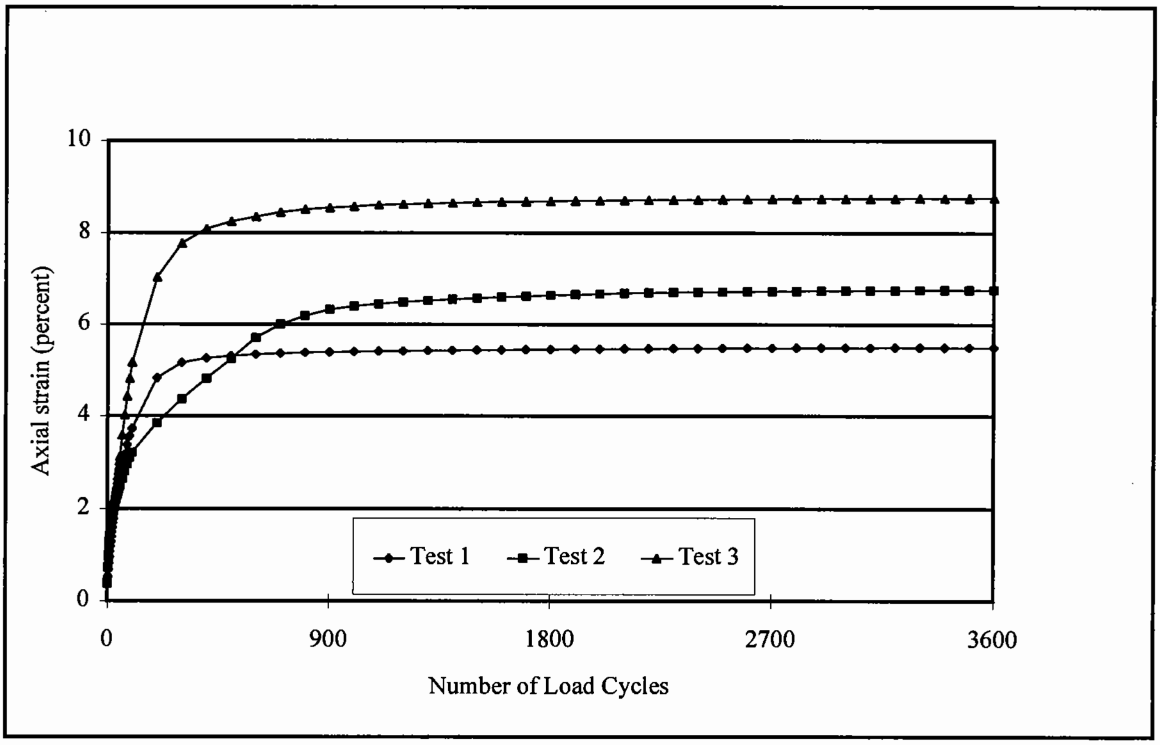


Figure 5.11 Repeated Load Tests : Ceramic Aggregate 200 Pen Binder Low Binder Content

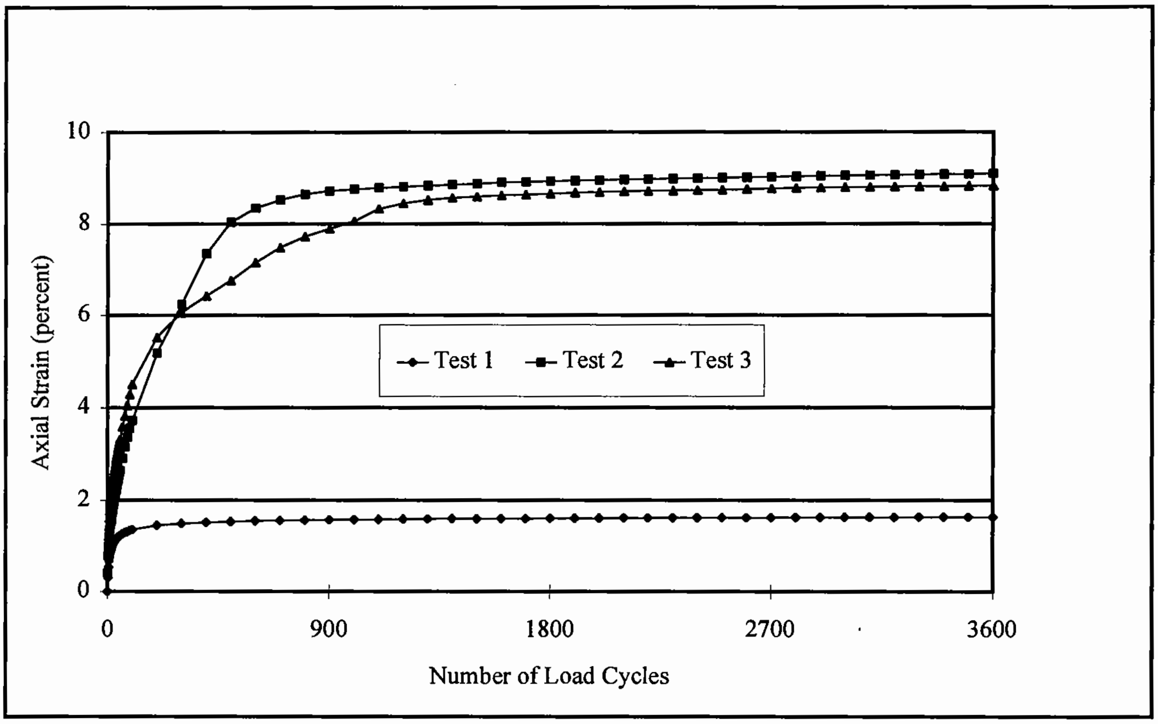


Figure 5.12 Repeated Load Tests : Ceramic Aggregate 200 Pen Binder High Binder Content

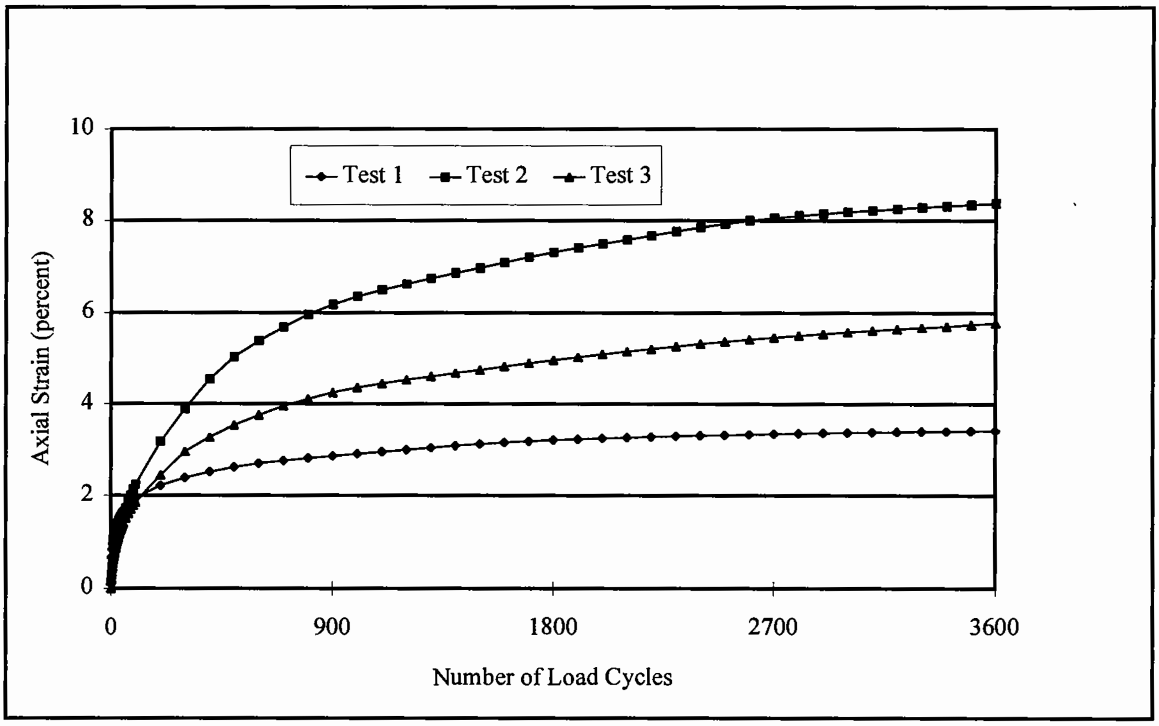


Figure 5.13 Repeated Load Tests : Ceramic Aggregate 50 Pen Binder Low Binder Content

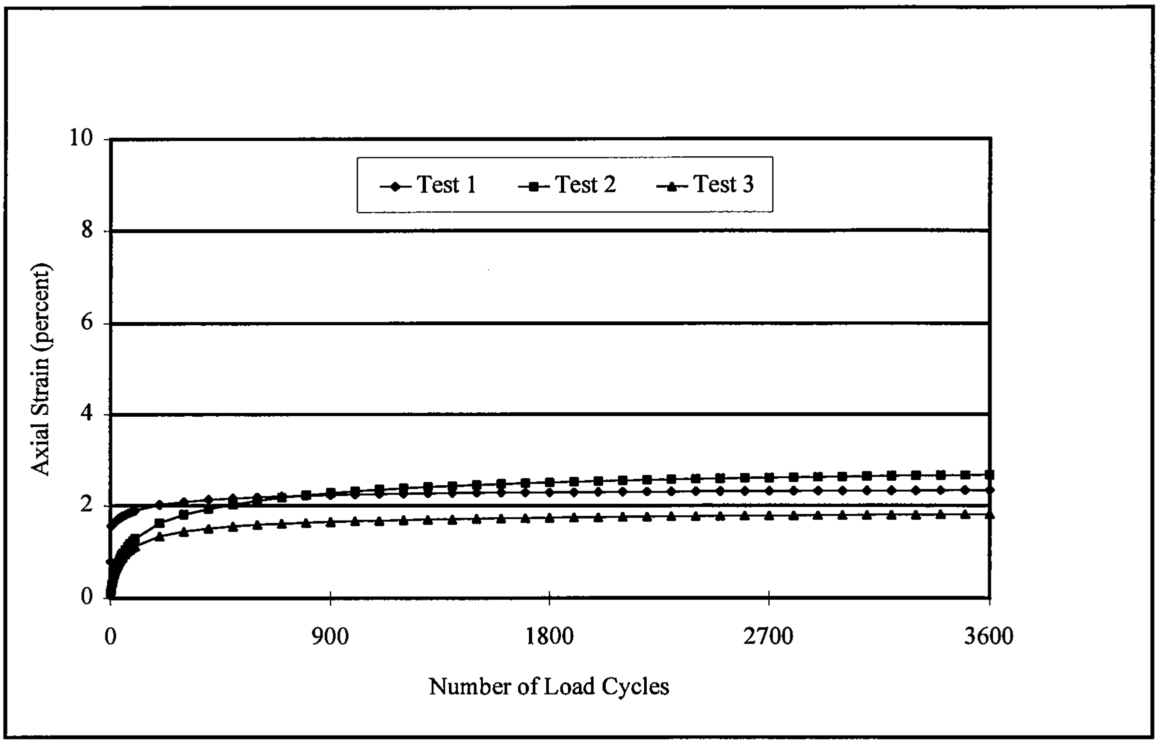


Figure 5.14 Repeated Load Tests : Ceramic Aggregate 50 Pen Binder High Binder Content

The data plots for the static tests on the mixtures with 50 Pen binder, Figures 5.15 and 5.16, show that failure occurred in all specimens at both low and high binder contents. This is in marked contrast to the response of the unbound material under static loading where there was no further development of strain beyond that which was induced immediately on application of the load. It would appear that under the long loading time of the static test the binder stiffness is too low to contribute positively to deformation resistance and the bitumen has simply acted as a lubricant. The high binder content specimens generally failed more quickly than those with low binder content, presumably because the increased volume of binder provided greater lubrication.

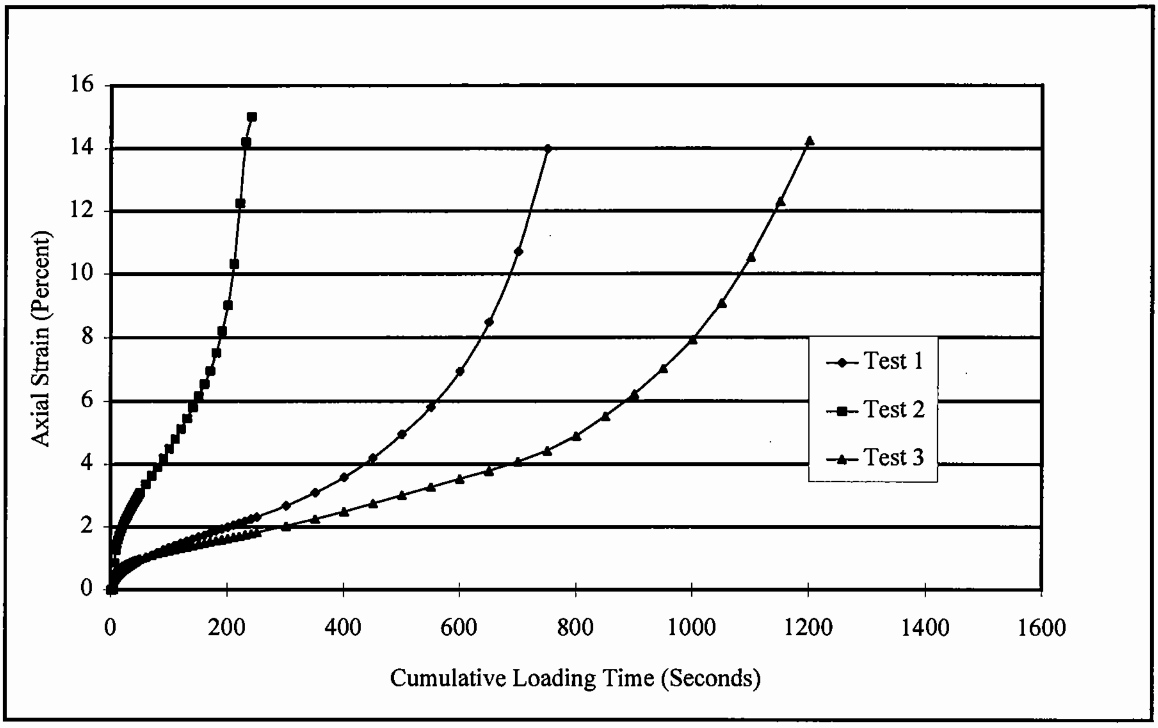


Figure 5.15 Static Tests : Ceramic Aggregate 50 Pen Binder Low Binder Content

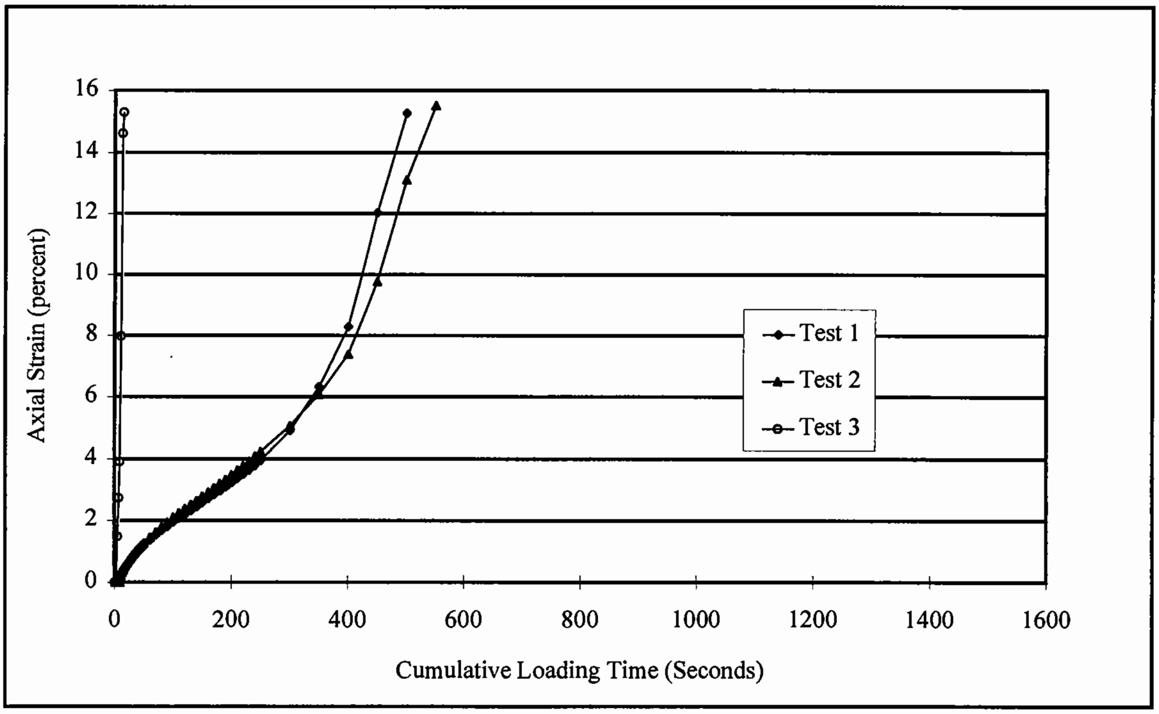


Figure 5.16 Static Tests : Ceramic Aggregate 50 Pen Binder High Binder Content

The behaviour of the bound material illustrated in the test data plots is reflected in the average values of mean strain rate, which are presented in Table 5.3. Mean strain rate was calculated in the same manner as described in Chapter Four, but the units for the values in Table 5.3 are microstrain/second and microstrain/cycle for the static and repeated load tests respectively.

Table 5.3 Mean Strain Rates for Bituminous Mixtures with Ceramic Aggregate

Mode of Loading	200 Pen Binder		50 Pen Binder	
	Low M_B	High M_B	Low M_B	High M_B
Static (μstrain/second)	-	-	312	4126
Repeated (μstrain/cycle)	18	17	16	5

Gritstone Mixtures : The key observation to be made on the performance of the gritstone mixtures under repeated loading is that there is no clear effect of either the grade of binder or binder content, as demonstrated by the data presented in Figures 5.17 to 5.20. There is some variation in the data but, overall, the performance is similar to that of the unbound gritstone under the same loading conditions, and is significantly better than that of the ceramic aggregate.

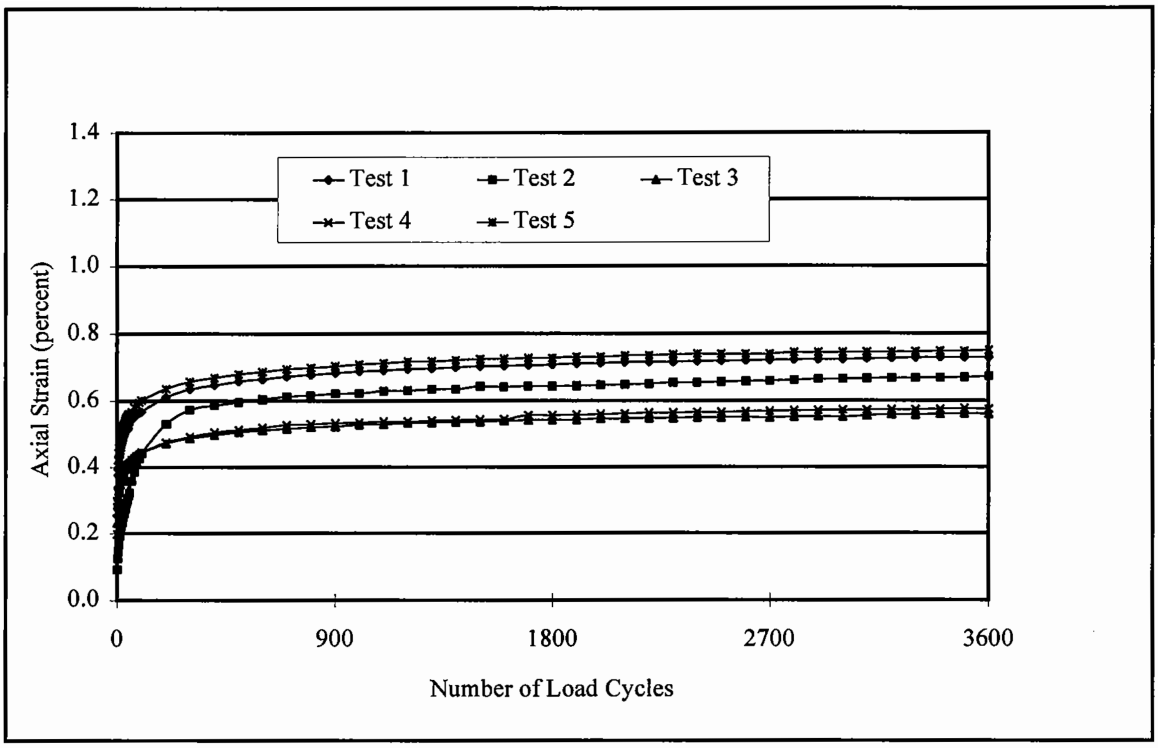


Figure 5.17 Repeated Load Tests : Gritstone 200 Pen Binder Low Binder Content

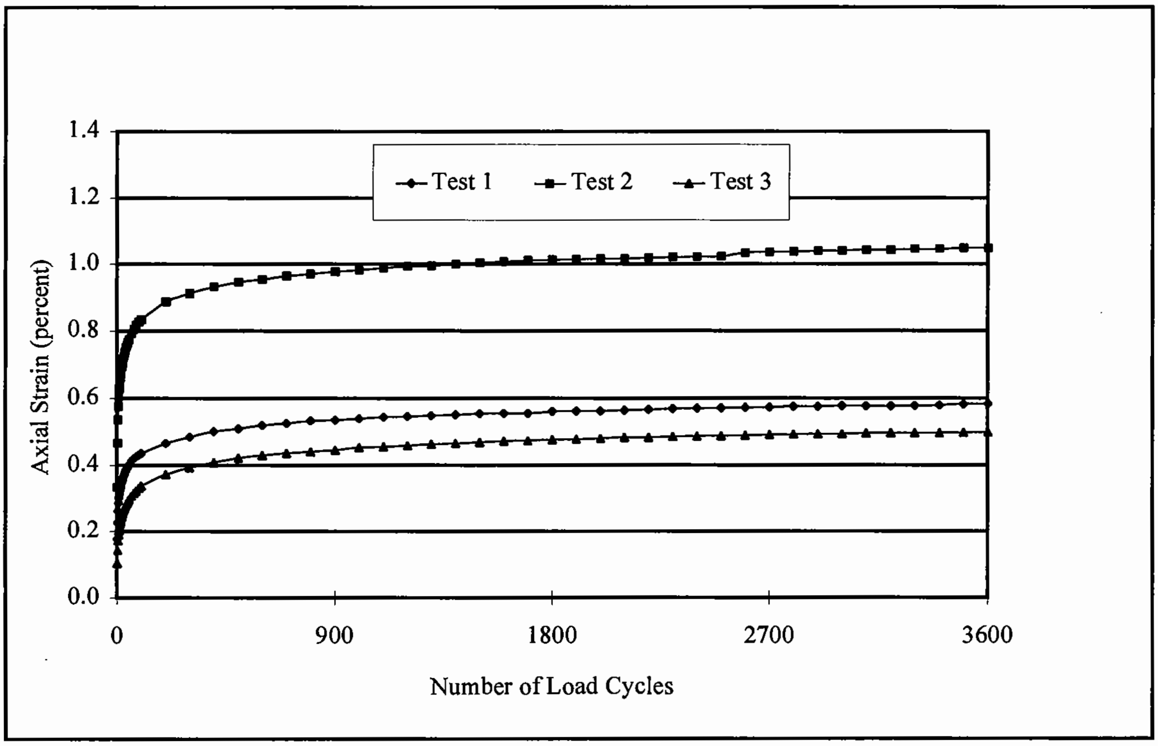


Figure 5.18 Repeated Load Tests : Gritstone 200 Pen Binder High Binder Content

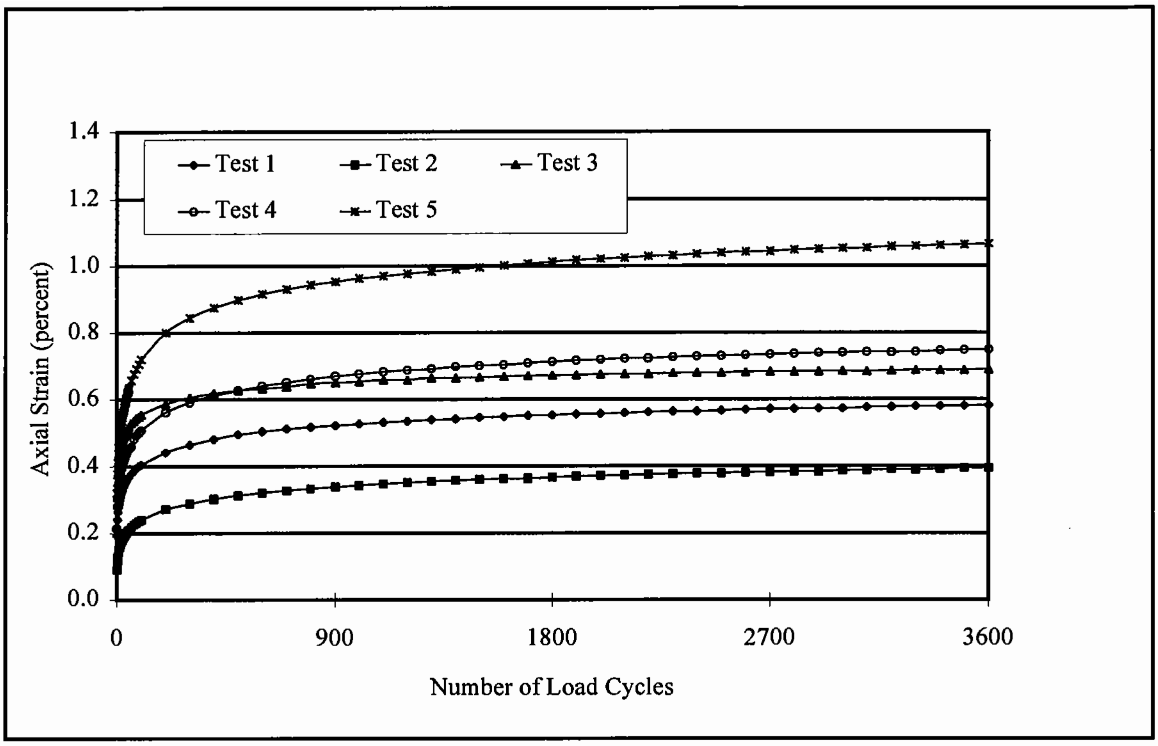


Figure 5.19 Repeated Load Tests : Gritstone 50 Pen Binder Low Binder Content

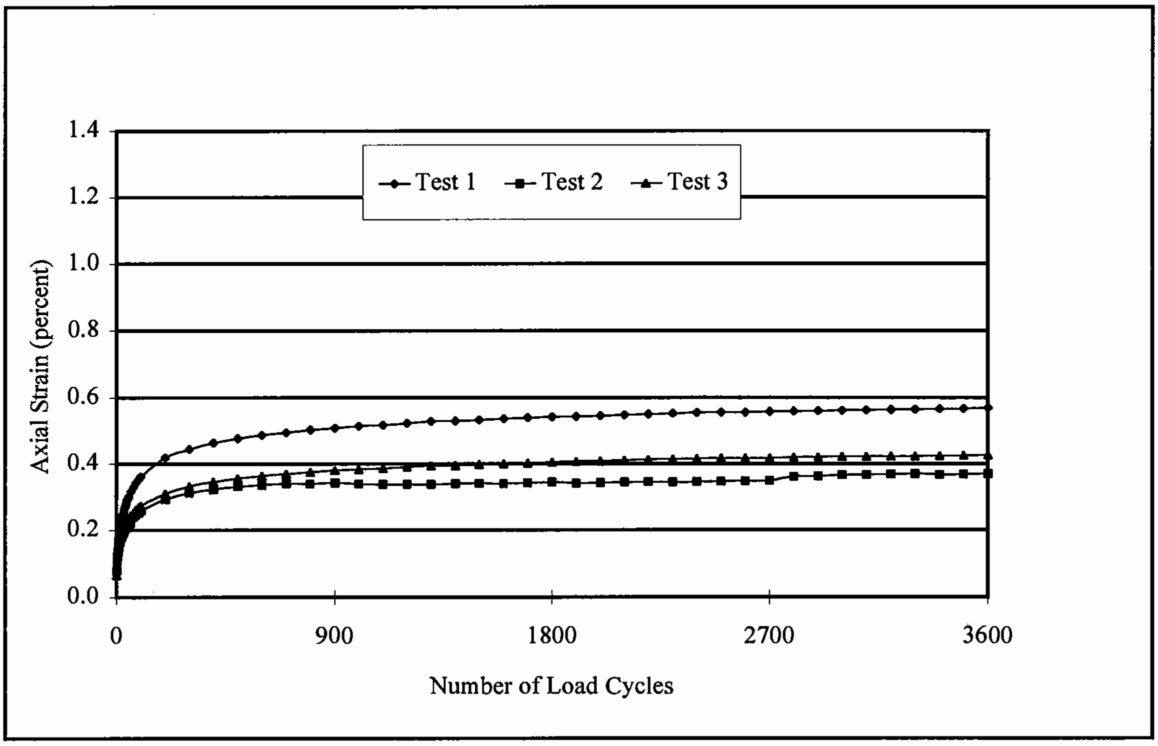


Figure 5.20 Repeated Load Tests : Gritstone 50 Pen Binder High Binder Content

Unlike the ceramic aggregate, neither the low nor high binder content gritstone mixtures failed under static loading, as can be seen from the data plots presented Figures 5.21 and 5.22. These plots do, however, show strain increasing with loading time up to the point where the load was removed, an effect which was not apparent in the static tests on the unbound gritstone and which can be attributed to viscous and visco-elastic deformation in the binder. Both slightly higher magnitudes and rates of strain are evident in the high binder content specimens which, having a greater volume of binder, would have increased potential for such deformation. These specimens also showed more recovery on removal of load indicating greater, though recoverable, strain in the binder. However, it would appear that despite low binder stiffness resulting from the long load duration, any lubricating effect of the bitumen which caused failure in the ceramic pebble mixtures was not sufficient to negate the interparticle friction of this coarse, angular aggregate.

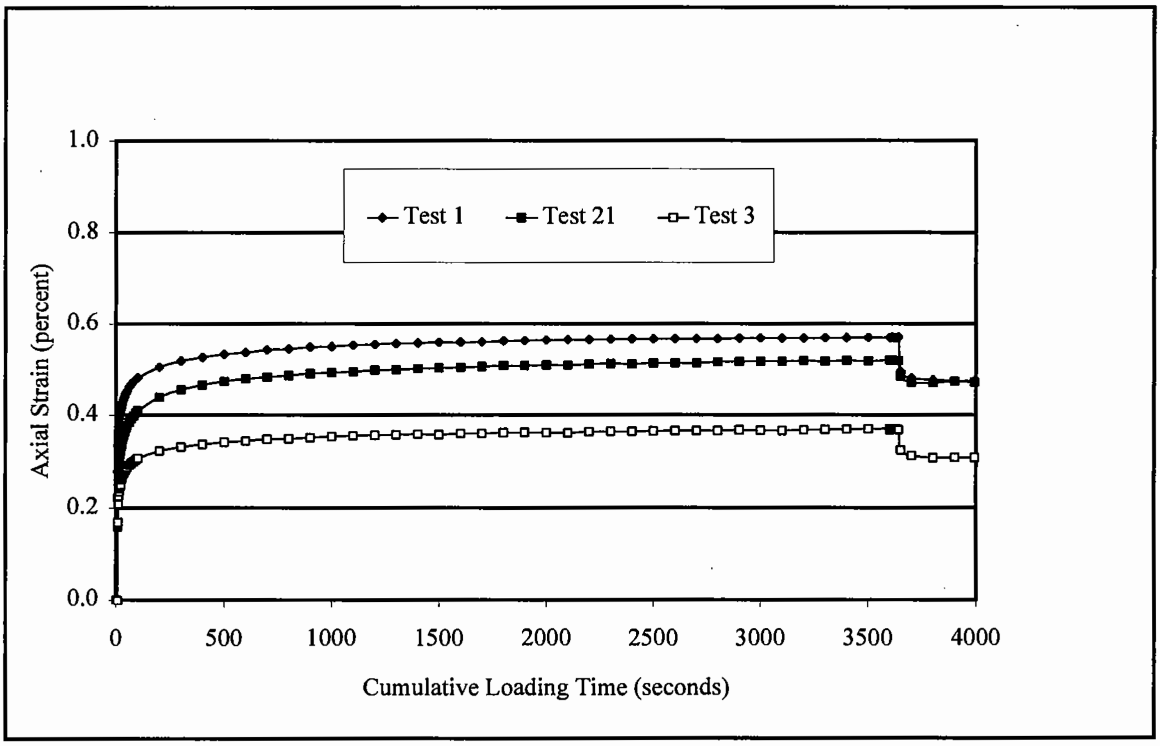


Figure 5.21 Static Tests : Gritstone 50 Pen Binder Low Binder Content

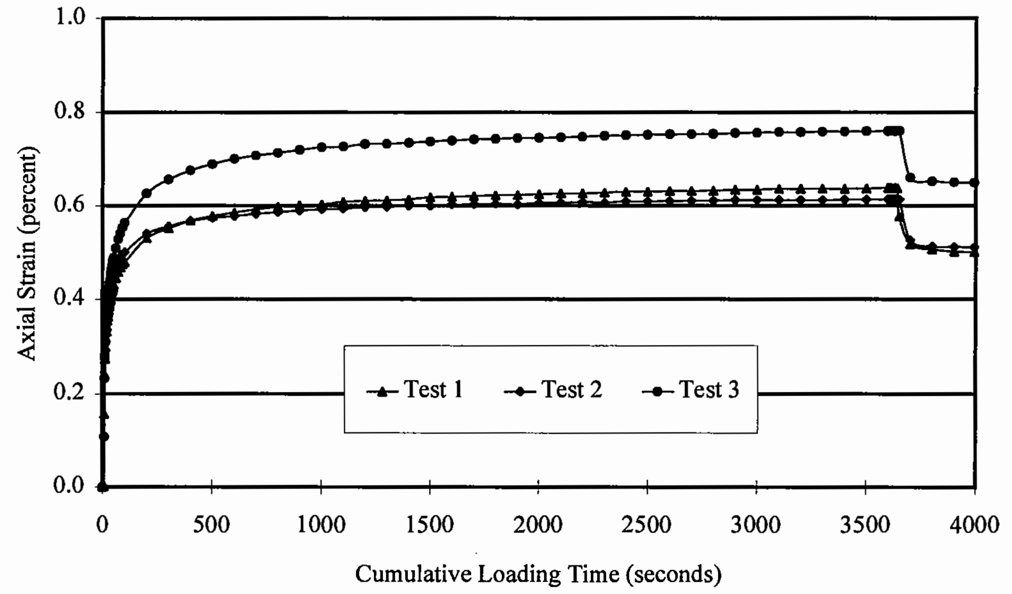


Figure 5.22 Static Tests : Gritstone 50 Pen Binder High Binder Content

Mean strain rates from the test programme on the bituminous mixtures with gritstone aggregate are presented in Table 5.4. These reinforce the analysis of the data plots in showing little effect of binder content or grade in the repeated load tests and indicating marginally poorer performance from the static test at high binder content. The values are in all cases much better than those obtained for the mixtures with the ceramic aggregate.

Table 5.4 Mean Strain Rates for Bituminous Mixtures with Gritstone Aggregate

Mode of Loading	200 Pen Binder		50 Pen Binder	
	Low M_B	High M_B	Low M_B	High M_B
Static (μstrain/second)	-	-	1.4	1.9
Repeated (μstrain/cycle)	1.2	1.4	1.4	1.1

5.3 Repeated Load Axial Test Programme

5.3.1 Aim of the Investigation

A preliminary investigation, carried out on plant-mixed 20mm DBM basecourse obtained from a limestone quarry, had produced the expected result that temperature does have an effect on performance in the RLA test. Specimens were obtained by drilling 150mm diameter cores from slabs placed within steel formwork and compacted using a twin drum vibratory pedestrian roller (see Chapter Seven). The data plots from tests at 30°C and 40°C on mixtures containing 200 Pen and 100 Pen binder, Figures 5.23 and 5.24 respectively, show significantly worse resistance to deformation at the higher temperature.

The principal effect of increasing the test temperature is to reduce the stiffness of the binder. In an attempt to determine whether change in performance of a mixture with test temperature could be entirely explained by the consequent change in the response of the binder a more detailed experiment was devised.

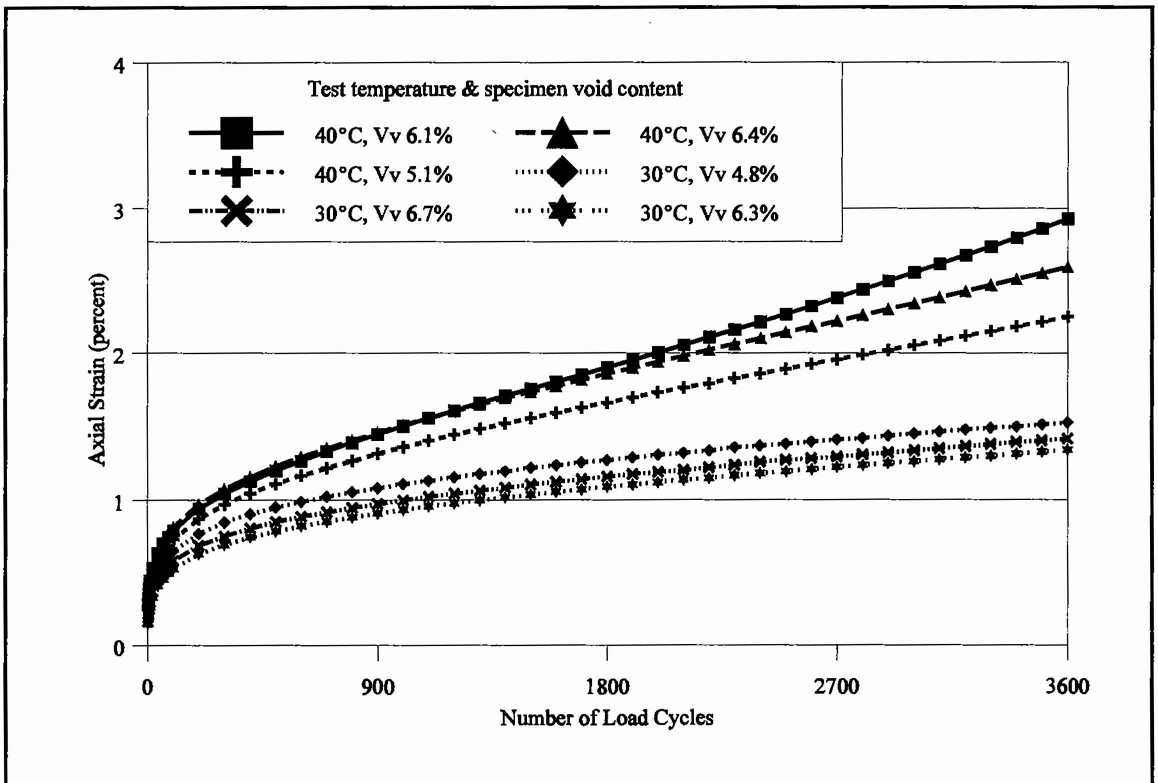


Figure 5.23 Effect of Test Temperature : 200 Pen 20mm DBM Basecourse

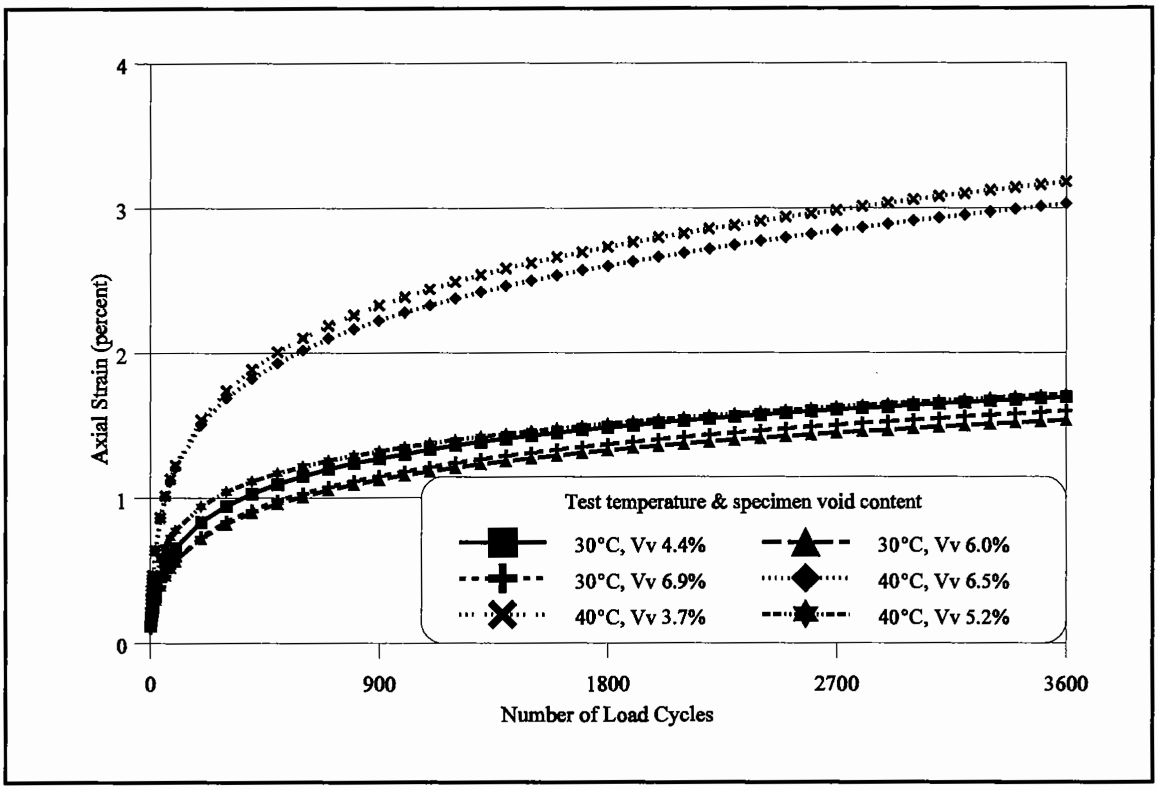


Figure 5.24 Effect of Test Temperature : 100 Pen 20mm DBM Basecourse

5.3.2 Selection of Materials

Two mixture types were chosen ; one continuously graded and one gap graded. The continuously graded material was a 20mm DBM basecourse, while the gap graded mixture was a 30/14 hot rolled asphalt (HRA) wearing course, selected because the 30% stone content should be insufficient to provide significant contact between coarse aggregate particles (154). The two materials, therefore, had very different aggregate structures as can be seen from the aggregate grading curves presented in Figures 5.25 and 5.26. To ensure that differences in the aggregate fraction were confined to the structure of the material the same sources of coarse aggregate - a gritstone as used in the triaxial test programme - fine aggregate and filler were used for both mixture types.

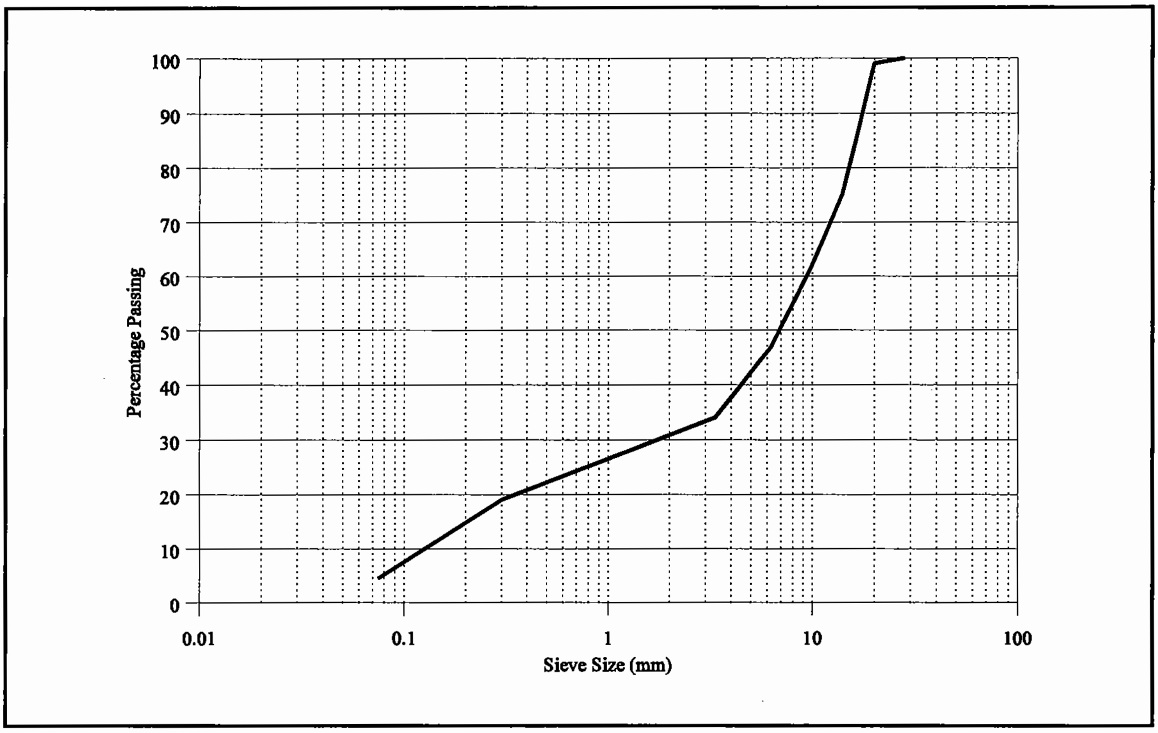


Figure 5.25 Target Grading Curve for the 20mm DBM Basecourse

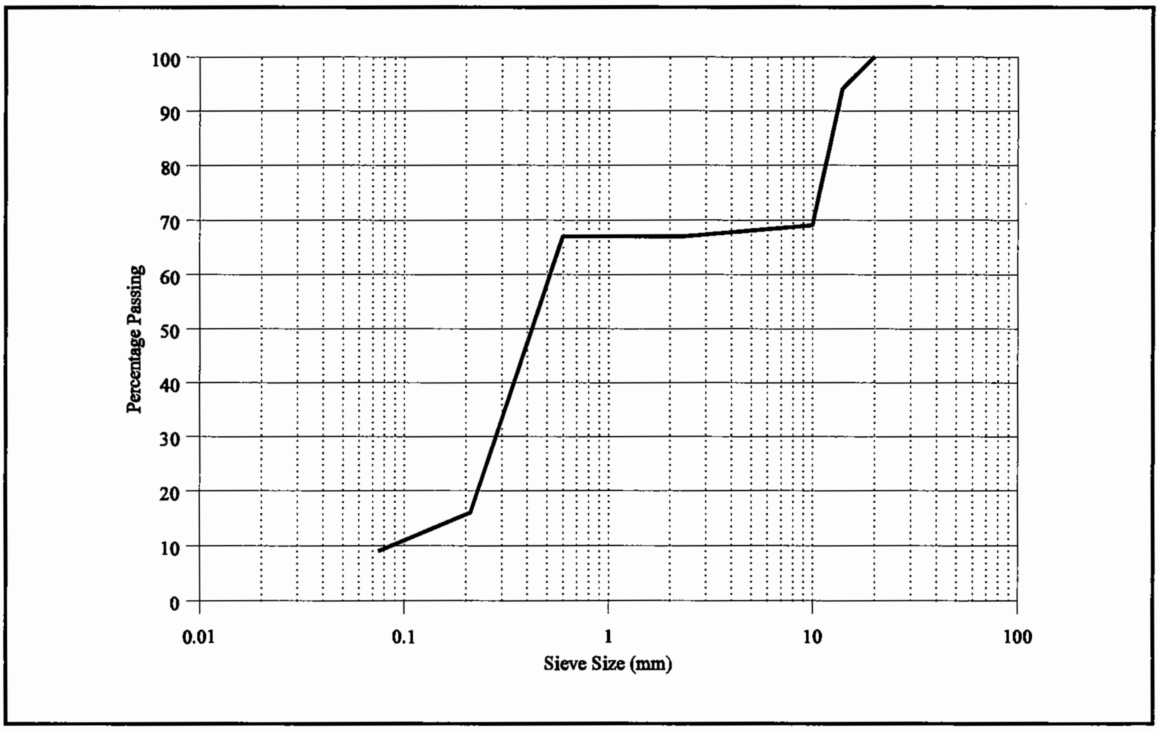


Figure 5.26 Target Grading Curve for the 30/14 HRA Wearing Course

As for the triaxial investigation two grades of bitumen, nominally a 50 Pen and a 200 Pen obtained from the same UK supplier, were used to provide a range in binder stiffness. The initial and recovered properties of the bitumens, are given in Table 5.5. The binder contents for the DBM and the HRA were 4.7% and 7.5% by mass, respectively.

Table 5.5 Initial and Recovered Binder Properties

Nominal Penetration Grade	Initial Properties		Recovered Properties		
	Penetration @ 25°C ($m \times 10^{-4}$)	Softening Point (°C)	Penetration @ 25°C ($m \times 10^{-4}$)	Softening Point (°C)	Penetration Index
50	53	51.7	44	53.8	-0.29
200	215	37.1	191	39.5	-0.19

5.3.3 Test Programme

Test specimens were obtained by taking 100mm diameter cores from slabs compacted using the roller compactor described in Chapter Three. Equiviscous temperatures for mixing and compaction were determined from initial binder properties.

In order to examine the response of the mixtures over a wide range of binder stiffness, RLA tests were carried out at temperatures of 20°C, 30°C and 40°C. The other test conditions were standardised as shown below :

Compressive Stress	:	100 kPa
Load pulse frequency	:	0.5 Hz
Load pulse duration	:	1 second
Test duration	:	3600 cycles
Pre-test conditioning	:	10 minutes @ 10 kPa (static)

5.3.4 Results of RLA Tests

The results of the RLA tests, presented in Tables 5.6 and 5.7 for the 200 Pen and 50 Pen mixtures respectively, are given in terms of ultimate strain, mean strain rate and also minimum strain rate. One reason for the use of mean strain rate is that it is derived from all of the data for a particular specimen, and has the advantage over ultimate strain that a numerical result is obtained even if the specimen fails during the test. However, mean strain rate is weighted to emphasise performance in the initial phase of the test where data collection rate is greatest and strain rates tend to be highest, at least in specimens which do not fail. For this reason, minimum strain rate, as used by the Australians for classification of mixture performance (128), has also been determined since this parameter should reflect the response of the specimen at a later stage of the test where strain rates are generally lowest unless failure occurs. Consideration of minimum strain rate in conjunction with mean strain rate should, therefore, give a more complete representation of response throughout the test.

Minimum strain rate is defined as the minimum value of $\dfrac{\Delta\epsilon}{\Delta n}$

where $\Delta\epsilon = (\epsilon_{i+1} - \epsilon_i)$, and ϵ_i is the deformation recorded at increment i

$\Delta n = (n_{i+1} - n_i)$, and n_i is the number of load cycles elapsed at increment i

The reported volumetric parameter is VMA which is considered a more appropriate indicator of volumetric composition than air void content for the comparison of different mixture types.

The test plots are shown in Figures 5.27 to 5.32. It should be noted that the upper limit of the vertical scale in Figures 5.30 and 5.31 is less than half that in the other figures. It is clear from these plots that at all temperatures with the 200 Pen binder, the continuously graded material has performed better than the gap graded. This trend is not, however, repeated for the 50 Pen mixtures where the DBM has performed generally better at 40°C, but at 30°C and also at 20°C the HRA appears slightly better.

Table 5.6 RLA Test Results : 200 Pen Mixtures

Test Temperature	Specimen Reference	Mixture Type	VMA (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain /cycle)	Minimum Strain Rate (microstrain /cycle)
40°C	TS 3/2	HRA	23.1	F	695	540.3
	TS 4/4	HRA	24.0	F	1918	1408.5
	TS 3/5	HRA	22.4	F	742	414.2
	TS 5/4	DBM	15.6	F	200	80.7
	TS 5/6	DBM	16.7	F	221	113.9
	TS 6/3	DBM	17.1	F	386	213.5
30°C	TS 3/4	HRA	23.1	F	248	87.8
	TS 3/6	HRA	22.7	F	216	79.9
	TS 4/2	HRA	24.2	F	384	201.2
	TS 5/3	DBM	15.0	F	55	10.6
	TS 6/2	DBM	17.0	F	236	64.2
	TS 6/5	DBM	16.8	F	216	51.9
20°C	TS 3/3	HRA	22.7	F	59	8.8
	TS 4/1	HRA	23.2	F	76	18.9
	TS 4/6	HRA	24.0	F	98	30.1
	TS 5/2	DBM	15.7	1.83	22	2.1
	TS 6/4	DBM	16.6	4.44	37	7.1
	TS 6/6	DBM	17.1	5.09	38	8.7

F denotes failure of the specimen under test

Table 5.7 RLA Test Results : 50 Pen Mixtures

Test Temperature	Specimen Reference	Mixture Type	VMA (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain /cycle)	Minimum Strain Rate (microstrain /cycle)
40°C	TS 1/2	HRA	22.0	F	49	8.9
	TS 1/5	HRA	22.2	F	92	17.4
	TS 1/6	HRA	21.9	F	61	12.0
	TS 7/2	DBM	17.7	F	95	17.9
	TS 8/1	DBM	15.6	2.41	35	2.8
	TS 8/3	DBM	15.8	4.20	47	5.7
30°C	TS 1/3	HRA	22.8	1.19	15	0.7
	TS 2/1	HRA	21.7	1.40	17	0.4
	TS 2/2	HRA	21.5	1.13	16	0.1
	TS 7/5	DBM	16.7	2.56	29	1.4
	TS 8/2	DBM	15.8	1.54	17	0.7
	TS 8/4	DBM	15.3	1.44	16	0.5
20°C	TS 1/1	HRA	22.4	0.62	9	0.1
	TS 1/4	HRA	21.8	0.41	5	0.0
	TS 2/4	HRA	21.6	0.45	6	0.1
	TS 7/1	DBM	17.5	1.12	13	0.8
	TS 8/5	DBM	15.9	0.71	8	0.5
	TS 8/6	DBM	15.5	0.59	7	0.3

F denotes failure of the specimen under test

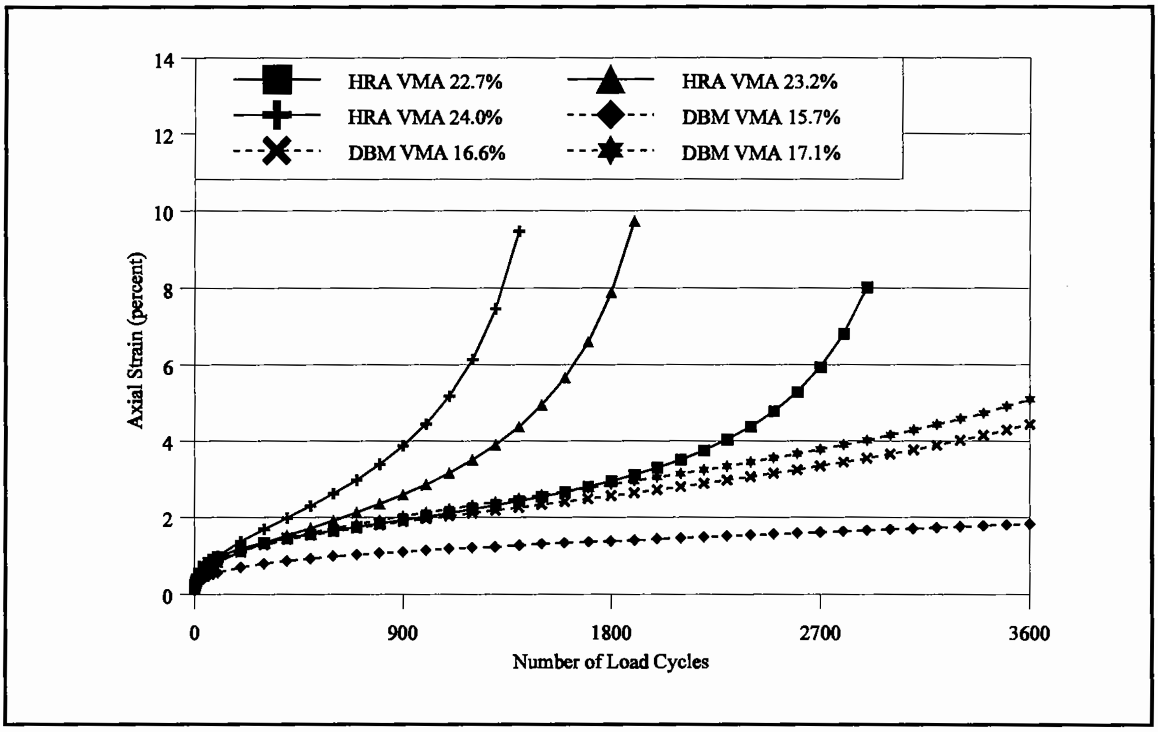


Figure 5.27 Effect of Test Temperature : 200 Pen Mixtures at 20°C

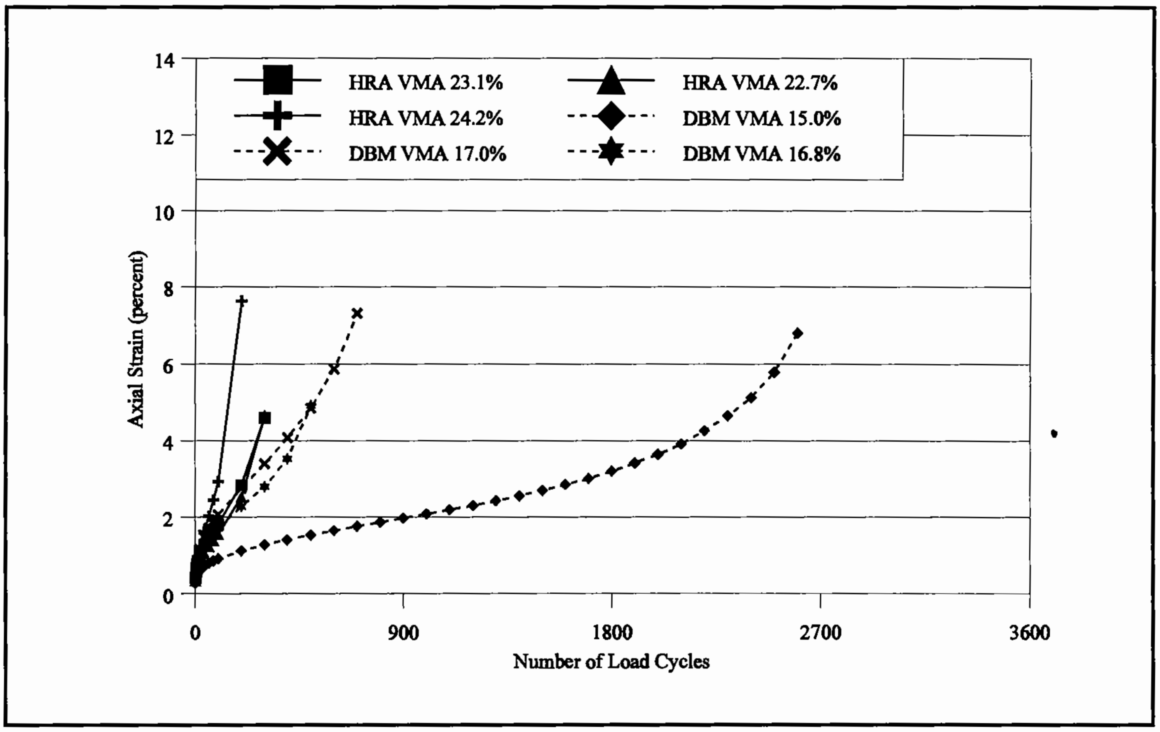


Figure 5.28 Effect of Test Temperature : 200 Pen Mixtures at 30°C

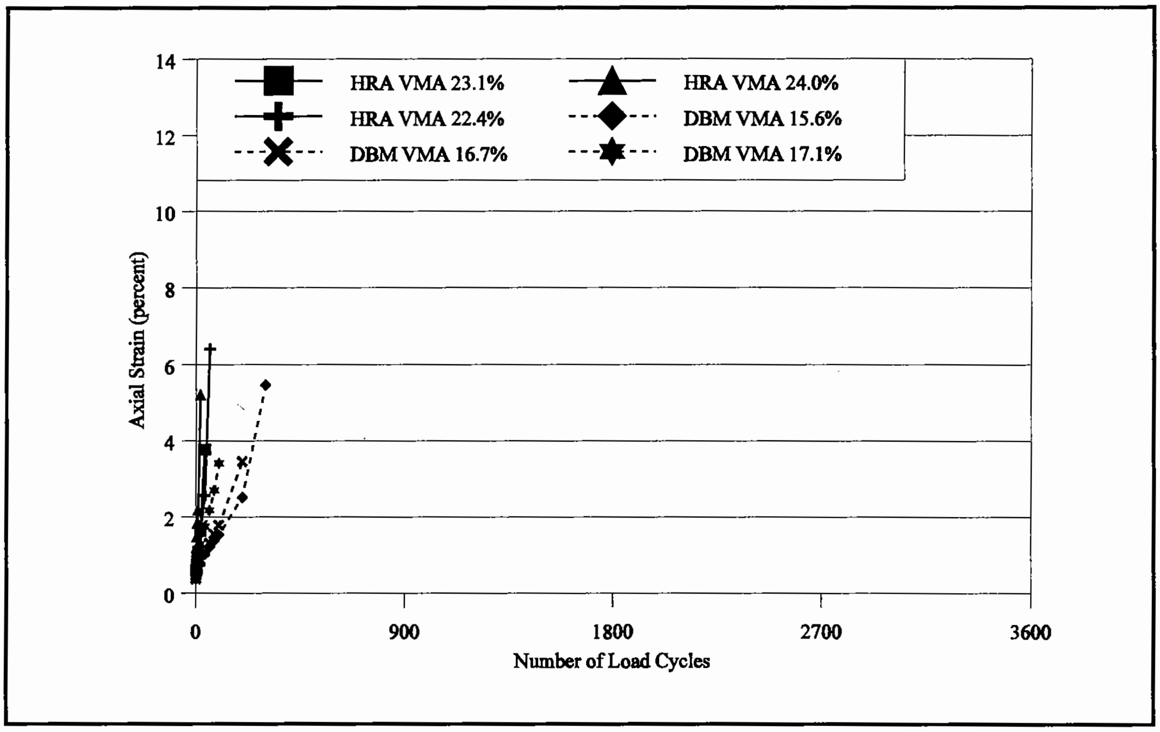


Figure 5.29 Effect of Test Temperature : 200 Pen Mixtures at 40°C

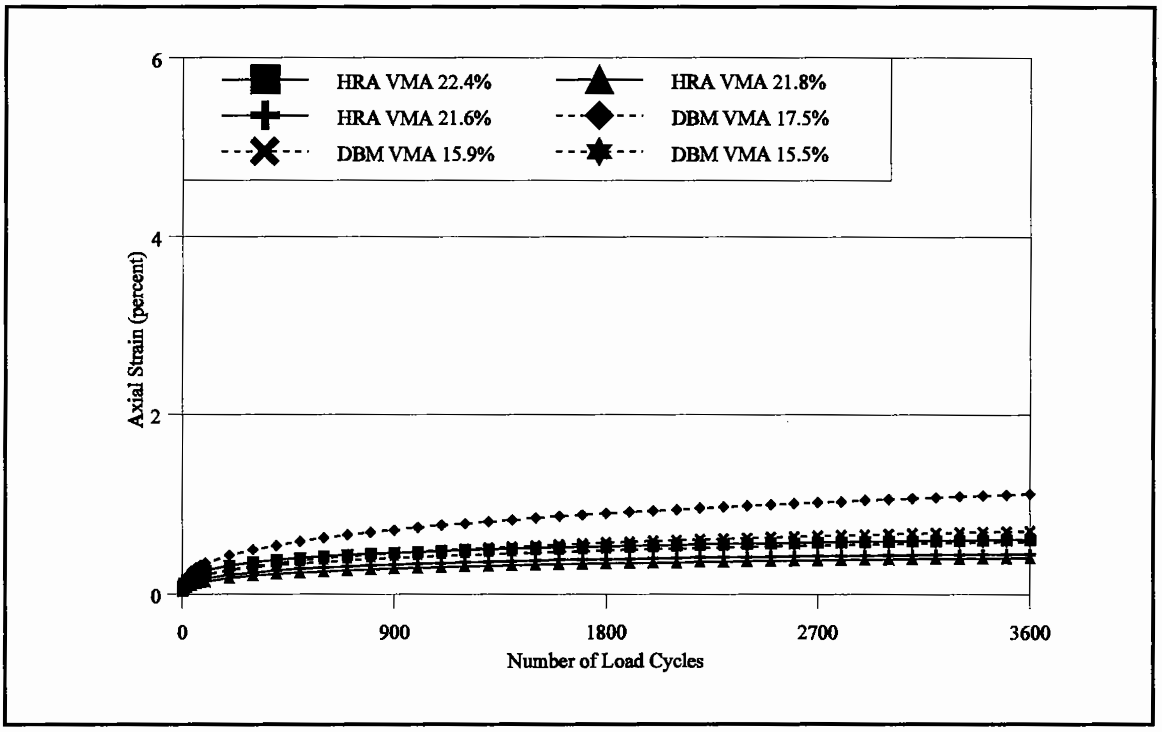


Figure 5.30 Effect of Test Temperature : 50 Pen Mixtures at 20°C

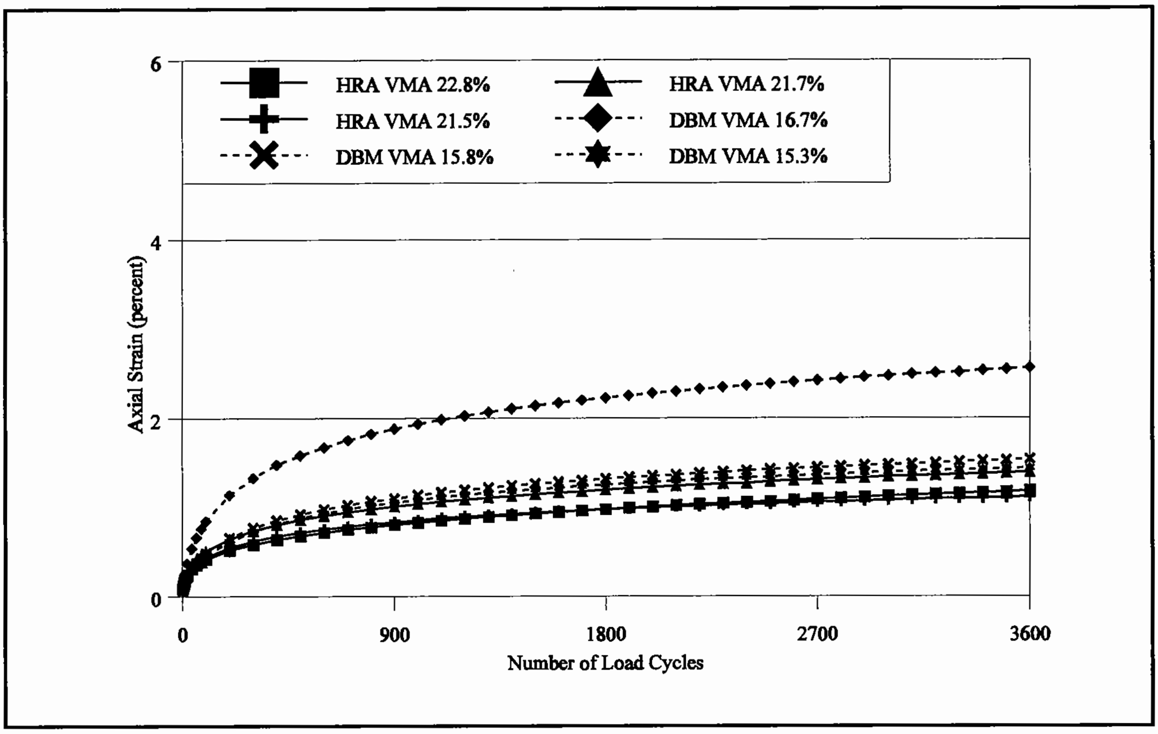


Figure 5.31 Effect of Test Temperature : 50 Pen Mixtures at 30°C

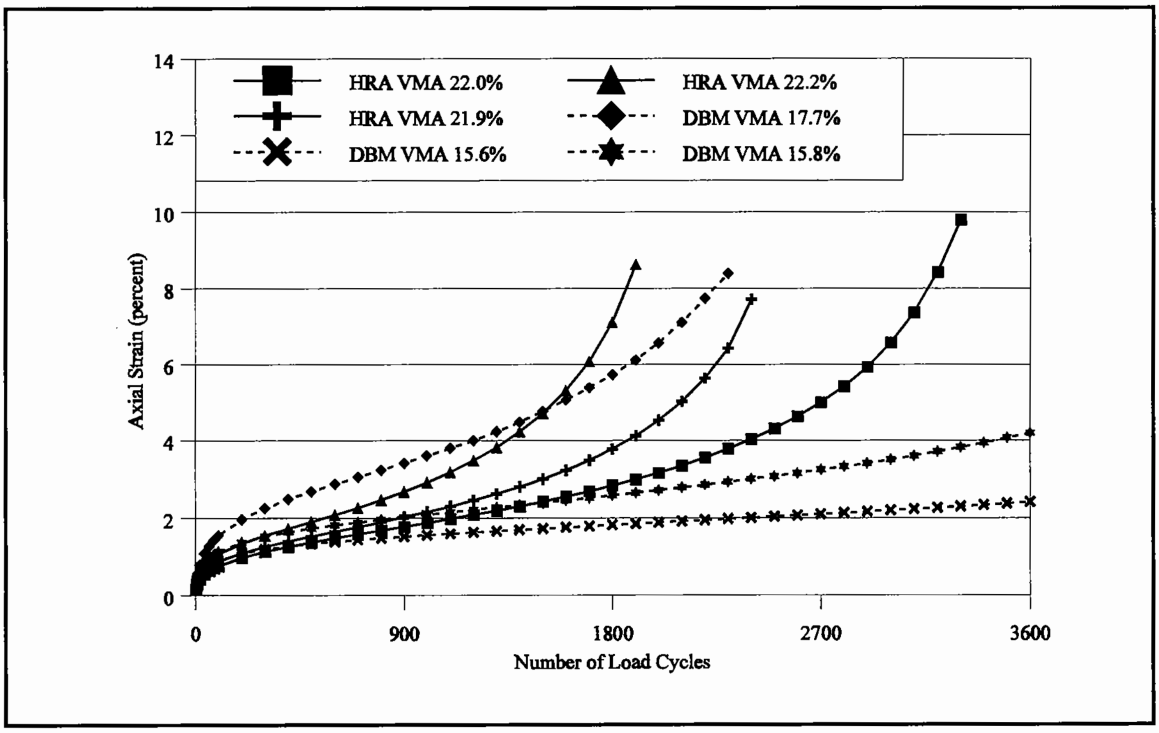


Figure 5.32 Effect of Test Temperature : 50 Pen Mixtures at 40°C

These results may be explained in terms of the effect of binder stiffness. At low binder stiffness aggregate structure is important which is why the DBM performed better than the HRA when 200 Pen bitumen was used. At high binder stiffness (the 50 Pen data at 20°C and 30°C) the gap graded material performs well with little permanent strain in the binder/fines/filler mortar but plastic strains do occur in the aggregate structure of the continuously graded material under the action of repeated loading. At 40°C with the 50 Pen material, the binder stiffness has evidently reduced to the point where the aggregate structure provides a greater proportion of the resistance to permanent deformation (Figure 5.32).

Figures 5.33 and 5.34 show, for all the mixtures, the effect of temperature on mean strain rate and minimum strain rate respectively, the plotted values being the average of three results. At the scale of these plots, the performance of both the 50 Pen mixtures is similar, showing a gradual deterioration with increase in temperature. The 200 Pen DBM shows a similar trend but the 200 Pen HRA exhibits a rapid deterioration in performance above 30°C. These results show that binder is dominant up to the point where its stiffness is reduced below a level at which it is able to contribute significantly to deformation resistance. Beyond this point, the role of the aggregate structure is dominant.

Figures 5.35 and 5.36 show, respectively, mean and minimum strain rate against temperature for the 50 Pen mixtures only, at an enhanced scale. These plots illustrate the change in dominance of response mechanisms, with aggregate structure becoming more important as binder stiffness decreases.

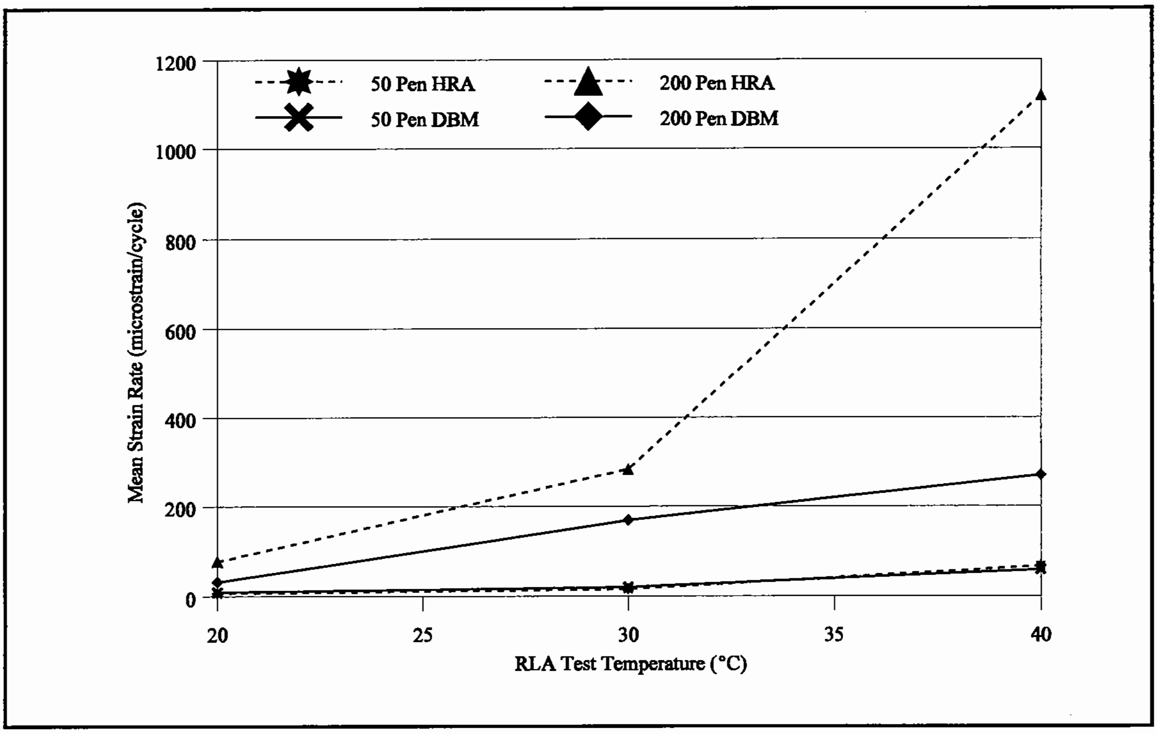


Figure 5.33 Effect of Test Temperature on Mean Strain Rate : All Mixtures

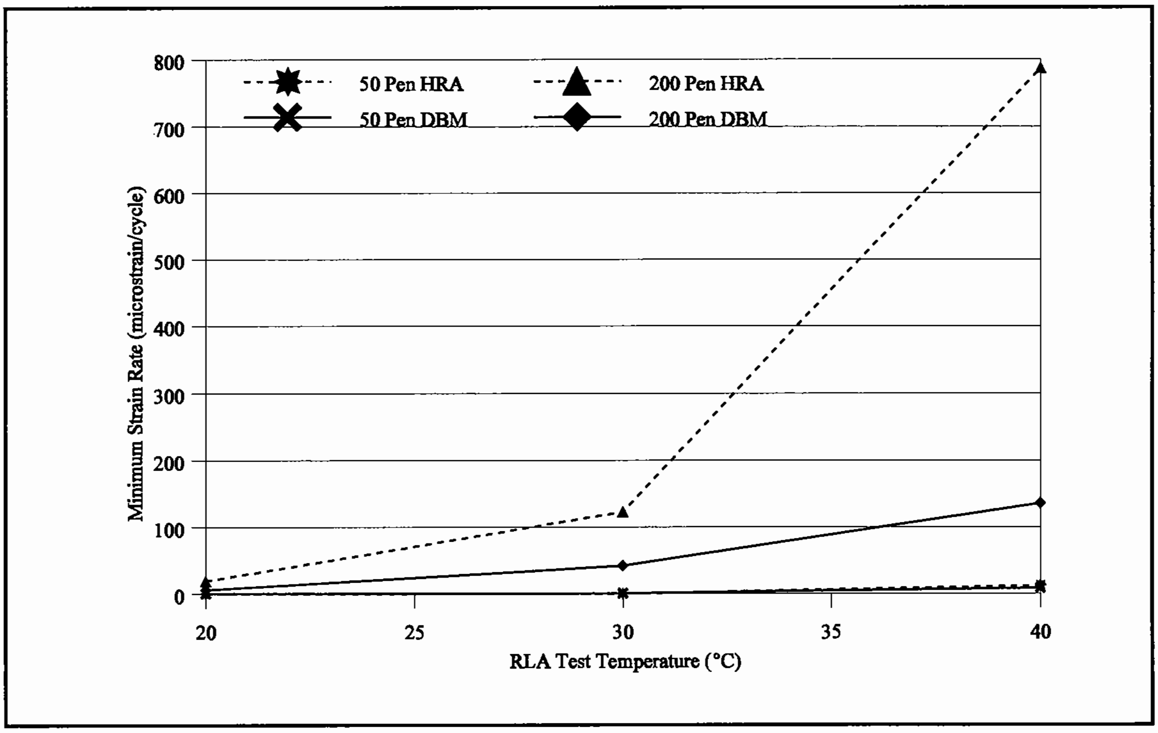


Figure 5.34 Effect of Test Temperature on Minimum Strain Rate : All Mixtures

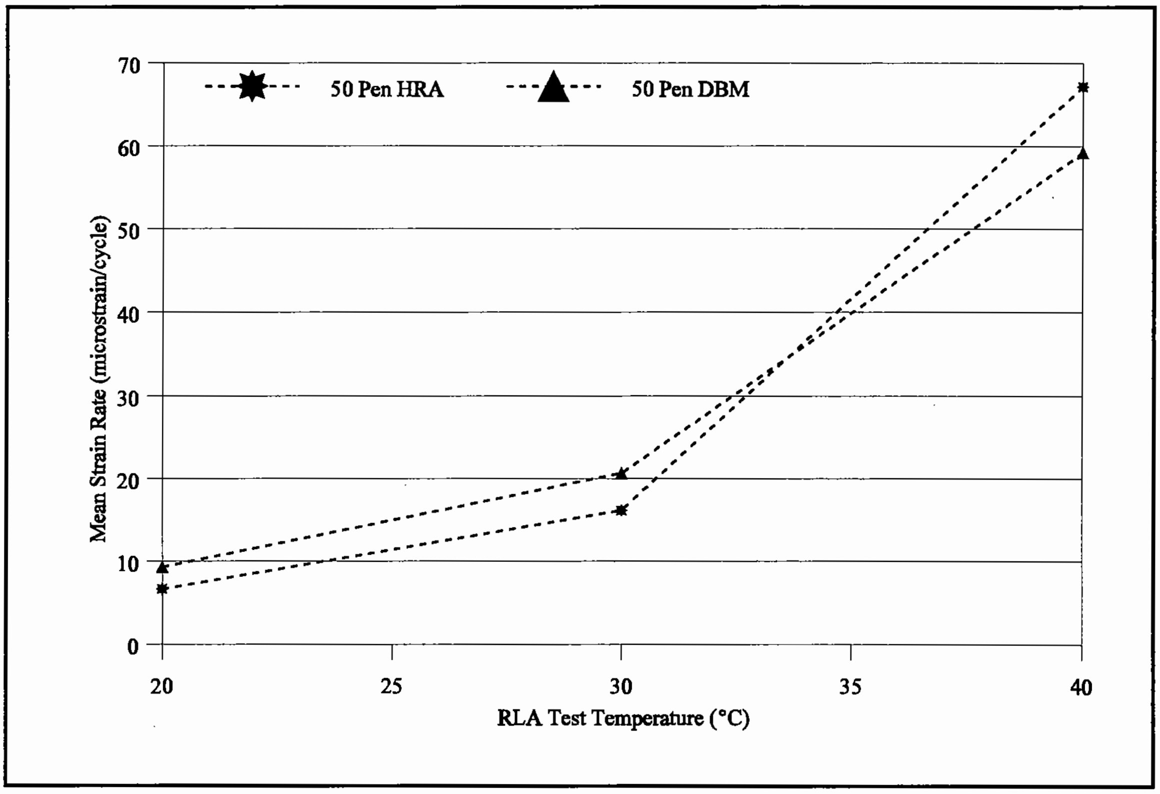


Figure 5.35 Effect of Test Temperature on Mean Strain Rate : 50 Pen Mixtures

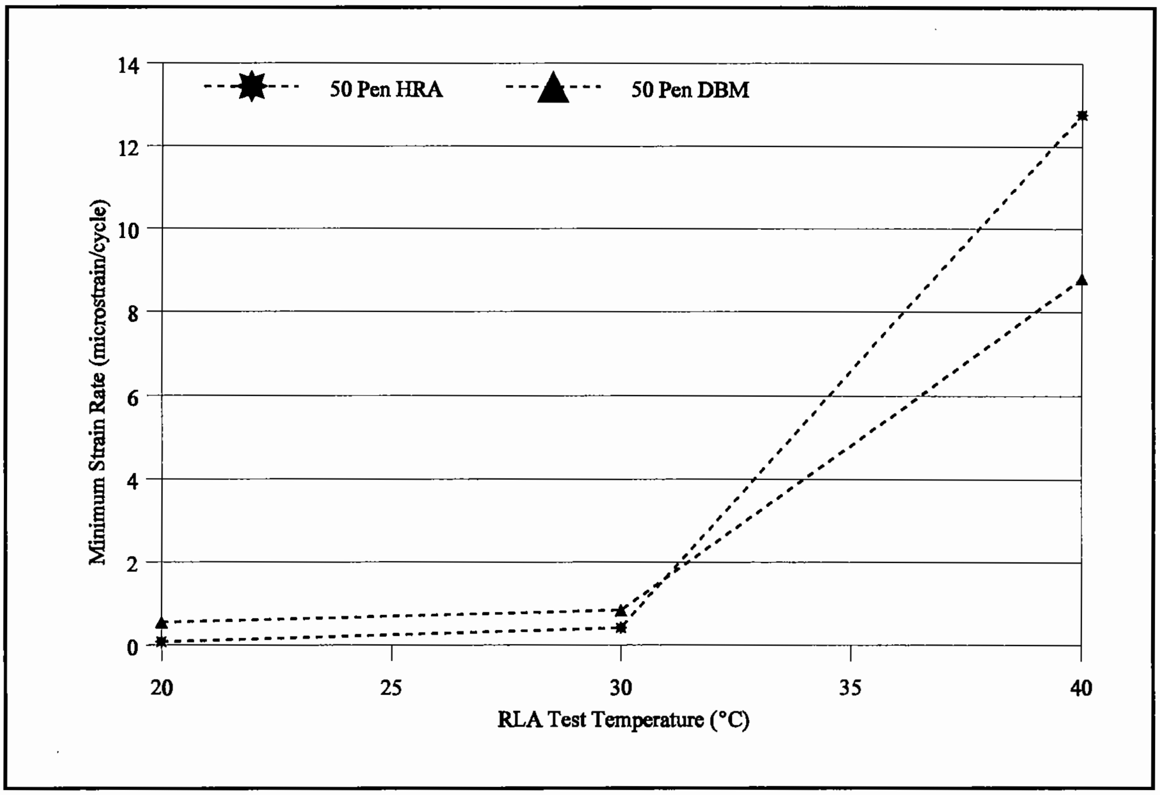


Figure 5.36 Effect of Test Temperature on Minimum Strain Rate : 50 Pen Mixtures

5.4 Discussion

The use of single-sized materials in the triaxial test programme produced an aggregate structure with high VMA but also having a high volume concentration of coarse aggregate, giving a contiguous framework for the transmission of load. The tests on both bound and unbound specimens demonstrated, without any complicating effects of grading, the importance of aggregate quality for resistance to permanent deformation in this structure which relies on interparticle friction and interlock. The dominance of the aggregate was emphasised by the similarity in the results for the bound and unbound gritstone, particularly under repeated loading, where the addition of bitumen evidently contributed little to performance. It should, however, be recognised that the volumes of binder used in these "artificial" mixtures were, in practical terms, very small so the effect might reasonably be expected also to be small.

Nevertheless, the influence of the binder on the performance of the smoother, rounded ceramic aggregate was much more pronounced, though the effect was not always beneficial. At lower stiffness, due to either the use of the softer 200 Pen or the long load duration of the static test, the addition of bitumen failed to restrain plastic deformation in this aggregate with low friction and interlock, and actually proved detrimental in apparently providing sufficient lubrication to promote this deformation. Only where the binder was present in sufficient quantity and with the highest stiffness, derived from the use of 50 Pen bitumen and repeated loading, did it improve the performance of the ceramic aggregate.

The tests on the unbound aggregates in which higher permanent strains were generally obtained under repeated loading provide further evidence of the importance of dynamic effects in causing plastic strains in an aggregate skeleton which do not occur under static loading with the same stress regime, and confirm similar findings reported by van de Loo (86).

In contrast to the triaxial test programme, the specimens tested in the investigation carried out in the NAT were all made from the same aggregates, but with gradings selected to produce structures with different principal mechanisms for the transmission of load and resistance to deformation. The mixtures chosen were typical dense mixtures used in flexible construction in the UK and as such had rather higher binder contents than the "artificial" mixtures used in the triaxial testing. The effects of varying binder stiffness on resistance to permanent deformation were, therefore, apparent in both the mixture relying on the binder/fines/filler mortar and also that with the continuous aggregate skeleton though the coarse aggregate in both was the gritstone as used in the triaxial tests.

Where the stiffness of the bitumen was highest the gap graded HRA marginally outperformed the continuously graded DBM, but the latter was far superior under conditions where that stiffness was lowest. It would appear, therefore, that while changes in mixture performance may be explained in terms of the effect of changes in binder stiffness, the magnitudes of those changes are not necessarily directly proportional because the influence of the aggregate structure increases as binder stiffness decreases. The results do, however, demonstrate that whatever the aggregate structure the stiffness of the bitumen in such dense mixtures has a significant effect on resistance to permanent deformation.

5.5 Conclusions

The tests on the typical dense mixtures carried out in the NAT demonstrated the significant role of the binder in resistance to permanent deformation, as had been observed in the work described in Chapter Four. Aggregate quality was shown to be of greater importance in the single-sized materials tested in the triaxial programme, though the quantities of bitumen added to the mixtures were small. However, these tests did show that the effects of the binder were proportionately greater on the poor quality aggregate, which is consistent with the findings of Chapter Four.

CHAPTER SIX
EFFECTS OF TEST VARIABLES

6.1 Introduction

The principal objective of this research was to assess and develop a test for the evaluation of resistance to permanent deformation suitable for use both in mixture design and, if possible, performance specification. Investigations reported in preceding Chapters have shown the RLA test to have considerable potential in this respect but that the response, and hence the assessment, of mixtures may be fundamentally affected by certain aspects of the test configuration. Of particular importance are the test temperature, insofar as it affects binder stiffness, and the use of confinement.

However, in development of the test for widespread routine use the influence of other more practical elements of the configuration and procedures must also be assessed. For this reason, a study was undertaken to assess the effect of variation in certain test variables on the result of the RLA test with the aim of establishing suitable conditions and procedures for incorporation into standards or guidelines for routine practice. The parameters considered in this study were pre-test conditioning, test stress and specimen length.

All RLA testing was performed without confinement as the vacuum confining system had not been developed at the time when this work was carried out. Determination of specimen volume for calculation of volumetric data was in all cases by immersion of the sealed specimen and maximum theoretical mixture densities were obtained by the Rice method.

6.2 Use of Conditioning Period

6.2.1 The Purpose of Pre-Test Conditioning

The use of a pre-test conditioning routine is common practice in many laboratory test procedures to obviate initial or "start-up" errors. One approach often adopted in permanent deformation testing of bituminous materials is to defer the commencement of deformation measurement until after the completion of a number of load cycles. In both Holland (70) and Australia (129) repeated load, unconfined uniaxial compression tests with configurations similar to the RLA test are used and, in both these cases, the test procedure requires 5 load cycles to be applied prior to recording permanent deformation so that the loading platens may "bed in". The same form of test is also used in South Africa (43), where the first 30 load cycles are used for the same purpose. A widely recognised standard procedure (130), presented in 1977, for the static creep test recommended the use of a static pre-load of up to 2% of the test stress applied for a duration of 10 minutes.

Such procedures simply form part of the process of setting up the specimen in preparation for testing. However, extended periods of cycling to change the state of the material are sometimes employed. Figure 6.1 shows an idealised plot of deformation against load cycles/duration of the form generally obtained from permanent deformation tests. Considerable variation between individual test specimens is often observed in the primary phase of the material response and procedures are sometimes adopted, often for research purposes (eg 131), in which measurements are deferred until the linear secondary phase has been reached. The conditioning regime for the LCPC wheeltracking test (101) is the application of 1000 wheel passes at ambient temperature prior to testing at 60°C which, it could be argued, goes beyond simply setting up of the specimen. Finn et al (59) have criticised the use of light pre-test conditioning regimes in the creep test for not changing the state of the specimen, and supported the use of more severe conditions on the basis that this would more closely correspond with densification and stiffening to be expected in service.

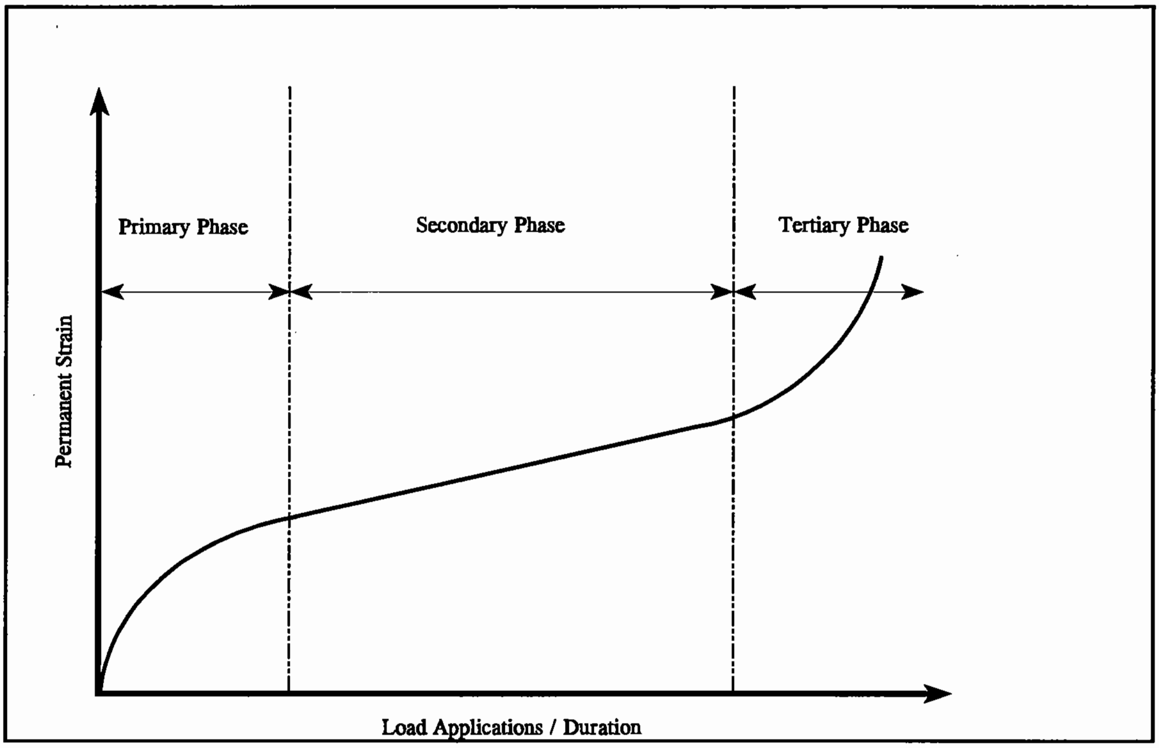


Figure 6.1 Schematic Creep Curve

6.2.2 Test Programme

A static pre-load of 10 kPa (10 percent of test stress) applied for a period of 10 minutes had been used as the standard conditioning routine for the RLA test in earlier research at Nottingham (132), principally to ensure that the platens were in intimate contact with the specimen prior to initiating the test loading regime. This regime was adopted at the start of this research but it was observed that strains recorded at the end of the conditioning period were variable and often of significant magnitude. This gave rise to concern that deformation may be occurring in the specimen during the conditioning period which would influence the reported result, particularly where this was quoted as the strain at the end of the test.

An investigation was, therefore, carried out into the effect of variations in the conditioning regime by simultaneously measuring axial permanent deformations both "off-sample" - ie., with the LVDT's located on the top platen, as is the normal configuration - and "on-sample". The "on-sample" measurements were made with the aid of a strain collar, shown schematically in Figure 6.2, to locate and support the LVDT's (126).

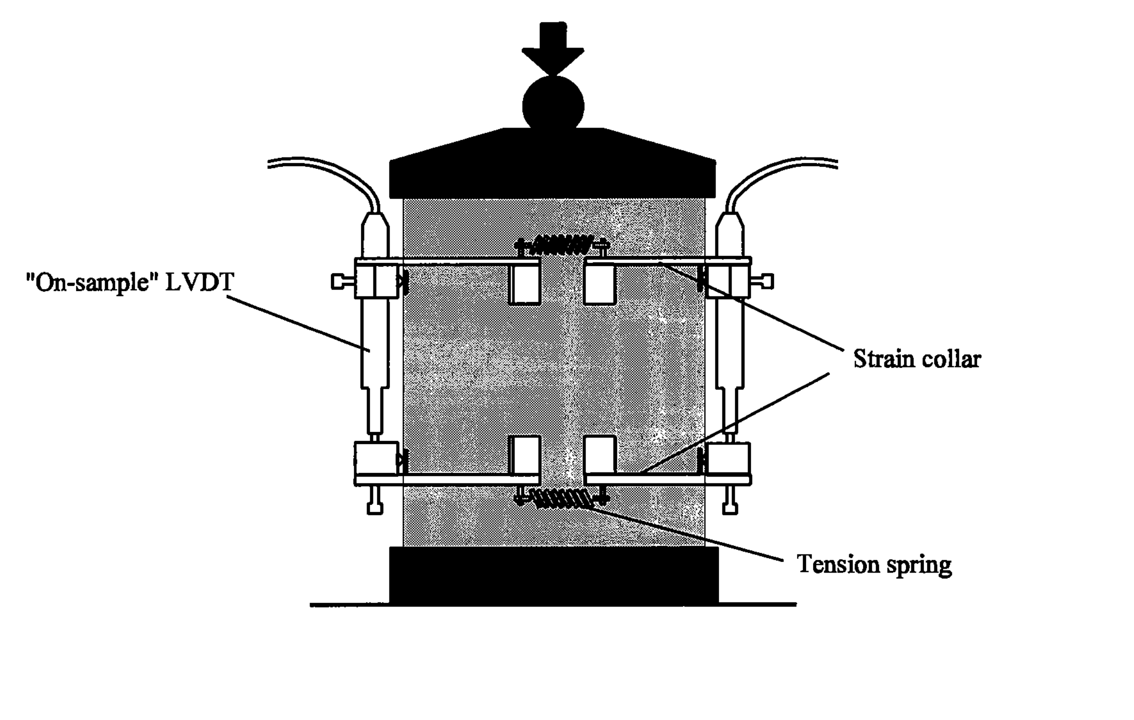


Figure 6.2 "On-Sample" Instrumentation

Specimens were obtained by drilling 100mm diameter cores from slabs of the same 30/14 HRA as used in the programme to assess the effect of binder stiffness, described in Chapter 5. The cores were trimmed, at both ends, to a length of approximately 100mm for testing. A jig was fabricated to hold the strain collar in position on the specimen while it was rigidly attached by means of an impact adhesive. The points of attachment were two pairs of brass pips screwed into the hoops of the strain collar and so arranged that the central 40mm of the length of the core would form the gauge length for measurement at two diametrally opposed positions.

Three pre-test conditioning regimes were assessed ; no conditioning, 5 kPa static load of 2 minutes duration and 10 kPa static load of 10 minutes duration. All tests were carried out at 30°C with an axial stress of 100 kPa applied at 0.5 H_z for 3600 cycles. Specimens were selected so that a similar range of void contents were covered for each conditioning regime.

6.2.3 Results

The strains recorded both "on- and "off-sample" during the conditioning period are presented in Table 6.1. These generally show very small "on-sample" strains for the 2 minutes at 5 kPa regime but rather higher values under 10 minutes at 10 kPa, demonstrating the development of deformation in the sample under these latter conditions.

Table 6.1 Strains Recorded During Conditioning

Conditioning Regime	Specimen Reference	Vv (%)	Strain Recorded During Conditioning (microstrain)	
			Off - Sample	On - Sample
None	TS 9/3	5.1	N/A	N/A
	TS 10/3	4.0	N/A	N/A
	TS 10/5	5.8	N/A	N/A
2 Minutes at 5 kPa	TS 9/5	5.6	421	0
	TS 10/4	3.9	258	4
	TS 10/6	5.1	429	141
10 Minutes at 10 kPa	TS 9/2	5.5	1144	569
	TS 9/6	5.1	1152	453
	TS 10/1	4.4	1237	393

Neither the test data plots, shown in Figures 6.3 to 6.5, nor the results presented in Table 6.2 reveal any discernable difference in performance during the test between the specimens which were not conditioned and those which were subjected to 5 kPa for 2 minutes.

The "off -sample" data does show lower strains and strain rates for the specimens conditioned for 10 minutes at 10 kPa than for those subjected to the other, less severe, conditioning regimes . However, the differences are slight and it is not possible to draw firm conclusions about the significance of this observation from this data.

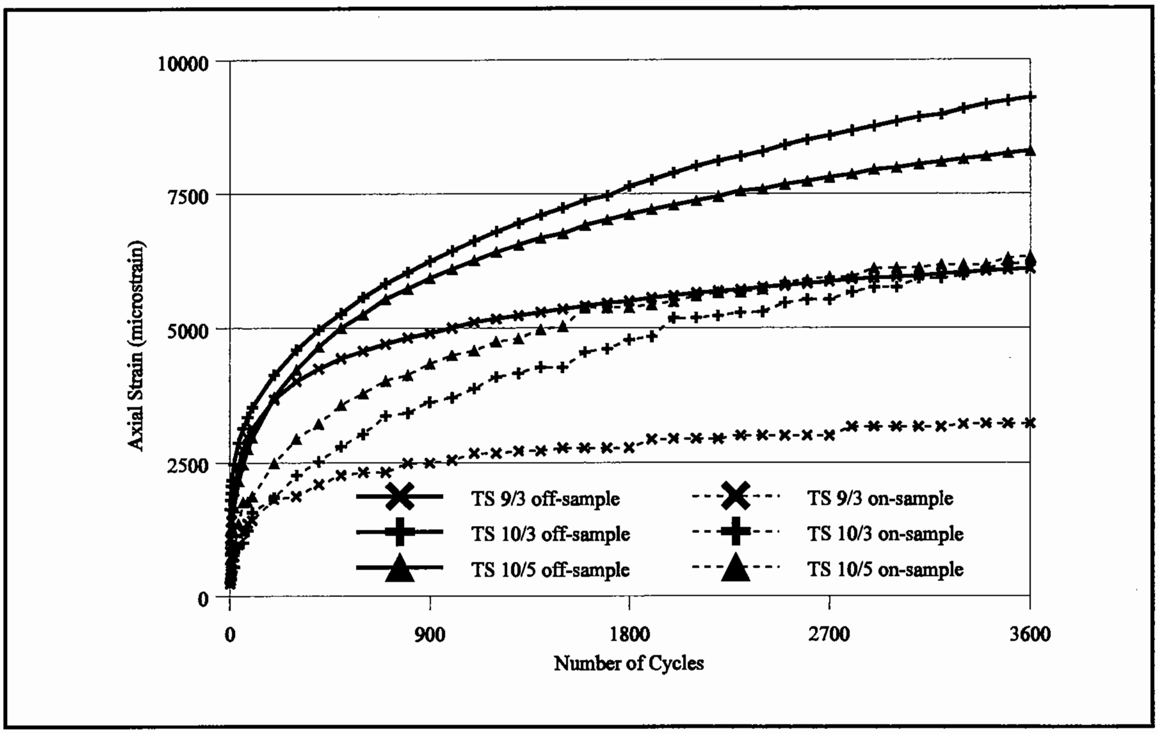


Figure 6.3 "On- and "Off-Sample" Data : No Conditioning

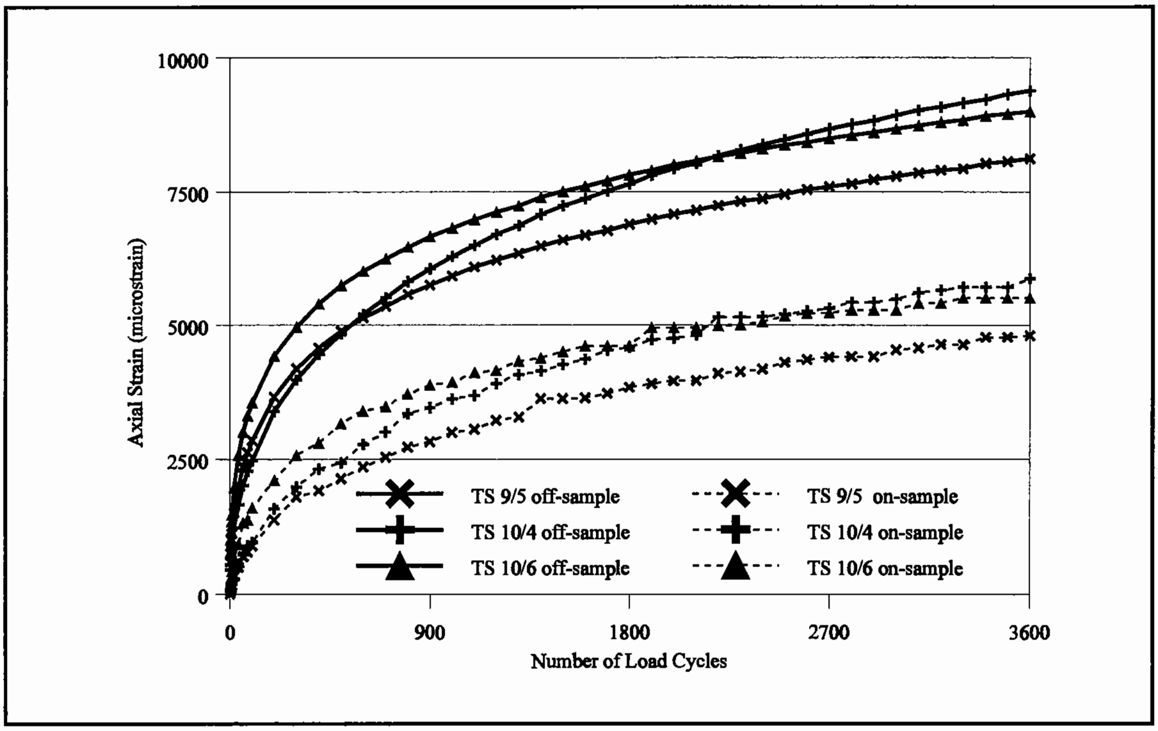


Figure 6.4 "On- and "Off-Sample Data" : 2 Minutes at 5 kPa

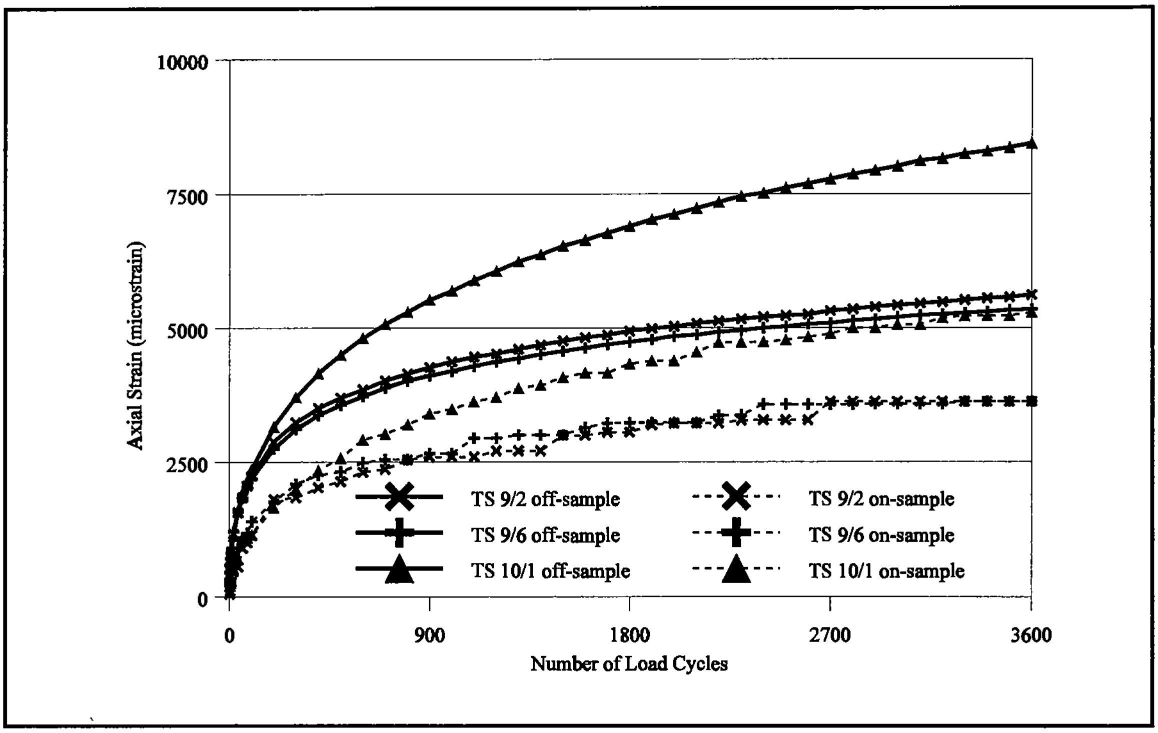


Figure 6.5 "On- and "Off-Sample Data : 10 Minutes at 10 kPa

Table 6.2 RLA Test Results : On- and Off-Sample Measurements

Conditioning Regime	Specimen Reference	Off-Sample Measurement			On-Sample Measurement		
		Ultimate Strain (%)	Mean Strain Rate (μstrain /cycle)	Minimum Strain Rate (μstrain/ cycle)	Ultimate Strain (%)	Mean Strain Rate (μstrain /cycle)	Minimum Strain Rate (μstrain/ cycle)
None	TS 9/3	0.61	11	0.2	0.32	5	0.0
	TS 10/3	0.93	9	0.5	0.62	5	0.0
	TS 10/5	0.83	10	0.3	0.63	6	0.0
2 Minutes at 5 kPa	TS 9/5	0.81	10	0.3	0.48	4	0.0
	TS 10/4	0.94	9	0.6	0.59	4	0.0
	TS 10/6	0.90	12	0.4	0.55	6	0.0
10 Minutes at 10 kPa	TS 9/2	0.56	8	0.2	0.36	4	0.0
	TS 9/6	0.54	8	0.1	0.36	4	0.0
	TS 10/1	0.84	8	0.5	0.53	5	0.0

The preferred philosophy for the use of pre-test conditioning is simply to set up the specimen correctly for the test rather than to alter its state to a partly tested/trafficked condition, since the initial response is an important element of the overall performance. On this basis, the magnitude and duration of the conditioning regime should be kept to a minimum. It would appear, therefore, from the data which were obtained, that 2 minutes at 5 kPa would be preferable to 10 minutes at 10 kPa, since the "on-sample" instrumentation indicated less deformation in the specimen under the former. Although the test plots showed little difference in performance between the unconditioned specimens and those loaded at 5 kPa for 2 minutes, it is recommended that pre-test conditioning be retained since the "off-sample" conditioning strains for the latter may indicate some " bedding in " and preparation of specimen ends may not always be to a uniformly high standard, as was the case in this exercise.

6.3 Use of Strain Rate as Reported Test Result

It is apparent in Figures 6.3 to 6.5 that the permanent deformation curves for the "off-sample" measurement are smoother than those obtained "on-sample". This is probably attributable to the "off-sample" measurement being an "average" of the response of the whole specimen, as suggested by Huschek (113), while the "on-sample" measuring system records local effects. However, the most striking observation is that while the characteristic shape of the curve for the "on-sample" and "off-sample" strains is similar in each case, particularly over the linear phase, the level of strain is always lower for the "on-sample" measurement. The reason for this is not clear, though it would imply large local effects, perhaps at or near the interface with the platens, outside the gauge length of the strain collar. It does, however, represent a strong argument for the use of a measure of strain rate rather than an absolute value of strain as a parameter to report as the test result, a case which is supported by the work of Brown and Cooper (68).

The fact that the different phases of the creep curve, as shown in Figure 6.1, are likely to occur at different stages in the RLA test for different materials renders impractical for routine analysis the calculation of strain rate by, say, linear regression over an arbitrarily chosen period of the test, as was used in the work reported in Chapter Three. This was one reason for the use of mean and minimum strain rates in the analysis of data presented in Chapters Four and Five.

It has already been noted that mean strain rate is weighted to emphasise performance in the initial phase of the test and the values for the "on- and "off-sample" measurements, presented in Table 6.2, tend to differ significantly and reflect more the differences in the overall levels of strain, arising mainly from differences in the primary phase of response, than the similarities in the characteristic shapes of the curves. The values of minimum strain rate measured "off-sample" do, however, better represent the performance of the specimens as indicated by the test plots than do the values of mean strain rate, since this parameter reflects more the behaviour of the specimen in the later stages of the test.

A comparison of the minimum strain rates obtained "on- and "off-sample" is confused by the irregularity in the "on-sample" data which has produced zero minimum strain rates in each case. This highlights a concern with the use of this parameter in that it may not be sufficiently sensitive to discriminate effectively between better performing materials which would exhibit low rates of strain. Nevertheless, this data does appear to support the use of both mean and minimum strain rate to give a more complete and reliable characterisation of performance than might be obtained by using either measure independently.

6.4 Effect of Stress Level

The principal aim of the investigation into the effect of varying the stress level in the RLA test was to determine whether the test could be accelerated by means other than increasing temperature because, as demonstrated in Chapter Five, changing temperature can change the mechanism by which a mixture resists permanent deformation. Any measure which increases rate of testing makes a test more commercially viable and increases the likelihood of its acceptance and use.

Initially tests were carried out on 150mm diameter cores cut from a slab of plant mixed 28mm DBM roadbase, which had been compacted with a pedestrian roller as part of an investigation into the effects of compaction method, described in Chapter Seven. The tests were performed with axial stresses of 100 kPa and 200 kPa. This latter stress is close to the maximum presently achievable with the NAT for this size of core. The other test conditions were as detailed below.

Test temperature	:	30°C
Compressive stress	:	100 kPa & 200 kPa
Load pulse frequency	:	0.5 Hz
Load pulse duration	:	1 second
Test duration	:	3600 cycles
Pre-test conditioning	:	10 minutes @ 10 kPa (static)

The results of these tests are presented in Table 6.5, and plots of the test data are shown in Figure 6.6.

Table 6.5 Effect of Test Stress : 28mm DBM at 100 kPa and 200 kPa

Test Stress (kPa)	Specimen Reference	V_V (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain/cycle)	Minimum Strain Rate (microstrain/cycle)
100	MYA SC4	4.2	3.67	41	2.1
	MYA SE2	3.1	2.55	32	1.3
	MYA SG4	4.7	3.26	36	2.5
200	MYA SA3	3.0	4.85	79	2.1
	MYA SA4	4.7	6.44	92	5.0
	MYA SC1	3.5	3.92	57	2.4

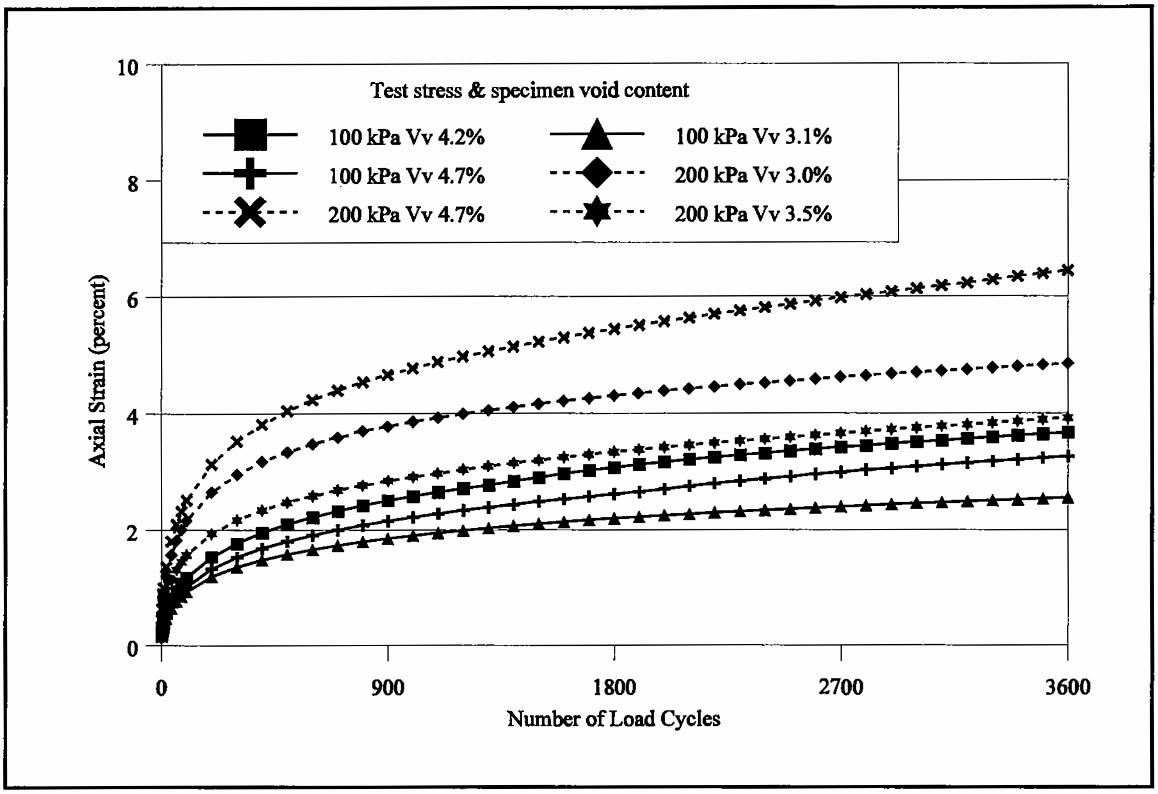


Figure 6.6 Effect of Test Stress : 28mm DBM Roadbase

It is apparent from the test plots that higher strains have resulted from the tests carried out at the higher stress level, and this is reflected in the values of mean strain rate given in Table 6.5. This, however, is largely due to higher strains in the initial phase of the test while the characteristic shapes of the curves are the same for both stress levels, as indicated by the general similarity in minimum strain rates. The use of a higher stress has not accelerated the onset of the linear phase, nor has it induced failure in any of the specimens.

In order to determine whether a more pronounced effect would be obtained at stress levels above 200 kPa, it was necessary to use specimens with a smaller cross sectional area. 100mm diameter cores were, therefore, drilled from 150mm diameter cores of a plant mixed 28mm HDM, also obtained during the study of the effects of compaction technique. These cores were trimmed to a length of approximately 100mm in preparation for testing with axial stresses of 100 kPa and 400 kPa. The results of these tests, which were otherwise under the same conditions as for the 28mm DBM, are presented in Table 6.6, and plots of the data are shown in Figures 6.7 and 6.8.

Table 6.6 Effect of Test Stress : 28mm HDM at 100 kPa and 400 kPa

Test Stress (kPa)	Specimen Reference	V_V (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain/cycle)	Minimum Strain Rate (microstrain/cycle)
100	MYB A4	2.4	1.46	14	0.9
	MYB B1	2.1	1.09	11	0.7
	MYB B4	3.2	1.65	16	1.1
	MYB D1	5.6	1.05	10	0.6
	MYB E1	4.5	1.18	12	0.7
	MYB F3	1.7	1.86	15	1.7
400	MYB C4	3.3	5.07	55	4.3
	MYB E4	2.6	6.38	65	4.4
	MYB F2	1.0	4.57	49	3.0
	MYB F4	2.4	5.67	57	4.6
	MYB G1	5.4	4.66	49	5.3
	MYB G4	1.9	5.71	52	7.2

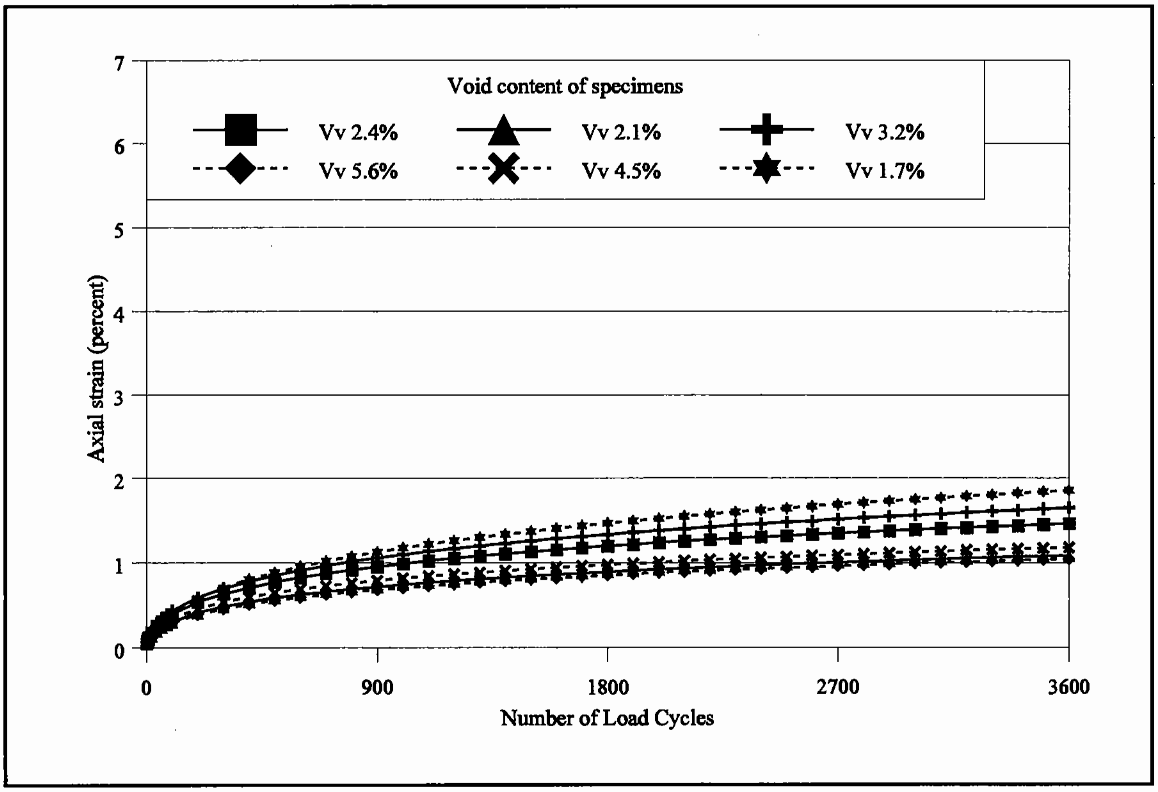


Figure 6.7 Effect of Test Stress : 28mm HDM Roadbase at 100 kPa

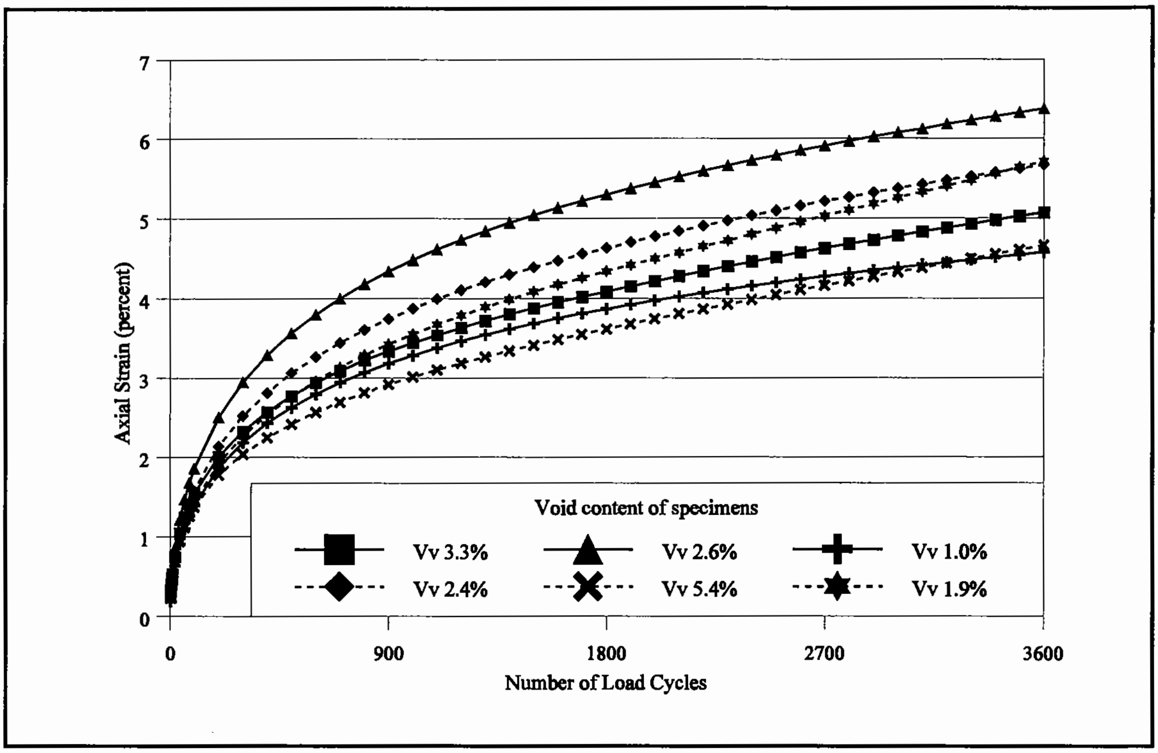


Figure 6.8 Effect of Test Stress : 28mm HDM Roadbase at 400 kPa

These results clearly show that both the overall level of strain and the rate of strain in the linear regime have been increased significantly by increasing the test stress from 100 kPa to 400 kPa. Since none of the specimens failed at either stress level it is not possible to assess what reduction in the test life of the specimen this would represent, though the onset of the linear phase does not appear to have been appreciably advanced by the use of the higher stress level.

On the basis of the data obtained it would appear that there is little merit in increasing test stress from 100 kPa to 200 kPa. An increase to 400 kPa has the potential to reduce the length of the test and, by virtue of increasing the magnitude of strain and strain rate, may make the test more discriminating. However, this would presently restrict testing to 100mm diameter specimens because the NAT is not capable of applying the load required for a 150mm diameter specimen.

6.5 Effect of Specimen Length

As part of an extensive parametric study for the static creep test (108), de Hilster and van de Loo investigated the influence of both specimen height and aspect ratio (ratio of height to diameter), and concluded that there was no consistent or significant effect provided there was minimal friction between the specimen and loading platens. The effect of end restraint in causing barrelling of the specimen during triaxial deformation tests has been demonstrated by Snaith (133). De Hilster and van de Loo proposed the use of silicone grease and graphite flakes to lubricate the ends of the specimen, and this was adopted in the 1977 creep test procedure (130) and also in draft standards for axial deformation testing developed in both the UK (134) and Australia (129).

However, since the length of cored specimens taken from site for axial deformation testing in the NAT is frequently constrained by the thickness of the compacted lift, which for basecourse materials is likely to be less than 60mm while cores from wearing course are likely to be thinner still, there remained a concern over the use of thin specimens, or at least specimens with a low aspect ratio. This concern related both to the potential influence of lateral restraint at the loading platens and also the effects of aggregate particles which are large in relation to the thickness of the specimen.

An investigation into the effect of specimen length on the result reported from the RLA test was, therefore, carried out. All specimens tested both in this exercise and the whole of this research were lubricated using graphite powder only. This was because it had been found in early tests that it was often difficult to remove the loading platen from the specimen at the end of the test when silicone grease had been applied, indicating that it may be counter-productive for minimising end restraint. Furthermore, it is understood that the use of graphite flakes and silicone grease in the Australian standard is being reviewed following the results of a study which has attributed the majority of variation in test results to inconsistencies in the friction developed between specimen and load platen (135).

The results of a series of RLA tests performed on 82mm diameter cores of a 25mm nominal size asphaltic concrete are presented in Table 6.7. The plots of mean strain rate against length for the specimens with void contents in the ranges 4.0 to 5.6 % and 7.4 to 9.4 %, which are shown in Figures 6.9 and 6.10 respectively, both indicate that performance of this material is insensitive to specimen length over the range considered. This is supported by the corresponding plots of minimum strain rate against specimen length presented in Figures 6.11 and 6.12.

Table 6.7 RLA Test Results for 25mm Asphaltic Concrete : (Mixture B0W)

Specimen Reference	Length (mm)	Aspect Ratio	V_V (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain/ cycle)	Minimum Strain Rate (microstrain/ cycle)
B0W 415B	47	0.57	5.5	0.73	11	0.4
B0W 214B	49	0.6	4.1	0.74	12	0.0
B0W 214A	69	0.84	4.2	1.25	25	0.3
B0W 115A	71	0.87	4.0	0.75	10	0.3
B0W 415A	71	0.87	5.0	0.67	10	0.2
B0W 114C	99	1.21	5.5	0.89	11	0.9
B0W 415C	99	1.21	5.6	0.65	9	0.4
B0W 214C	100	1.22	4.1	0.68	9	0.4
B0W 115B	149	1.82	5.2	1.02	11	0.2
B0W 318B	48	0.59	7.9	1.73	20	2.0
B0W 418A	70	0.85	7.4	2.68	22	4.6
B0W 318C	70	0.85	8.1	F	64	11.5
B0W 371C	71	0.87	8.4	4.07	22	4.2
B0W 371B	73	0.89	9.4	3.18	23	4.7
B0W 137A	74	0.9	9.4	2.16	20	3.7
B0W 337A	91	1.11	8.4	2.42	20	4.2
B0W 318A	99	1.21	8.1	2.09	18	3.5
B0W 450B	109	1.33	8.1	0.93	10	0.7
B0W 337B	129	1.57	8.2	1.82	18	2.7
B0W 418B	150	1.83	8.9	F	15	8.7

F denotes failure of the specimen under test

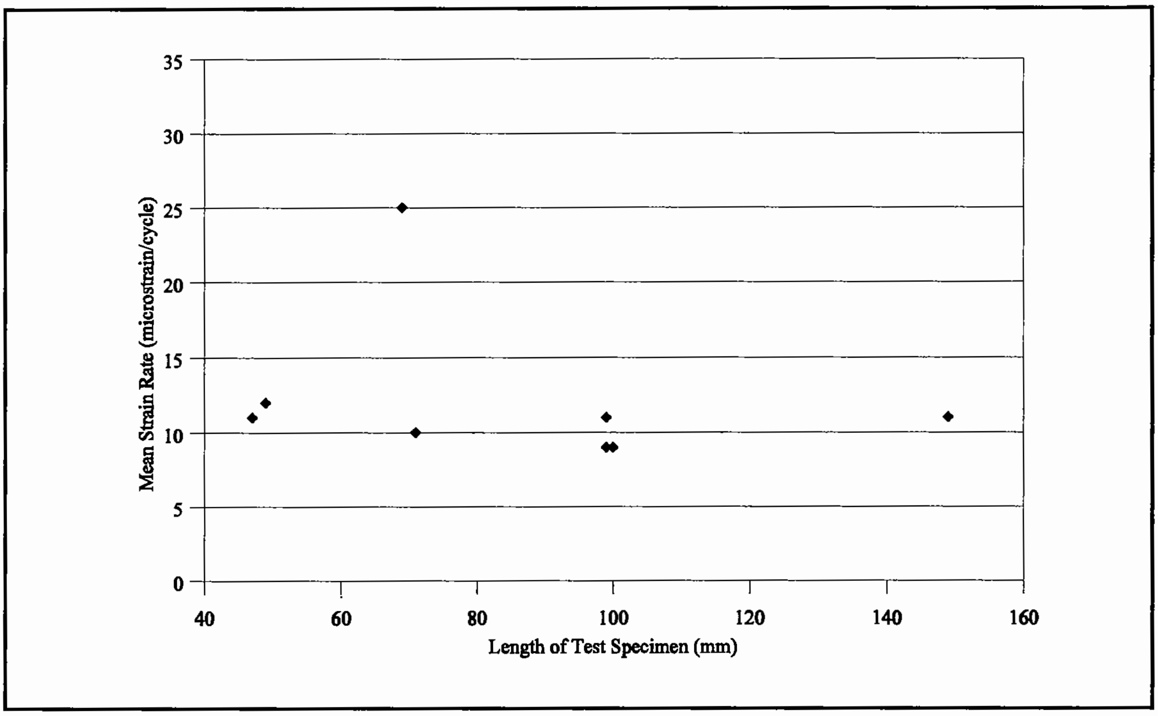


Figure 6.9 Effect of Specimen Length on Mean Strain Rate
Asphaltic Concrete Mixture B0W : Void Content Range 4.0 to 5.6%

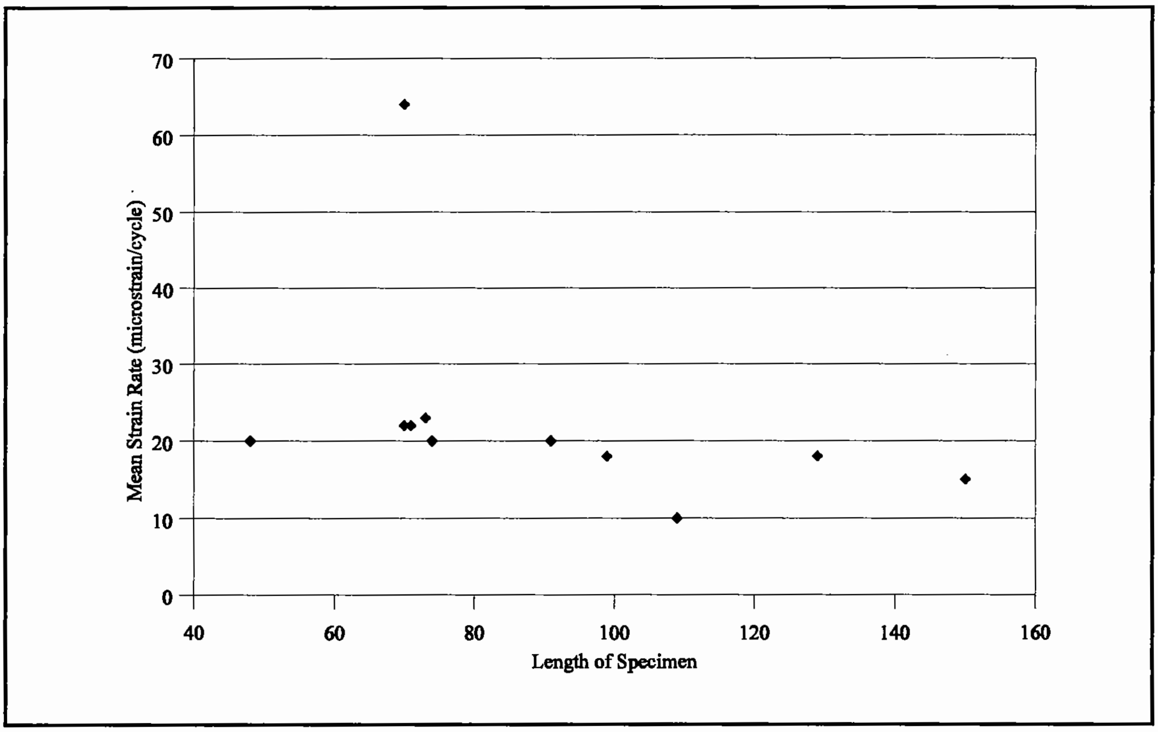


Figure 6.10 Effect of Specimen Length on Mean Strain Rate
Asphaltic Concrete Mixture B0W : Void Content Range 7.4 to 9.4%

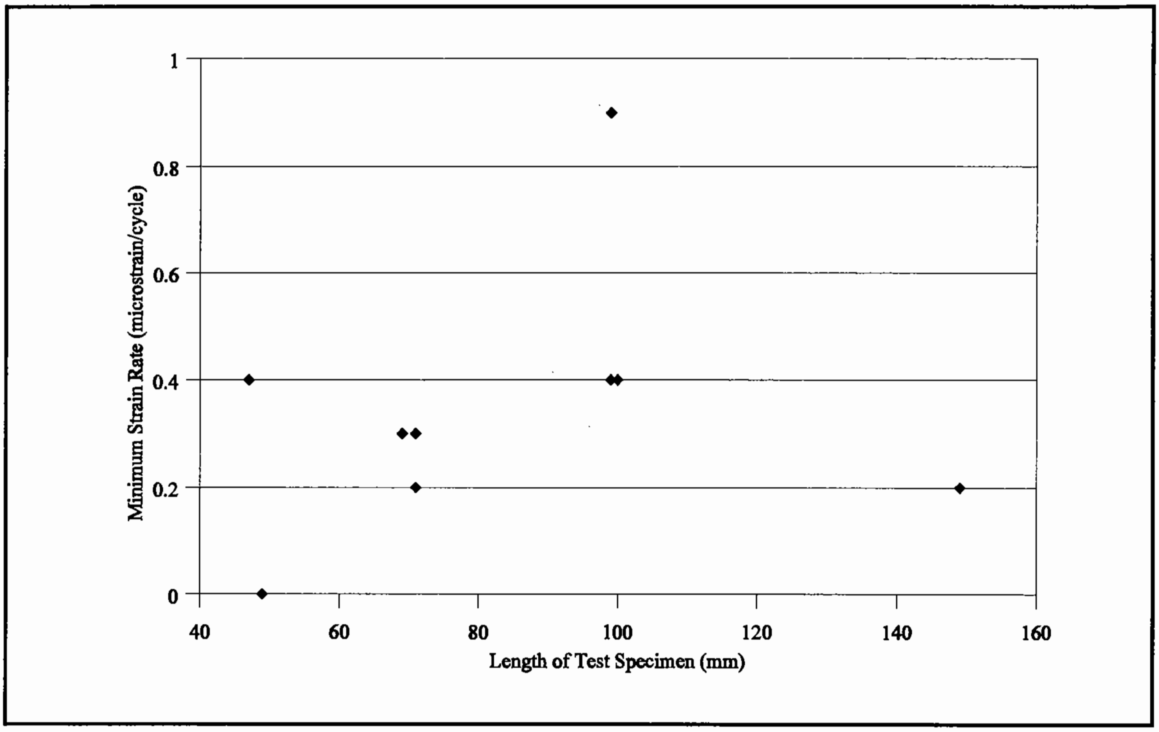


Figure 6.11 Effect of Specimen Length on Minimum Strain Rate
Asphaltic Concrete Mixture B0W : Void Content Range 4.0 to 5.6%

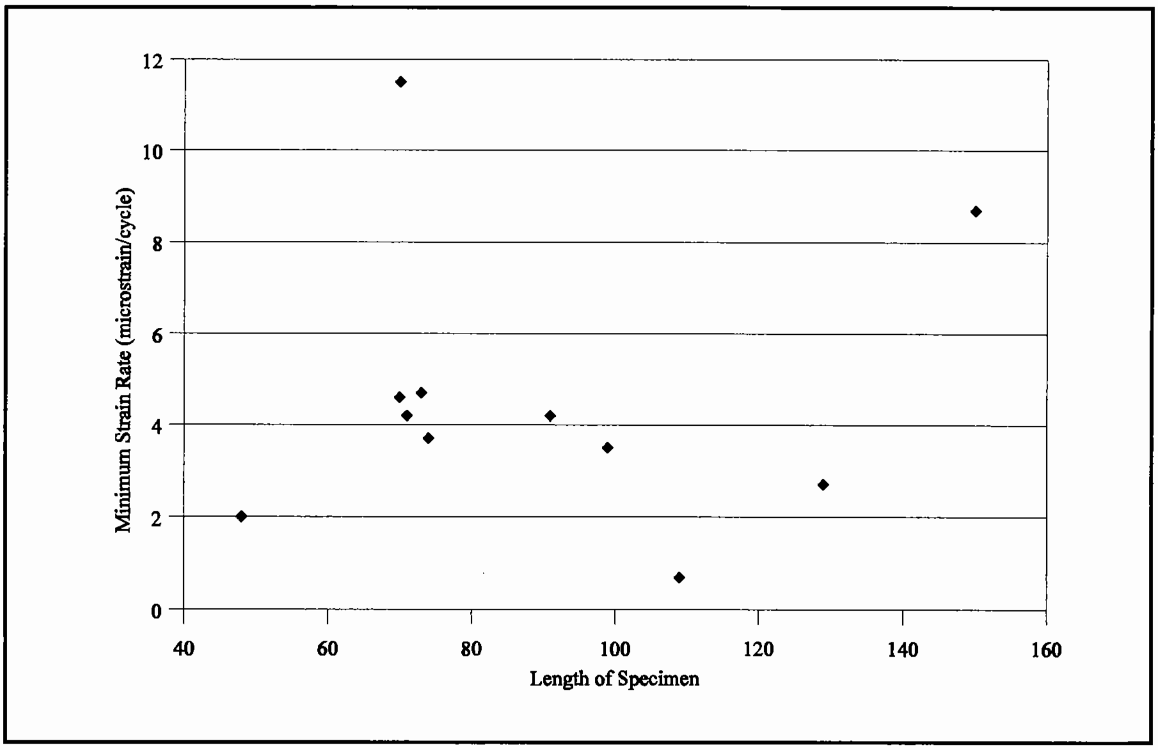


Figure 6.12 Effect of Specimen Length on Minimum Strain Rate
Asphaltic Concrete Mixture B0W : Void Content Range 7.4 to 9.4%

Similar insensitivity to specimen length was shown by the results of RLA tests on 82mm diameter cores of another 25mm asphaltic concrete. The results are given in Table 6.8 and plots of mean and minimum strain rate against specimen length are shown in Figures 6.13 and 6.14 respectively.

Table 6.8 RLA Test Results for 25mm Asphaltic Concrete : (Mixture B1W)

Specimen Reference	Length (mm)	Aspect Ratio	V_V (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain/ cycle)	Minimum Strain Rate (microstrain/ cycle)
B1W 325B	58	0.71	5.9	1.65	17	1.9
B1W 325A	98	1.20	6.3	1.95	18	2.6
B1W 247A	150	1.83	6.7	2.28	19	3.9
B1W 426B	48	0.59	7.9	4.48	33	7.6
B1W 226B	70	0.85	8.4	3.52	26	6.5
B1W 326B	110	1.34	8.4	4.85	31	8.9
B1W 226A	69	0.84	8.6	3.48	26	6.2
B1W 426C	47	0.57	9.3	3.42	31	5.8
B1W 426A	47	0.57	9.7	4.10	33	7.1

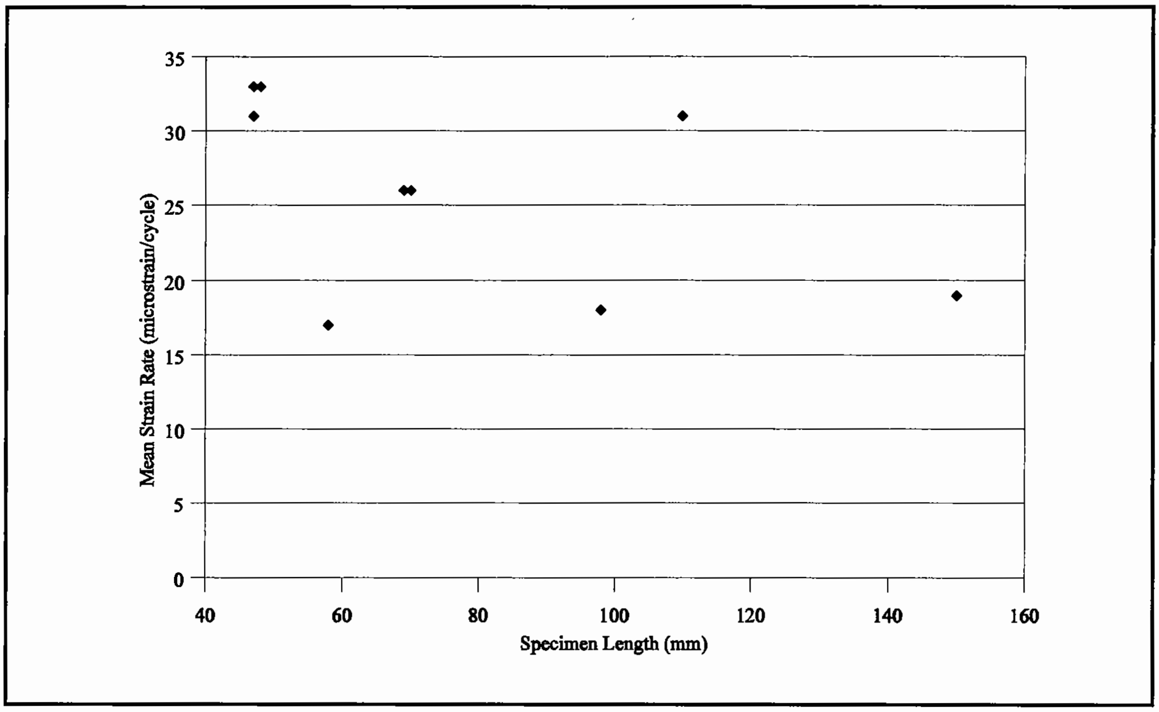


Figure 6.13 Effect of Specimen Length on Mean Strain Rate Asphaltic Concrete Mixture B1W

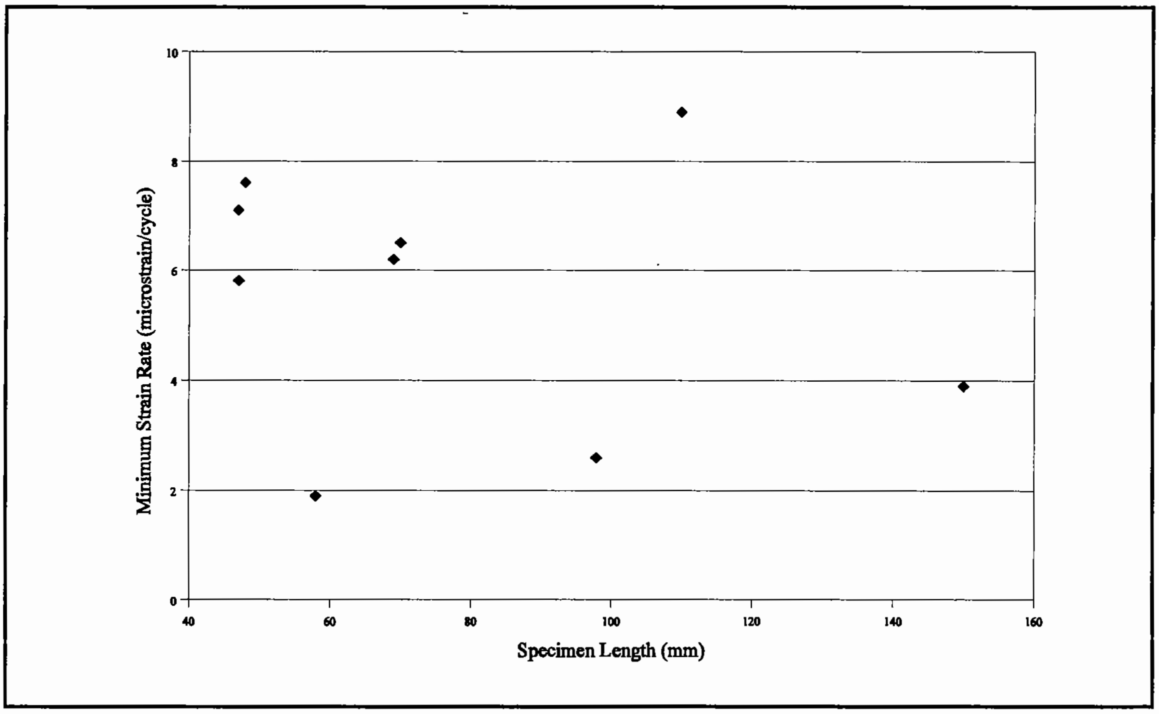


Figure 6.14 Effect of Specimen Length on Minimum strain Rate Asphaltic Concrete Mixture B1W

Tests were also carried out on cores of a 200 Pen 20mm DBM basecourse taken from the slab manufactured at a limestone quarry during the investigation of the effect of compaction method. These cores were 150mm in diameter and some were simply trimmed to length in preparation for testing while 100mm diameter cores were drilled from others to enable a wider range of aspect ratios to be obtained. The results of the RLA tests, which were carried out at 30°C under an axial stress of 100 kPa are presented in Table 6.9 below. There is no indication of any relationship between mean strain rate and aspect ratio in the plot shown in Figure 6.15. Figure 6.16 shows a plot of minimum strain rate against aspect ratio and the data for the lower void content material presented in this figure does appear to show a trend of an increase in minimum strain rate with an increase in aspect ratio, but this is not borne out by the data for the higher void content material, and it would appear from the scatter of the data that the void content has not had a significant effect on this parameter. The overall indication is that there is no relationship between performance and aspect ratio, which is consistent with the earlier results.

Table 6.9 RLA Test Results 200 Pen 20mm DBM Basecourse

Specimen Reference	Length	Aspect Ratio	V_V (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain/ cycle)	Minimum Strain Rate (microstrain/ cycle)
M53 B3T	46	0.31	2.8	3.17	48	1.6
M53 F4T	53	0.54	3.7	3.47	52	2.3
M53 B3M	70	0.47	3.8	2.33	33	2.0
M53 E1T	29	0.29	3.9	3.15	44	1.8
M53 B1T	100	0.67	4.2	3.75	47	3.5
M53 C1	150	1.00	5.1	-	47	7.2
M53 E1M	100	1.01	5.4	3.58	36	6.1
M53 B3B	50	0.33	8.6	1.93	23	1.9
M53 B1B	50	0.33	8.6	2.45	24	3.5
M53 F4B	56	0.57	8.9	2.61	26	4.0
M53 F4M	55	0.56	9.3	1.05	15	0.4

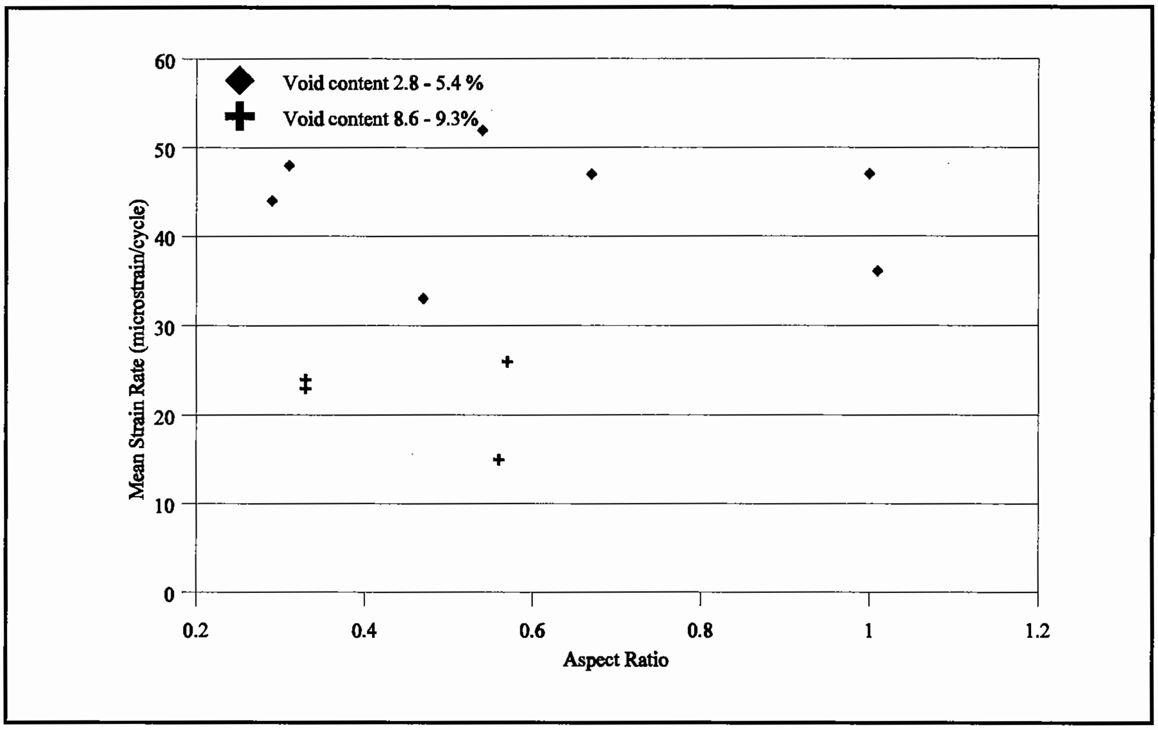


Figure 6.15 Effect of Aspect Ratio on Mean Strain Rate
200 Pen 20mm DBM Basecourse

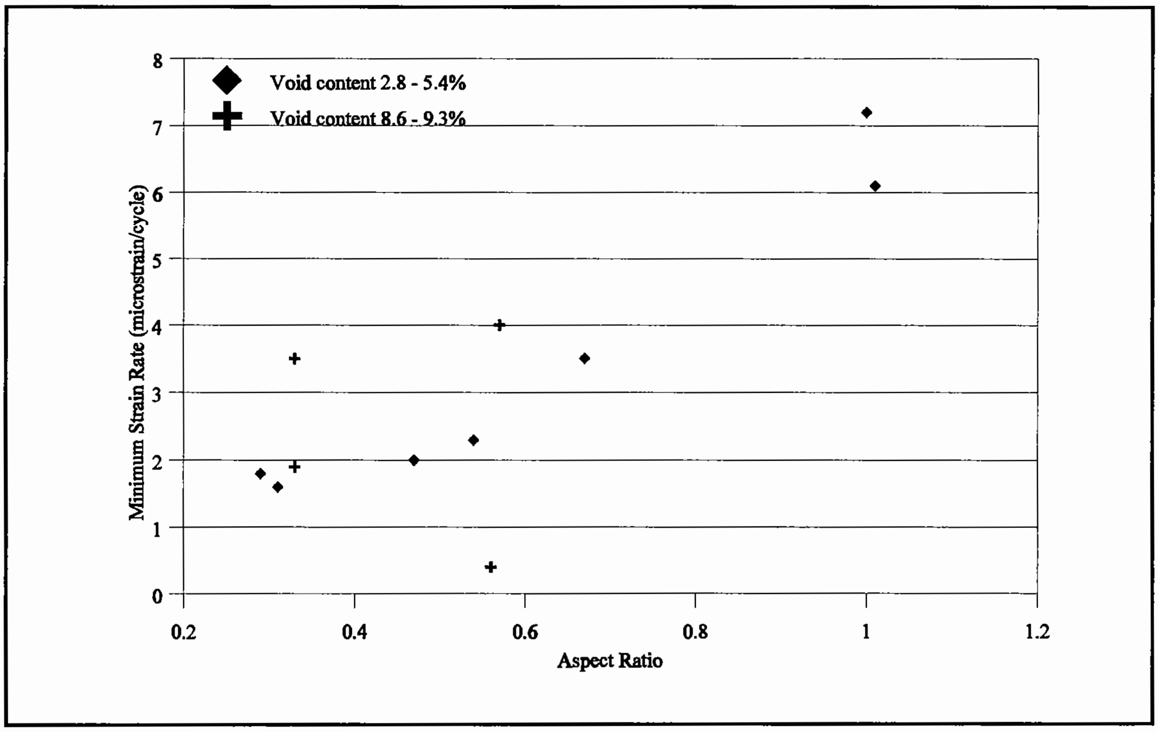


Figure 6.16 Effect of Aspect Ratio on Minimum Strain Rate
200 Pen 20mm DBM Basecourse

On the basis of the results of these series of tests it was considered that no further investigation was merited. Specimens of 25mm material cut to 47mm in thickness, and 20mm material cut to 46mm were tested without apparently affecting the result, so it would seem that neither specimen length nor aspect ratio, over the ranges considered, have any significant effect. This is consistent with the findings of de Hilster and van de Loo for the static creep test (108), provided that the ends of specimens are lubricated to minimise lateral restraint. However, it is recommended that specimens with a thickness at least twice the value of the maximum aggregate size be used where possible to minimise the potential for bridging of large aggregate particles between the loading platens.

6.6 Summary

Based on the outcome of the investigations reported in this Chapter it is recommended that RLA testing be carried out at an axial stress of 100 kPa with a pre-test conditioning regime of 5 kPa static load applied for a duration of 2 minutes to seat the platens on the specimen.

There is no evidence to suggest any influence of specimen length or aspect ratio on the result of the test when the ends of the specimens have been carefully prepared and lubricated using graphite powder only. However, it is considered that as a matter of good practice the use of thin specimens should be avoided wherever possible.

The reporting of both mean and minimum strain rates is suggested as an effective means of characterising the performance of a specimen.

CHAPTER SEVEN
COMPACTION STUDY

7.1 Introduction

Nevitt (136) has stated that to consider compaction as densification is an oversimplification since the process involves the application of forces which may vary, depending on the technique employed, in magnitude, direction and duration with consequent effect on the resulting structure. Studies in which the structure of the aggregate has been examined in detail (63,64,65) have revealed that preferred orientation of aggregate particles can vary with the method of compaction. Aggregate structure has a strong influence on resistance to permanent deformation, as demonstrated by the results presented in Chapter Five, and it has been found that the use of different laboratory compaction techniques has produced differences in performance in tests for the measurement of resistance to permanent deformation (68,70,137,138).

7.2 Related Research

Method of specimen compaction is, therefore, an important consideration in the assessment of resistance to permanent deformation and several studies have been undertaken to compare different techniques. In the development of the NCHRP AAMAS mixture design procedure (44) the properties of samples prepared by five laboratory compaction devices were compared with those of field cores. Cores were taken from pavement sections shortly after construction when the materials had been exposed to only "minimal" traffic, and bulk mixture samples were taken during the production run. These samples were, however, allowed to cool and then reheated for the manufacture of specimens in the laboratory, on occasion more than one month later, a procedure which is likely to alter significantly the properties of the binder and also, therefore, the mixture (139). Furthermore, the comparison of permanent deformation properties between field and laboratory compacted specimens in this study, which ultimately recommended the use of gyratory compaction, was based entirely on indirect tensile testing which is not a widely recognised or used configuration for assessing resistance to permanent deformation.

An evaluation of laboratory compaction procedures was also undertaken at UCB during the initial phase of SHRP contract research for the development of mixture tests for the Superpave system (45). This exercise, the elements of which pertinent to resistance to permanent deformation have been reported by Sousa et al (69), was restricted to the comparison of three laboratory devices - gyratory, kneading and rolling wheel - with only limited data from field samples. These field samples comprised cores taken from two roads which had been in service for several years and bagged samples of the loose mixture, taken at the time of construction, which were reheated and compacted in the laboratory. The mixtures tested were asphaltic concretes, but the vast majority (98%) of tests to measure permanent strain characteristics were carried out under creep loading, axial or shear, which in the light of the results presented in Chapters Three and Five may not give the best indication of the effects of aggregate structure. The outcome was a recommendation for the use of rolling wheel compaction, though SHRP eventually selected gyratory compaction for Superpave.

Nunn (71) and Button et al (140) have reported investigations specifically intended to compare the performance of laboratory and site compacted specimens, though in both these exercises laboratory specimens were manufactured from samples of the constituent materials mixed in the laboratory and not from plant-mixed samples taken at the time of production and placing. In the case of the work described by Nunn there were significant differences in the void contents of specimens obtained by the different methods.

Similar work by Harvey et al (141) was designed to compare the properties of cores from three sites with the corresponding laboratory compacted specimens. However, mixture samples were obtained from only two of the sites and it is assumed, though not explicitly stated, from the dates of construction that these samples required reheating for compaction in the laboratory. Moreover, cores from two of the sites were taken after the material had been heavily trafficked for some months.

7.3 Purpose of the Compaction Study

It was apparent that there was a shortage of data which would permit a direct comparison of the properties of specimens of exactly the same material, mixed and handled in the same way and subjected to both laboratory and site compaction techniques. The opportunity to obtain such data existed under the LINK Bitutest project since the active participation of several industrial organisations would provide access to sites where bituminous materials were being laid, as well as the facilities for sampling of the loose mixture and coring from the completed pavement. The participation of these organisations also provided a reason for carrying out such a study, since concerns over the use of the PRD equipment (62) for routine specimen preparation had been expressed by several of their representatives, who had stressed the importance of being able to fabricate specimens representative of site compacted material to the development of harmonised European standards.

In addition to an evaluation of the PRD apparatus, the use of which for specimen compaction had been proposed in earlier research into the design of bituminous mixtures carried out at Nottingham University (142), there was also a need to establish the effect of method of compaction on the RLA test, which was essential if the test was to be used to make a quantitative assessment of materials, as would be the case in performance specification.

7.4 Field Trials

Preliminary studies in the laboratory had shown differences in performance in the RLA test between specimens compacted by Marshall hammer, roller compactor - as described in Chapter Three - and the PRD equipment, which confirmed the viability of further investigation. This was to be achieved by carrying out field trials in which plant-mixed material for the manufacture of PRD specimens would be sampled either at the plant during production or from a wagon en route to site. It was decided that in addition to the PRD specimens, slabs of material compacted by pedestrian roller should also be produced, a technique which had been successfully employed as a means of obtaining specimens for testing in earlier research (20).This would provide both an additional method of compaction for comparison in the study and, it was assumed, a reasonable substitute for site compacted material should cores from the road pavement not be available.

A large steel form was, therefore, designed and constructed for the production of slabs of material measuring 4250 x 850mm in plan and up to 250mm thick. The width was selected to accommodate a Bomag 65 vibrating twin drum roller, and the mould, which is shown assembled in Figure 7.1, was fabricated in sections for ease of transportation (Figure 7.2). A jig which could be easily located and secured on the mould was also fabricated to support the coring rig and ensure that its axis was normal to the surface of the slab. The facility to obtain a large number of cored specimens from the same mixture batch, compacted by equipment assumed to be representative of site compaction plant, meant that use of the formwork was also ideal for the manufacture of specimens for the investigation of the effect of test variables reported in Chapter Six.

Since the success of this study required the cooperation and assistance of the project's industrial partners, guidelines for a suitable experiment were circulated early in the project to encourage response and participation. These guidelines are presented in Appendix D and the field trials which were undertaken are described in the following sections.

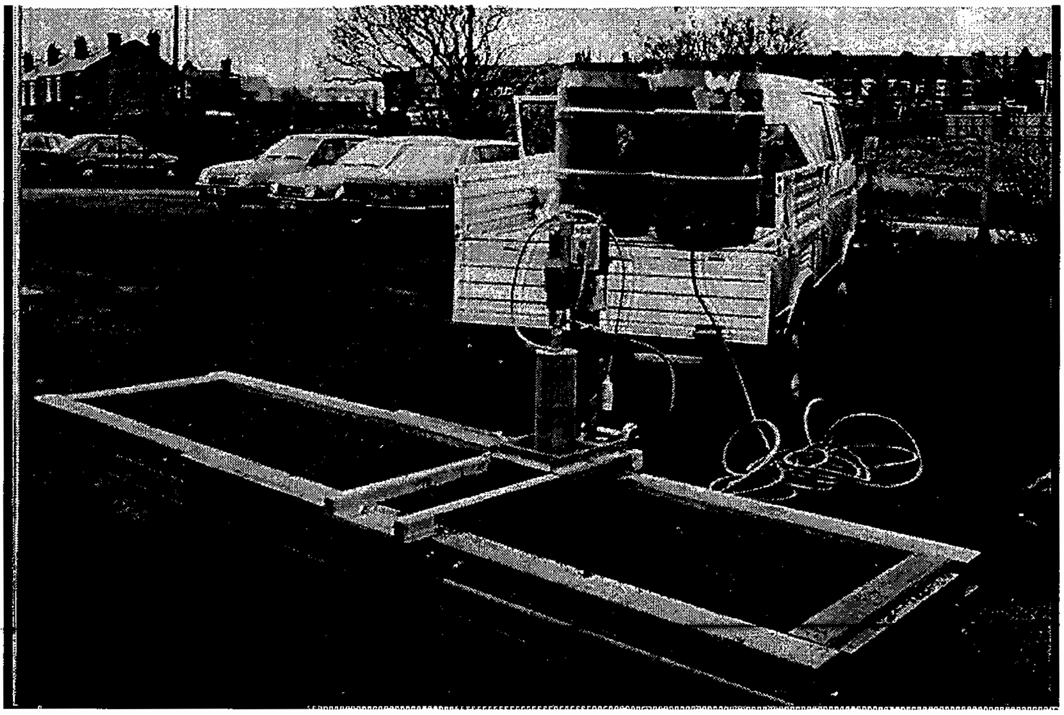


Figure 7.1 Portable Steel Formwork

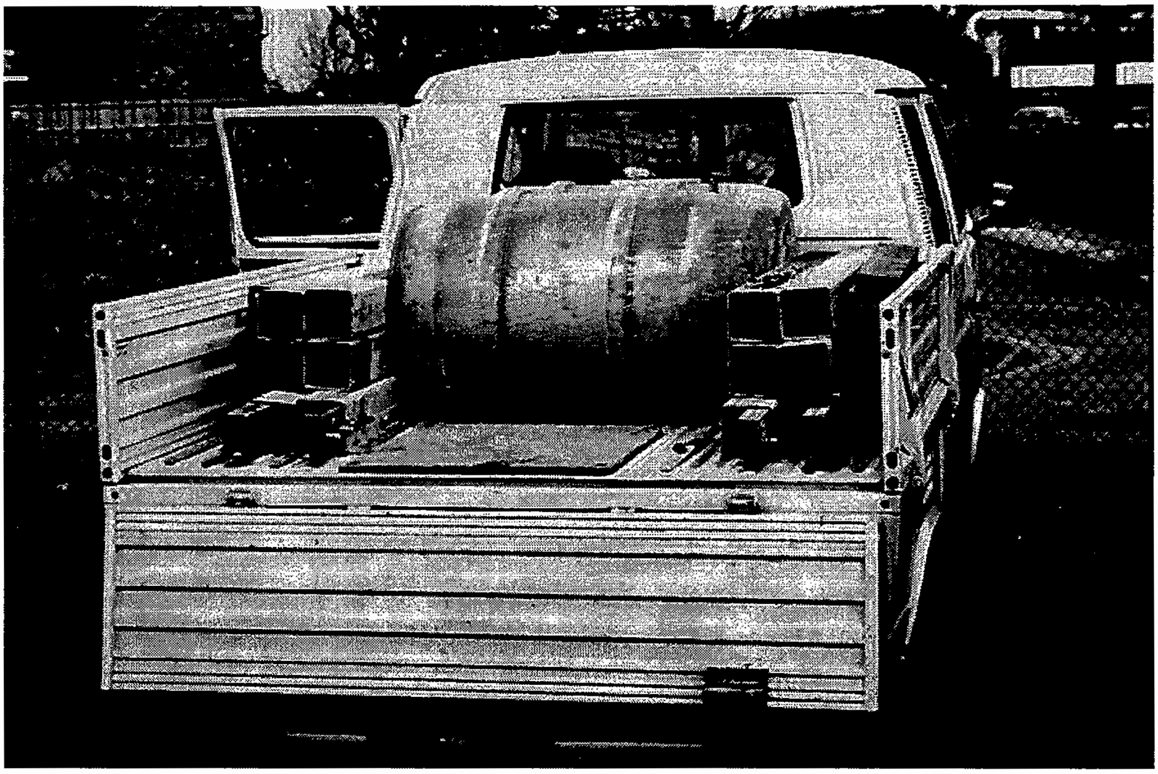


Figure 7.2 Formwork Dismantled for Transportation

7.5 Trials at Wimpey's Pant Quarry

7.5.1 Evaluation of 200 Pen 20mm DBM Basecourse

First use of the formwork was at one of Wimpey Asphalt's plants at a limestone quarry in North Wales, where the scope of the work was limited to making a comparison between the properties of material compacted using the PRD equipment and the same material compacted in the formwork using a vibrating roller. Wimpey produced a single batch of 20mm DBM basecourse with 200 pen binder, which was transported from the batch plant by loading shovel. Loose material was sampled from the shovel and placed directly in 9 PRD moulds, while further samples were taken for compositional analysis by Wimpey's site laboratory. The remainder of the batch was placed in the formwork. The material was raked level by hand and compacted, of necessity, in a single lift approximately 200mm thick using the vibrating roller. The temperature of the mix just prior to commencing compaction was approximately 115°C. Twenty passes of the roller were made ; seventeen with vibration and three without. Following completion of work on the slab, the nine PRD specimens were compacted to target levels of PRD of 93, 97 and 100, following the guidelines given below which were developed in earlier research (142).

Compaction level (approximate)	Compaction time (seconds)	Compaction temperature (°C)
100 PRD	120	SP + 92
97 PRD	60	SP + 50
93 PRD	30	SP + 40

where : Compaction time is for each end of the specimen and SP is the ring and ball Softening Point of the bitumen.

Twenty eight 150mm diameter cores were cut from the slab two days after compaction, as illustrated in Figure 7.3.

Figure 7.3 Locations of Cores in Slab

Both PRD specimens and cores were trimmed to a length of 70mm in preparation for RLA testing. The material in the slab cores did not appear to have segregated and, though the bottom 30 - 40mm was badly voided and was discarded, it was possible to obtain two specimens from each. Determination of maximum theoretical density was by the Rice method and specimens were sealed during measurement of volume by immersion. Binder content was reported by Wimpey from compositional analysis as 4.3% by mass of mixture.

Slab cores and PRD specimens with similar void contents were selected and subjected to the RLA test in the NAT under the following conditions.

Test temperature	:	30°C
Compressive stress	:	100 kPa
Load pulse frequency	:	0.5 H_Z
Load pulse duration	:	1 second
Test duration	:	3600 cycles
Pre-test conditioning	:	10 minutes @ 10 kPa (static)

The RLA test results are presented in Table 7.1, together with the air void content of each specimen. Figure 7.4 shows the test plots from the upper slab cores and Figure 7.5 shows the plots for the PRD specimens with similar void contents (i.e. PB1, PB2 and PB3). Better performance is indicated for the PRD specimens. This is confirmed by the values of ultimate permanent strain, mean strain rate and minimum strain rate shown in Table 7.1.

The results for the lower slab cores are presented in Figure 7.6 while Figure 7.7 shows the plots for PRD specimens PA1, PA2 and PA3, these being of similar void content. The tabulated data are presented in Table 7.1. Again the PRD specimens exhibited better performance than the slab cores.

Table 7.1 RLA Test Results for 200 Pen 20mm DBM Basecourse

Specimen Source	Specimen Reference*	V_V (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain/cycle)	Min. Strain Rate (microstrain/cycle)
Slab Cores	SCA3T	2.9	2.40	36	1.5
	SCB2T	3.0	3.33	45	2.1
	SCD2T	3.1	2.83	42	1.6
	SCE3T	3.0	3.28	48	1.8
	SCF2T	2.8	2.50	39	1.4
	SCG3T	3.0	3.59	48	1.8
	SCA3B	4.5	1.56	23	1.0
	SCB2B	4.7	1.48	22	1.0
	SCD2B	4.8	1.26	20	0.9
	SCE3B	4.7	1.61	23	0.9
	SCF2B	4.9	1.33	20	0.9
	SCG3B	4.7	1.66	26	1.0
PRD	PB1	3.5	1.03	20	0.1
	PB2	2.3	1.84	34	0.5
	PB3	2.2	1.53	26	0.7
	PA1	3.8	1.05	18	0.5
	PA2	5.8	1.07	15	0.3
	PA3	4.8	0.86	15	0.5

* T denotes upper slab/lift cores, B denotes lower slab cores

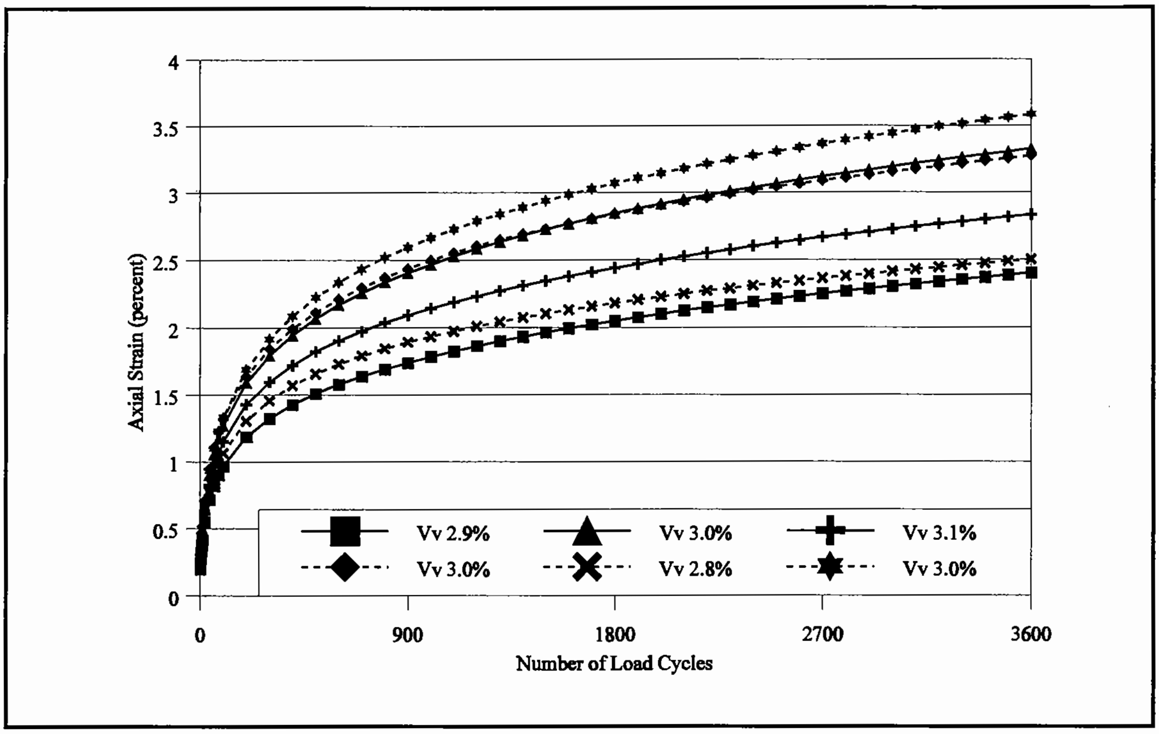


Figure 7.4 200 Pen 20mm DBM Basecourse : Upper Slab Cores

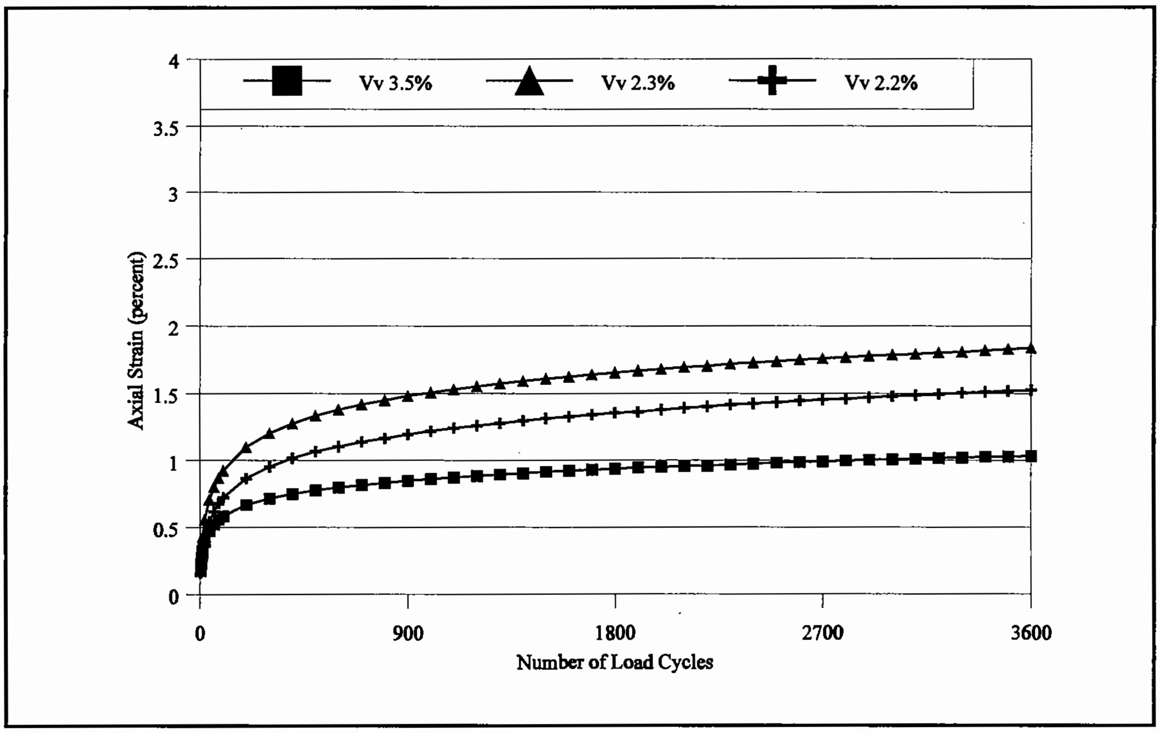


Figure 7.5 200 Pen 20mm DBM Basecourse : PRD Specimens (Low Voids)

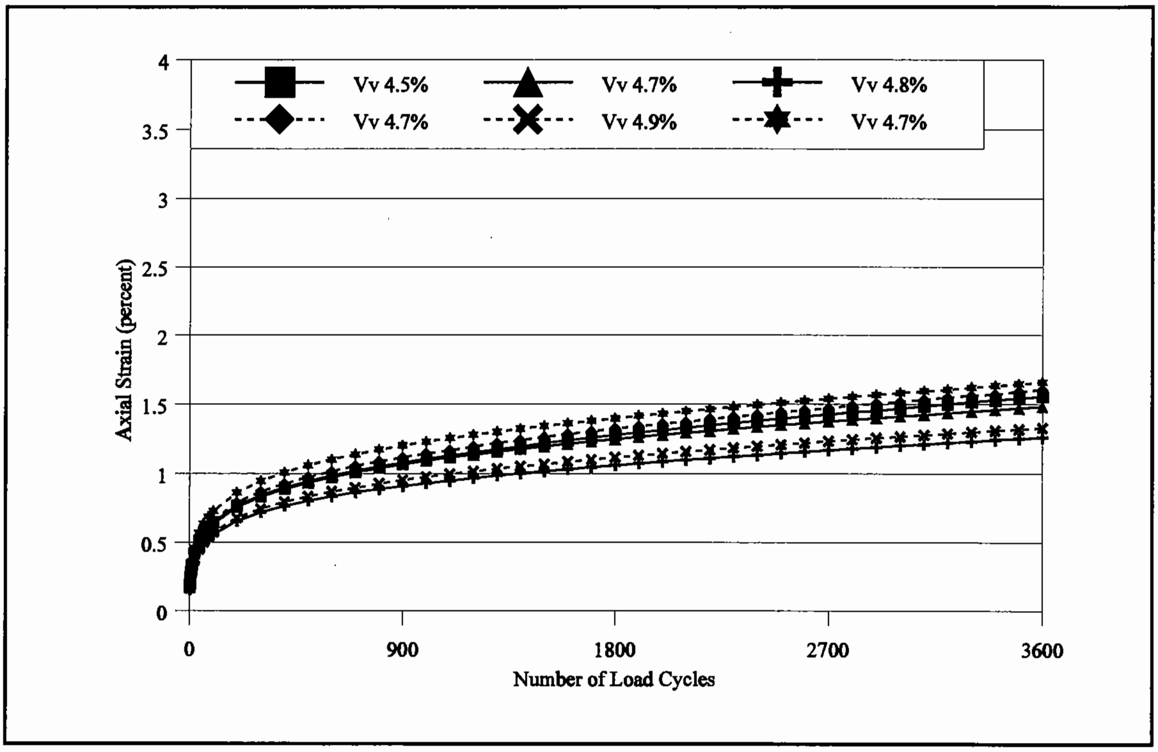


Figure 7.6 200 Pen 20mm DBM Basecourse : Lower Slab Cores

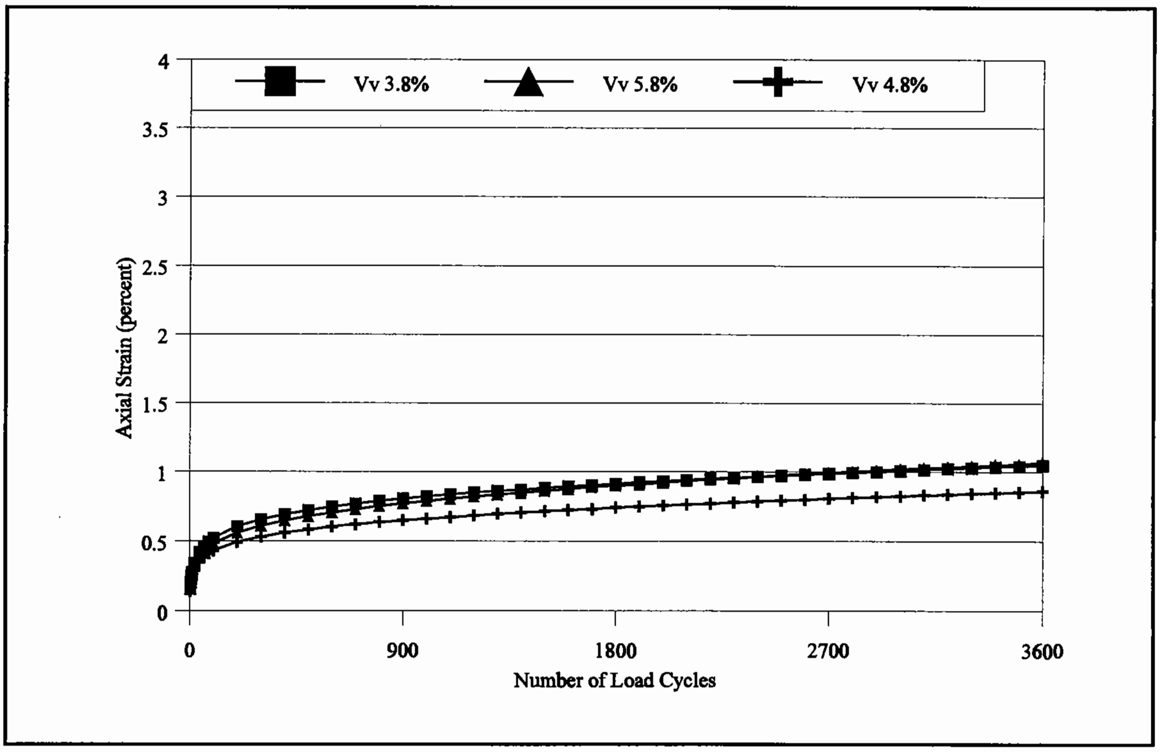


Figure 7.7 200 Pen 20mm DBM Basecourse : PRD Specimens (High Voids)

It is of interest to note that the material with a void content in the region of 5% performed better for both PRD specimens and cores than the material with a void content of around 3%, possibly indicating insufficient void space to accommodate the binder/fines mortar in these latter specimens with resulting loss of frictional contact between aggregate particles (see Chapter Two). The effects of compaction to very high densities are illustrated by the results for PRD specimens PC1, PC2 and PC3 which are given in Table 7.2. These specimens were compacted to refusal (100 PRD) and their performance is considerably worse than those compacted by the same means but with higher void contents.

Table 7.2 RLA Test Results : 200 Pen 20mm DBM PRD Specimens Compacted to Refusal

Specimen Reference	V_V (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain / cycle)	Min. Strain Rate (microstrain / cycle)
PC1	1.4	2.79	37	1.4
PC2	1.2	2.51	35	1.3
PC3	1.5	3.24	46	0.9

7.5.2 Evaluation of 100 Pen 20mm DBM Basecourse

The exercise on 200 Pen DBM, described above, was repeated at the same quarry on a 20mm DBM basecourse with 100 Pen binder. This material, the binder content of which was determined from compositional analysis by Wimpey as 4.7%, came from a normal production run and was taken from a truck just after loading at the batch plant. The slab was compacted with twenty two passes of the roller ; seventeen with vibration and five without. The temperature of the mixture just prior to commencing rolling was approximately 140°C.

Results for the RLA testing on cores and PRD specimens, carried out under the same test conditions as for the 200 Pen material, are presented in Table 7.3, together with the void contents of the specimens which were determined by the same means as for the 200 Pen material.

Table 7.3 RLA Test Results for 100 Pen 20mm DBM Basecourse

Specimen Source	Specimen Reference	V_V (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain/cycle)	Min. Strain Rate (microstrain/cycle)
Slab Cores	SC2A3T	4	1.34	17	0.6
	SC2B2T	5.2	1.2	14	0.6
	SC2C3T	4.7	1.06	14	0.4
	SC2D2T	5	0.96	14	0.4
	SC2E3T	4.7	1.2	16	0.4
	SC2F2T	3.2	1.22	16	0.3
	SC2G3T	4.7	1.39	18	0.5
PRD	P3A1	3.7	0.54	10	0.1
	P3A2	3.1	0.78	14	0.1
	P3A3	2.2	0.54	9	0.1
	P3B1	0.8	0.69	11	0.3
	P3B2	1	1.16	17	0.5
	P3B3	0.8	1.26	16	0.4
	P3C1	0.9	2.35	23	1.6
	P3C2	0.3	1.84	18	1.2
	P3C3	0.5	1.76	17	1.3

The RLA test plots for the cores from the slab, which had void contents in the range 3.2% to 5.2%, are shown in Figure 7.8. The PRD specimens with the lowest densities had void contents in the range 2.2% to 3.7%, with an average value 1.5% less than the average for the slab cores. The test plots for these specimens (P3A1, P3A2 and P3A3), Figure 7.9, show better performance than the slab cores. This is also illustrated by the data in Table 7.3. It is also clear from this data that, at the very high levels of compaction achieved in the remaining PRD specimens resistance to permanent deformation decreases with decreasing void content. It may, then, be reasonable to assume that PRD specimens with void contents closer to the slab cores would have shown even better performance.

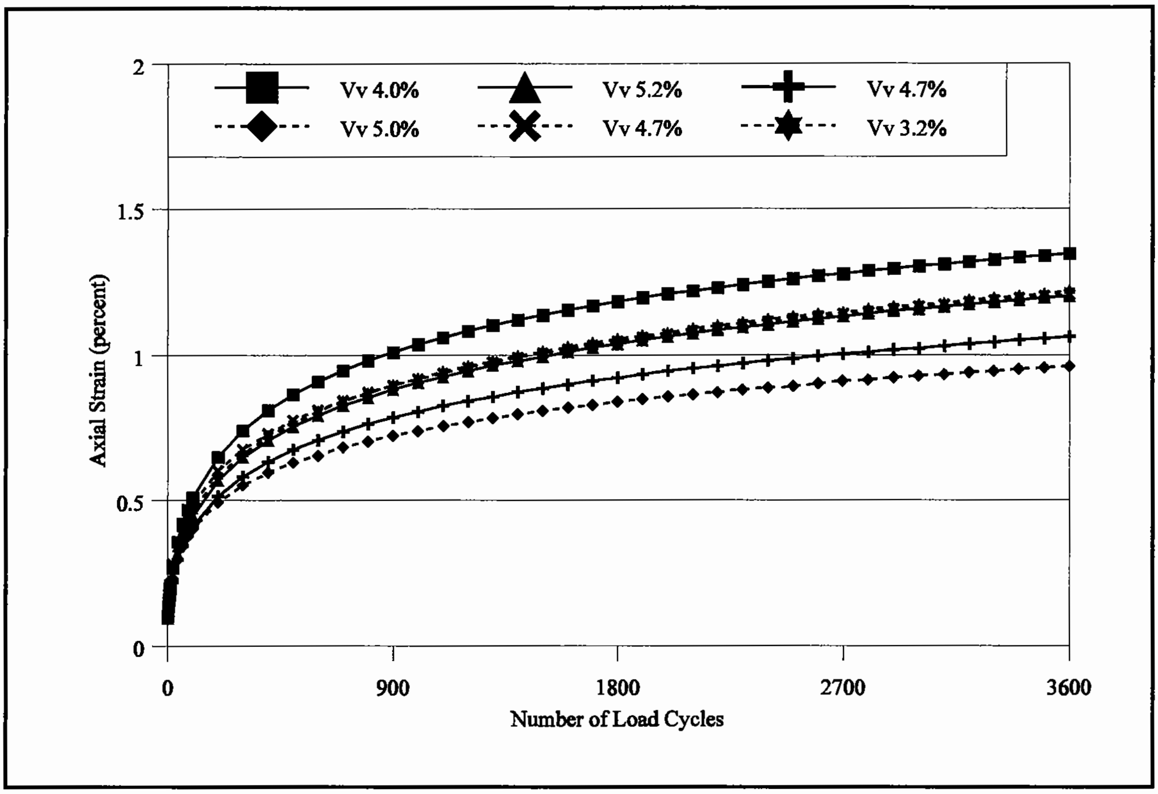


Figure 7.8 100 Pen 20mm DBM Basecourse : Slab Cores

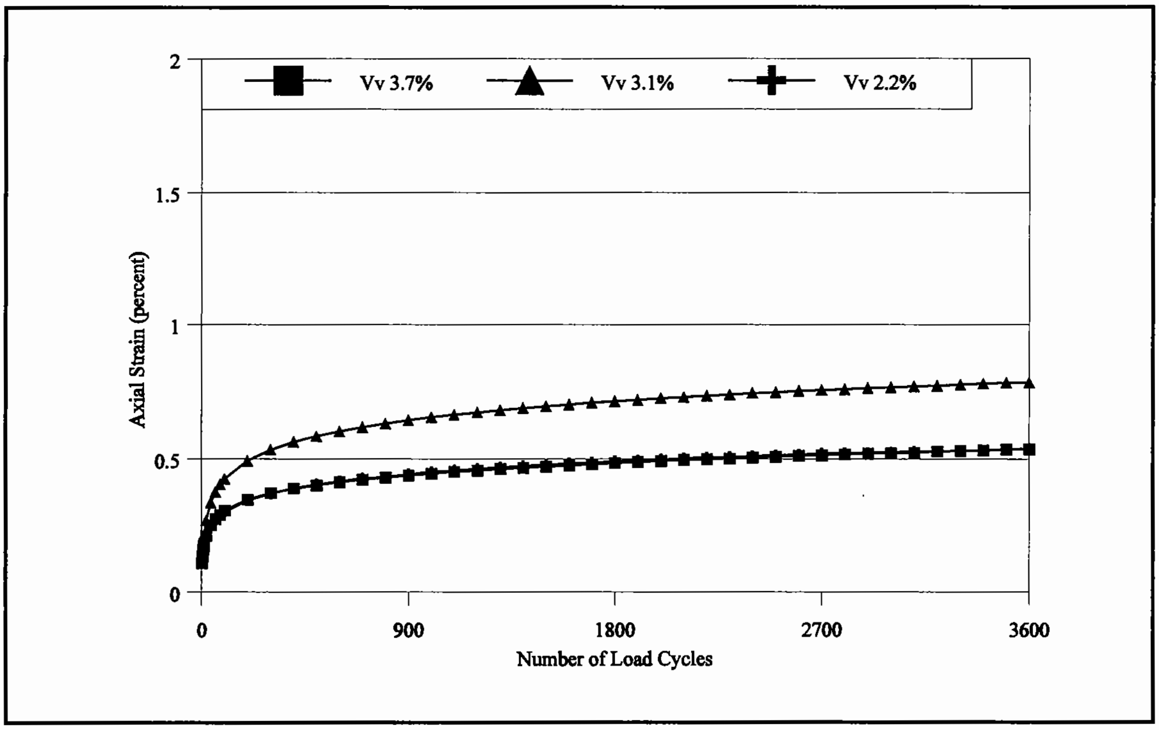


Figure 7.9 100 Pen 20mm DBM Basecourse : PRD Specimens

7.6 Trials Carried Out in Conjunction with Staffordshire County Council

7.6.1 Evaluation of 28mm DBM 50

Staffordshire CC responded to the request for assistance in this research and circulation of the experiment guidelines by organising an investigation on a 28mm DBM 50 roadbase. This material, which contains 50 Pen binder, was sampled from a truck en route to an M6 reconstruction site for the manufacture of PRD specimens, and 150mm diameter cores were subsequently taken at the location of that truckload in the pavement after compaction. The specimens obtained were then made available for RLA testing, which was carried out under the same conditions as used for the 20mm DBM basecourse obtained from Wimpey. Testing was restricted to those cores and PRD specimens most similar in void content, which was also determined in the same manner as for the Wimpey material. The binder content of the roadbase was reported by Staffordshire CC as 4.0% by mass of mixture.

The RLA test plots for the road cores are shown in Figure 7.10 while those for the PRD specimens are shown in Figure 7.11. These plots indicate better performance from the PRD specimens and this is reflected in the values of ultimate permanent strain, mean strain rate and minimum strain rate given in Table 7.4

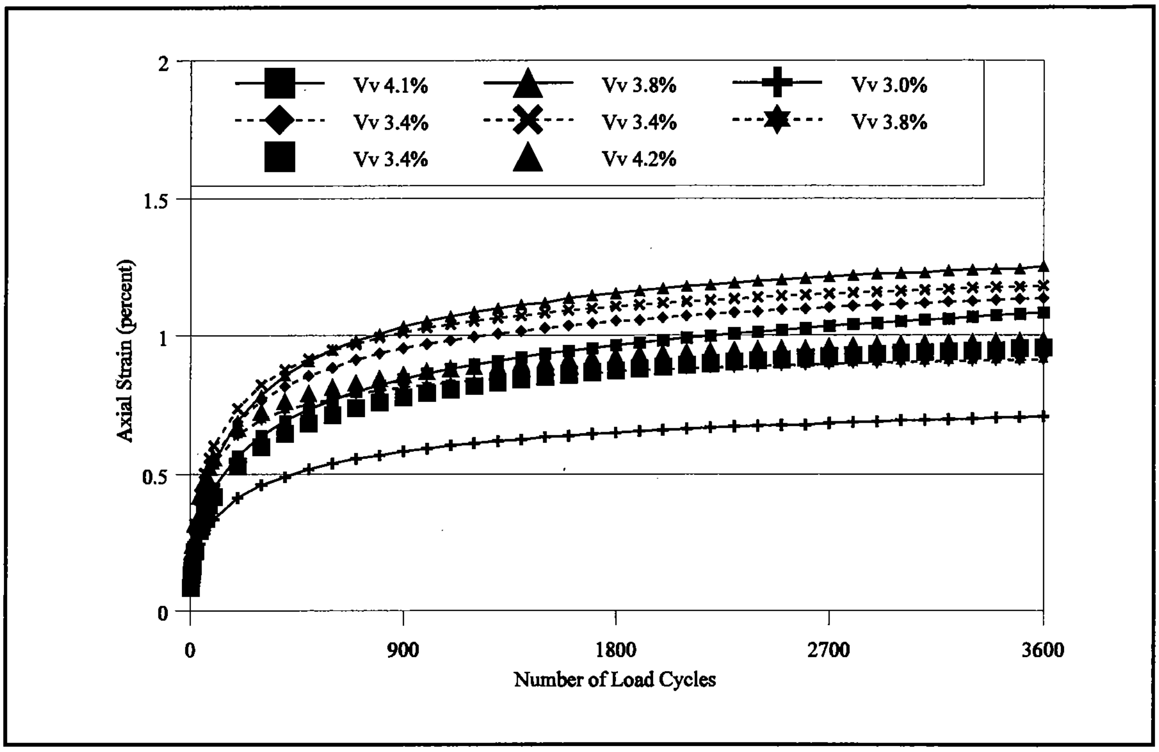


Figure 7.10 28mm DBM 50 : Road Cores

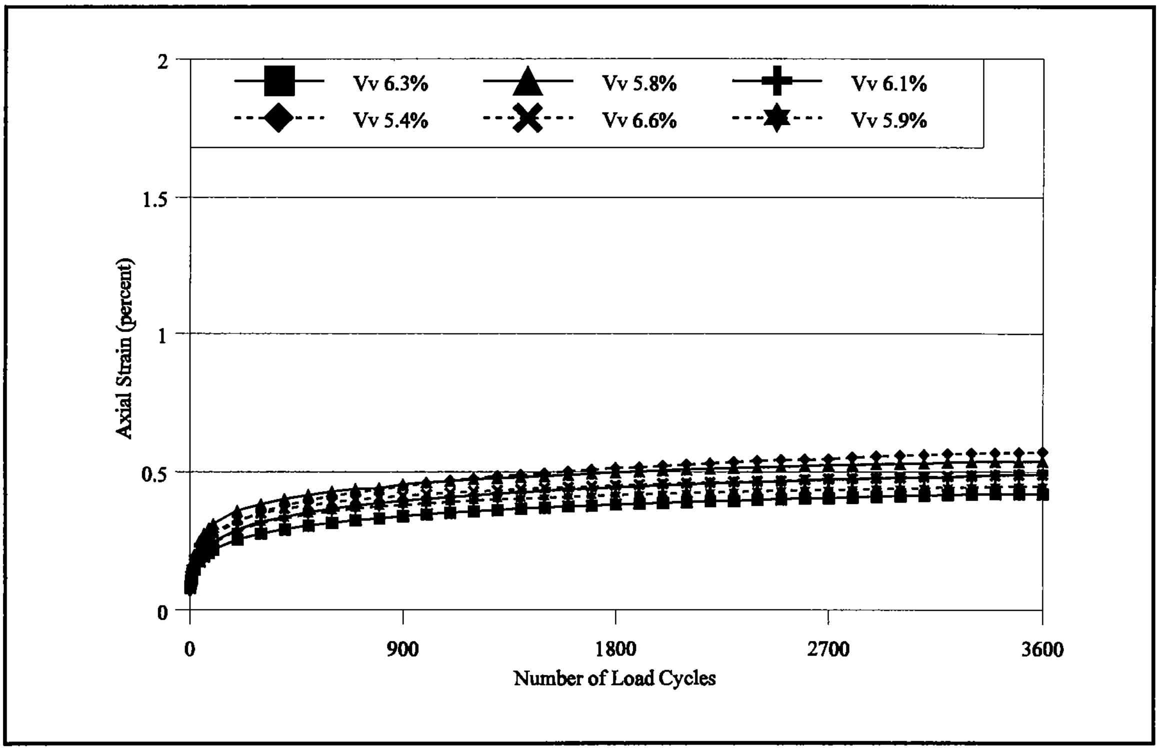


Figure 7.11 28mm DBM 50 : PRD Specimens

Table 7.4 RLA Test Results for 28mm DBM 50

Specimen Source	Specimen Reference	V_V (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain/cycle)	Minimum Strain Rate (microstrain/cycle)
Road Cores	M6C2	4.1	1.08	15	0.3
	M6C3	3.8	1.25	18	0.1
	M6C4	3.0	0.71	11	0.0
	M6C5	3.4	1.13	18	0.2
	M6C6	3.4	1.18	20	0.2
	M6C7	3.8	0.91	17	0.1
	M6C8	3.4	0.96	14	0.1
	M6C9	4.2	0.98	20	0.0
PRD	M6PC	6.3	0.42	7	0.0
	M6PE	5.8	0.54	10	0.0
	M6PJ	6.1	0.49	8	0.0
	M6PO	5.4	0.57	9	0.1
	M6PR	6.6	0.49	9	0.1
	M6PT	5.9	0.45	7	0.0

7.6.2 Further Evaluation of 28mm DBM 50

A repeat of the exercise described in the preceding Section was organised by Staffordshire CC so that on this occasion use could be made of the portable steel formwork for the manufacture of a slab. As before, a truck was routed to the M6 reconstruction site via Staffordshire CC's laboratory, where material was sampled to fill the steel formwork and nine PRD moulds and also for compositional analysis. The slab was compacted using a Bomag 65 roller, with vibration, and the PRD specimens were compacted to target density levels of 93, 96 and 100 PRD. Coring of that truckload from the compacted mat was carried out by Staffordshire CC and the 150mm diameter cores were delivered for testing. On receipt it was found that the cores comprised two lifts of roadbase and it was unclear which lift was from the sampled truckload. Differences in composition and RLA test performance subsequently identified which lift was of the correct material type. However, in view of the confusion, it cannot be reliably assumed that the cores were taken from the truckload sampled for the slab and PRD specimens. Nevertheless, as the field cores, slab cores and PRD specimens are of the same material the comparison is still worthwhile.

The slab cores were of sufficient length for two test specimens to be obtained from each. The specimens from all sources were 150mm in diameter and were trimmed to a length of approximately 70mm in preparation for testing. Rice gravities were determined and the specimens were sealed for measurement of volume by gravimetric means. The binder content of the material was determined from compositional analysis by both Staffordshire and Buckinghamshire County Councils, yielding an average value of 3.7% by mass of mixture. RLA testing was carried out as for the previous exercise, except that the test temperature was increased to 40°C in order to reduce binder stiffness with the aim of accentuating the role of the aggregate structure. The results are presented in Table 7.5.

It can be seen from the void contents that the density of the slab compacted material was significantly lower than that found in the road cores, and this inhibits reliable comparison of the properties of the specimens from these two. However, the PRD specimens compacted to refusal (Specimen references M6P3/1, M6P3/2, M6P3/3), did have similar void contents to the road cores. Figures 7.12 and 7.13 show the RLA test plots for those PRD specimens and the road cores respectively. The plots indicate little difference in performance, as do the values of ultimate permanent strain, mean strain rate and minimum strain rate which are given in Table 7.5

Table 7.5 RLA Test Results for 28mm DBM 50 (All tests carried out at 40° C)

Specimen Source	Specimen Reference*	V_V (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain/cycle)	Minimum Strain Rate (microstrain/cycle)
Road Cores	M6RC3T	3.5	0.55	10	0.1
	M6RC4T	4.7	0.81	15	0.1
	M6RC5T	3.3	0.70	14	0.0
	M6RC6T	5.1	0.88	19	0.2
Slab Cores	M6SCA2T	8.7	1.42	27	0.4
	M6SCB3T	8.7	1.33	24	0.5
	M6SCC2T	8.4	1.51	28	0.6
	M6SCD3T	8.5	1.36	25	0.1
	M6SCE3T	8.7	1.47	26	0.4
	M6SCF2T	9.3	1.26	22	0.4
	M6SCG3T	8.7	1.23	24	0.4
	M6SCA2B	12.9	1.21	16	1.01
	M6SCB3B	12.8	1.13	16	0.8
	M6SCC2B	12.2	0.87	13	0.7
	M6SCD3B	12.7	1.16	15	1.3
	M6SCE3B	11.8	1.00	16	0.3
	M6SCF2B	12.0	1.17	16	0.7
	M6SCG3B	13.1	1.26	16	1.1
PRD	M6P1/1	12.2	1.13	15	1.0
	M6P1/2	12.3	1.18	12	1.3
	M6P1/3	13	1.40	19	0.7
	M6P2/1	6.1	0.63	10	0.1
	M6P2/2	6.2	0.63	12	0.1
	M6P2/3	6.2	0.62	11	0.0
	M6P3/1	3.5	0.52	9	0.1
	M6P3/2	3.9	0.70	13	0.0
	M6P3/3	3.6	0.42	7	0.0

* T denotes upper slab/lift cores, B denotes lower slab cores

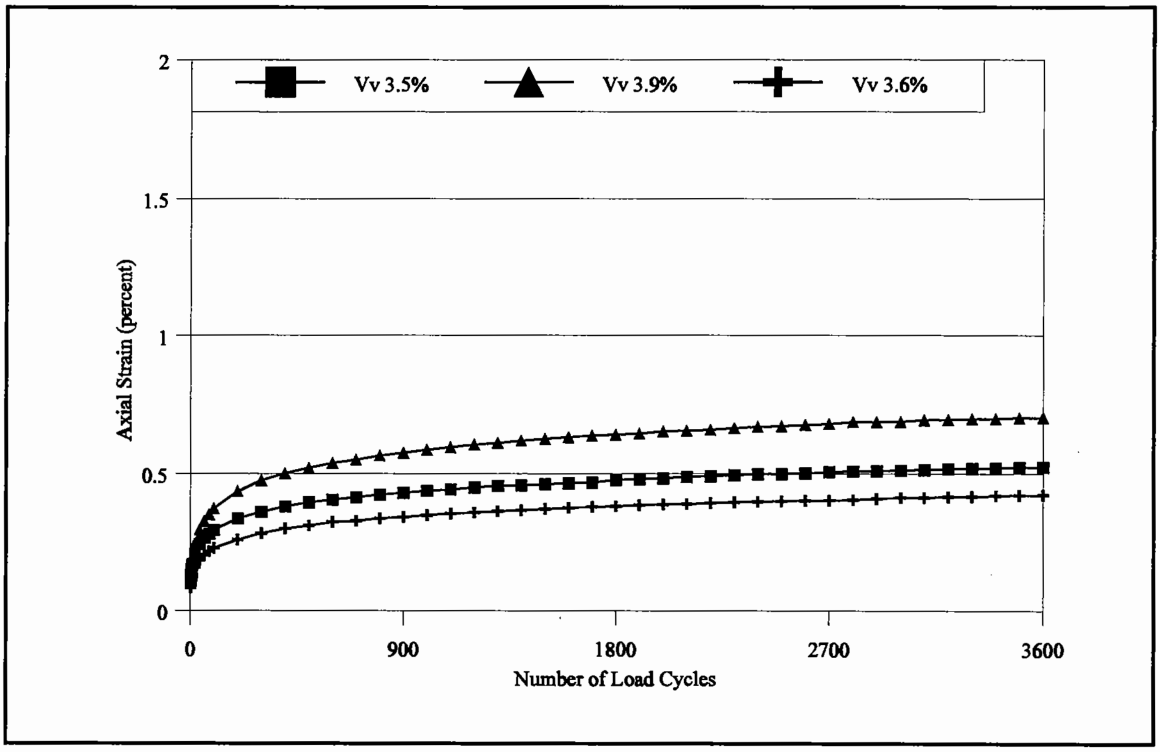


Figure 7.12 28mm DBM 50 : PRD Specimens (Low Voids)

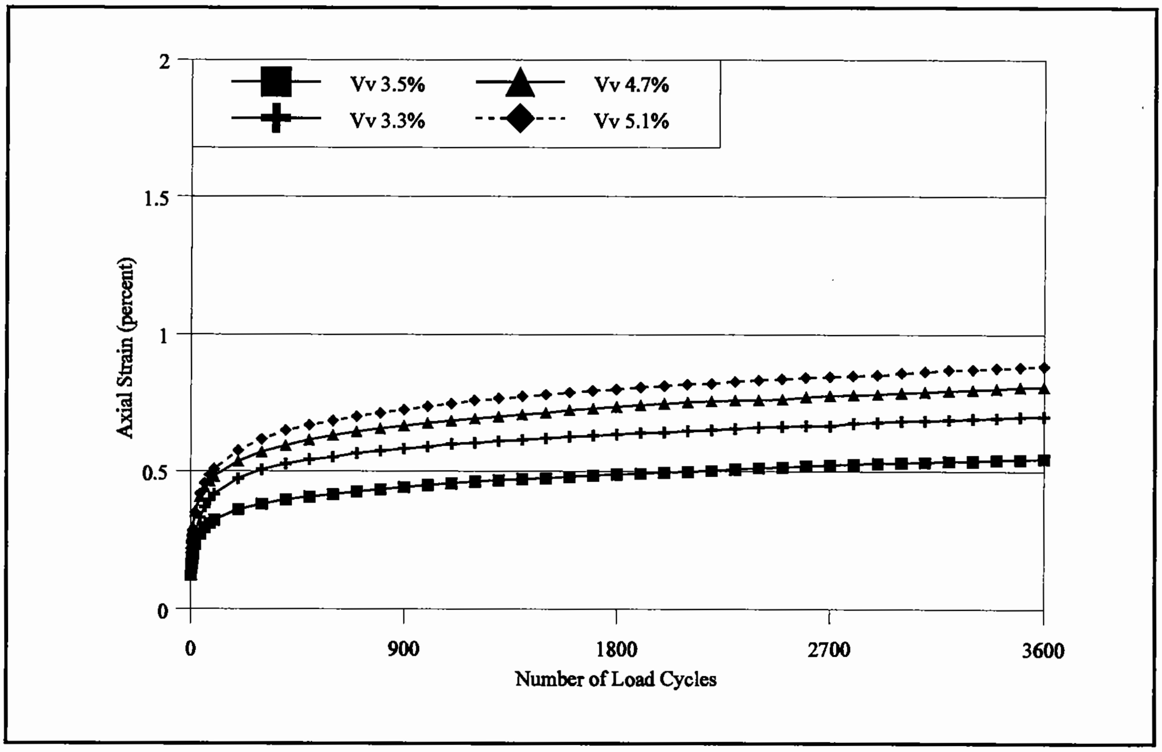


Figure 7.13 28mm DBM 50 : Road Cores

The PRD specimens targetted at 93 PRD (Refs M6P1/1, M6P1/2 and M6P1/3) and the lower slab cores were found to have similar, though unrealistically high, void contents. The RLA test plots, (Figures 7.14 and 7.15) and the data in Table 7.5 indicate that no effect of compaction method on performance can be discerned for these specimens. It may be that at such levels of void content the performance is more influenced by the degree of compaction than the method of compaction.

It is of interest to note that the upper slab cores which had lower void contents than those from the lower slab exhibited markedly poorer performance in terms of both ultimate strain and mean strain rate, although they could not be regarded as overfilled with void contents in the region of 8 - 9% .

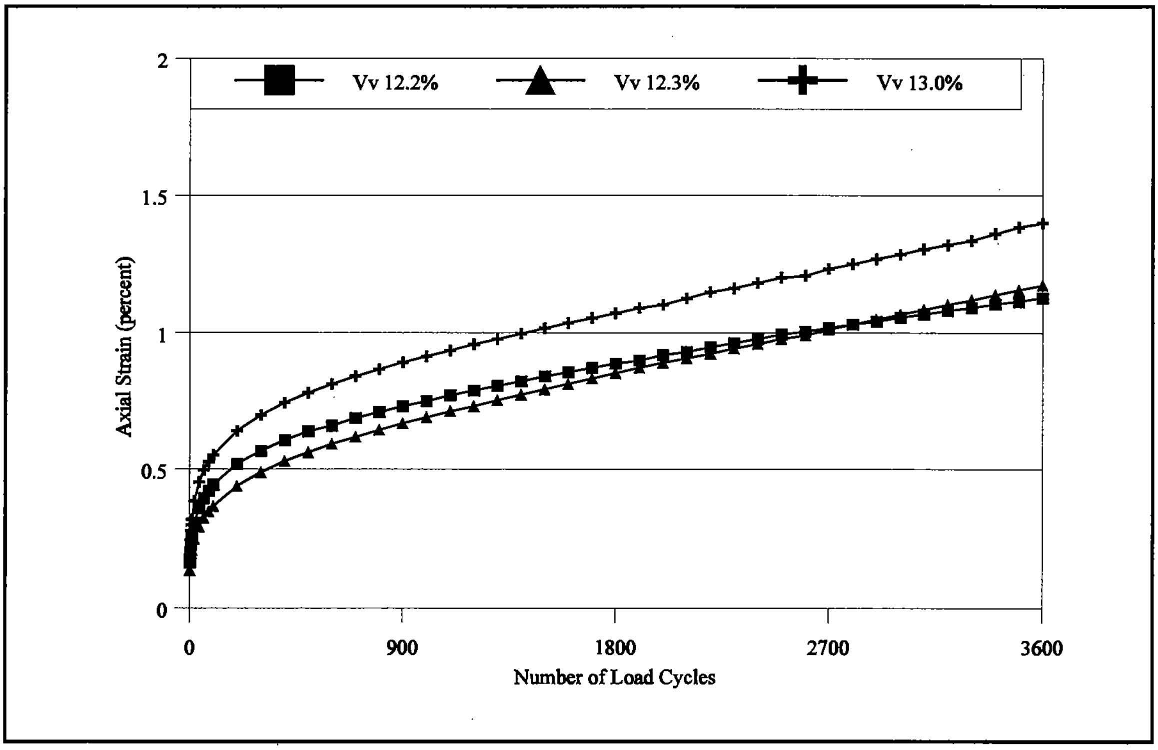


Figure 7.14 28mm DBM 50 : PRD Specimens (High Voids)

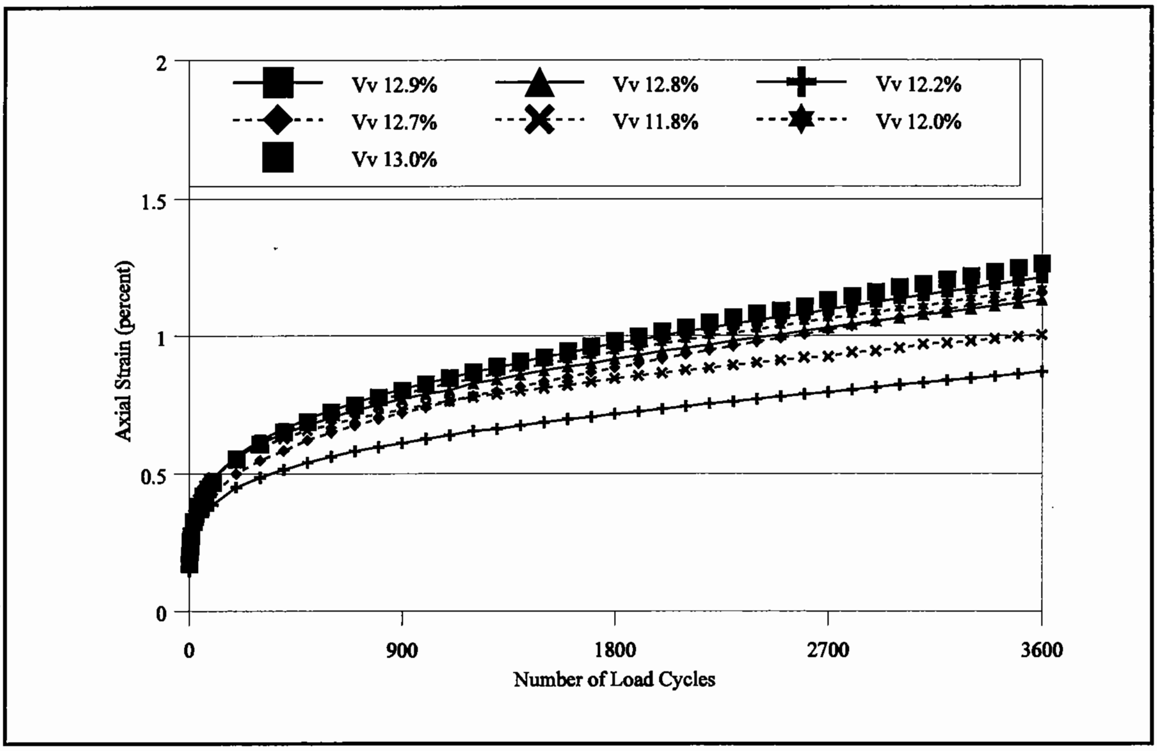


Figure 7.15 28mm DBM 50 : Slab Cores

7.7 Trials Carried Out in Conjunction with Mobil and Foster Yeoman

7.7.1 Evaluation of 100 Pen 28mm DBM Roadbase

A motorway reconstruction contract supplied by Foster Yeoman from their drum mix facility at Purfleet provided an excellent opportunity to make a comparison between site compacted material and the same material compacted by other techniques, since both the plant and the site were close to Mobil's Coryton refinery. Mobil's Research and Technical Services Laboratory is based at Coryton, where a laboratory roller compactor had recently been installed. These facilities, together with the opportunity to manufacture PRD specimens and set up the portable steel formwork, presented an ideal location for this work.

The first exercise at this location was carried out on a 100 Pen 28mm DBM roadbase, a truckload of which was diverted to Coryton en route from the drum mix plant at Purfleet to site. Approximately two tonnes of material was sampled for the manufacture of a slab in the steel formwork, eighteen PRD specimens and two small slabs using Mobil's laboratory roller compactor. The truck then continued to the site on the M25, where the location of the load was recorded in order that cores could be taken after compaction.

Figures 7.16 and 7.17 show the filling of the formwork and compaction of the slab. The temperature of the DBM immediately before compaction was 140°C. A Bomag 60 vibrating roller was used to compact the slab, which received 5 passes without vibration followed by 15 passes with vibration and finally a further 5 passes without. Eighteen PRD specimens were then manufactured, as shown in Figure 7.18, with temperatures at the start of compaction ranging from 130°C to 70°C and compaction times ranging from 120 to 30 seconds on each face.

Mobil's laboratory roller compactor, which is shown in Figure 7.19, is similar in principle of operation to the University's (Chapter Three), though is more sophisticated in the manufacture. Different levels of compactive effort were applied to the two slabs made in the roller compactor. In each case, the temperature immediately prior to compaction was 140°C.

No attempt was made to control or record the compaction regime on site since this would, by definition, be typical site compaction and this was the only requirement for the exercise.

Twenty eight, 150mm diameter cores were cut from the slab in the steel formwork at Coryton, and Foster Yeoman drilled six 150mm cores from the compacted mat on site. Four, 100mm diameter cores were cut from each of the two slabs made in the laboratory roller compactor.

Figure 7.16 Filling of Formwork

Figure 7.17 Compaction of Slab

Figure 7.18 Manufacture of PRD Specimens

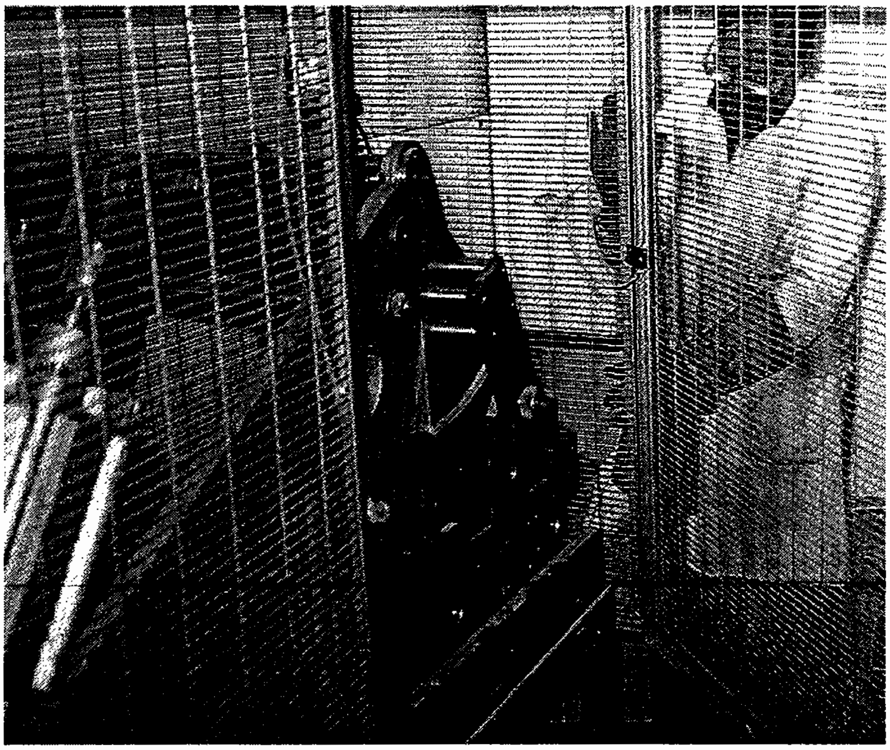


Figure 7.19 Mobil's Roller Compactor

RLA tests were carried out under the same conditions as those described in Section 7.5.2, ie at 40°C, on specimens with similar void contents. The results of these tests are presented in Table 7.6 and plots of the test data are shown in Figures 7.20 to 7.23.

Table 7.6 RLA Test Results for 100 Pen 28mm DBM Roadbase

Specimen Source	Specimen Reference	V_V (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain/cycle)	Minimum Strain Rate (microstrain/cycle)
Road Cores	MYA R1	2.3	3.31	59	2.2
	MYA R2	2.5	3.60	65	1.5
	MYA R3	2.0	3.12	59	1.7
	MYA R4	2.5	3.31	55	1.5
PRD	MYA P2	1.4	3.20	44	2.0
	MYA P3	2.3	1.92	28	1.4
	MYA P4	4.3	1.56	18	1.6
	MYA P5	4.1	1.47	25	0.7
	MYA P6	2.6	2.64	27	2.9
	MYA P7	4.1	3.08	35	2.6
Slab Cores	MYA SA2	2.9	4.97	76	2.6
	MYA SB2	2.8	2.69	40	1.6
	MYA SB3	2.5	3.29	52	1.9
	MYA SC3	2.9	2.13	33	1.1
	MYA SF2	2.7	2.20	34	1.2
	MYA SG2	2.5	2.91	43	1.6
Laboratory Roller Compactor	MYA XS2A	3.3	4.93	61	5.8
	MYA XS2B	3.2	4.76	43	7.8
	MYA XS2D	3.2	3.37	37	4.2

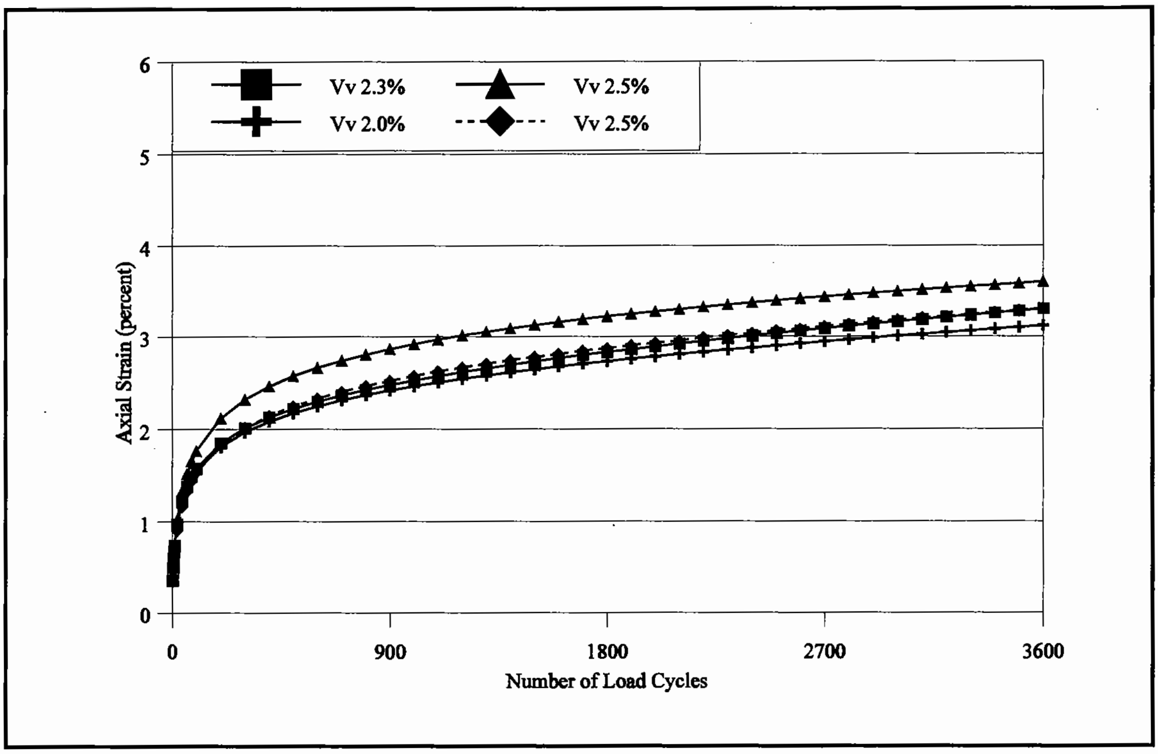


Figure 7.20 28mm DBM Roadbase : Road Cores

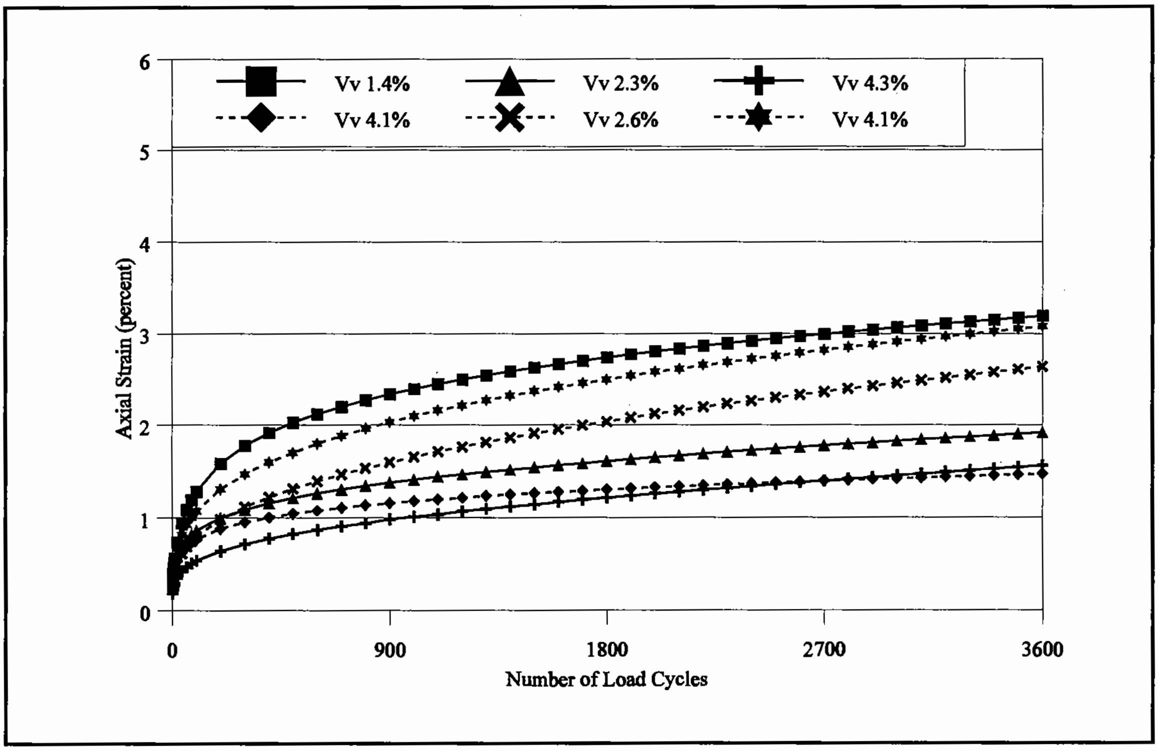


Figure 7.21 28mm DBM Roadbase : PRD Specimens

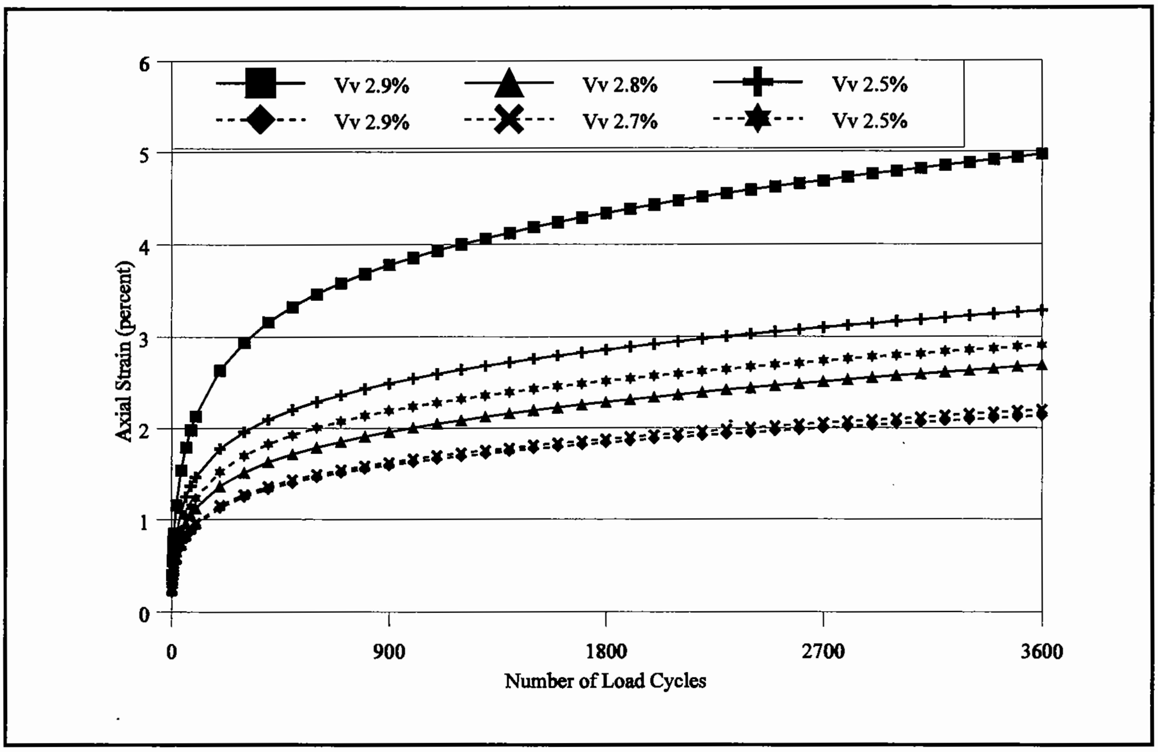


Figure 7.22 28mm DBM Roadbase : Slab Cores

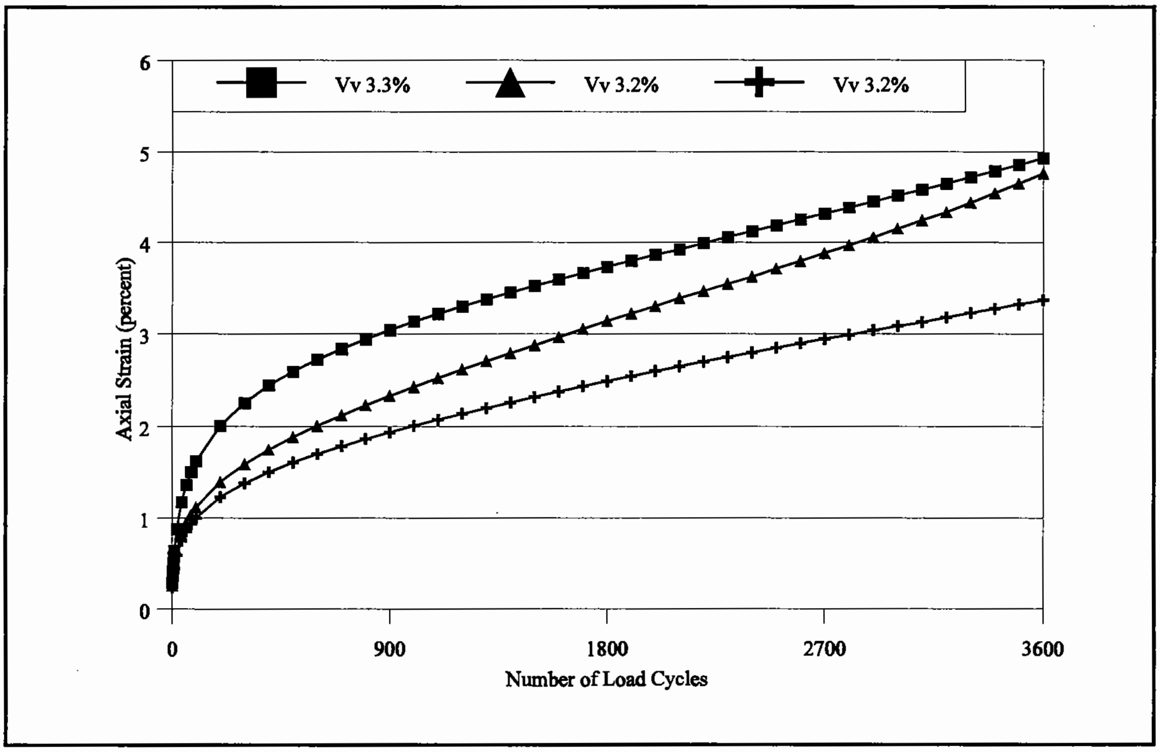


Figure 7.23 28mm DBM Roadbase : Roller Compactor Cores

From inspection of the plots, it appears that the road cores produced the most consistent results but the performance of the PRD specimens was generally better. The results for the slab cores showed some scatter but, with the exception of one result, were comparable with or better than those for the road cores. However, the most significant finding was that the specimens produced by the laboratory roller compactor showed much worse performance than any of the others. The values of mean strain rate in Table 7.6 would indicate that these specimens performed generally better than the road cores, despite this not being borne out by the plots of the test data which show continuing high rates of strain for the roller compacted cores towards the end of the test (Figure 7.23). This trend is, however, identified by the values of minimum strain rate reported in Table 7.6 which are significantly higher for the laboratory roller compacted cores than for the other specimens. This demonstrates the value of considering minimum strain rate in conjunction with mean strain rate and the need for care in using the latter, the weighting of which to the early part of the test has been discussed in previous Chapters, in isolation.

7.7.2 Evaluation of 28mm HDM Roadbase

Continued supply by Foster Yeoman to the motorway reconstruction contract allowed a second trial to be carried out at Coryton, this time on a 28mm Heavy Duty Macadam (HDM) roadbase (21), which is a continuously graded material similar to DBM but with a slightly higher filler content. Rolling of the slab in the steel formwork commenced with the temperature of the material at 140°C and a total of 24 passes of the roller were made ; 17 with vibration and 7 without. On this occasion, 9 PRD specimens were manufactured with compaction temperatures ranging from 140°C to 90°C and compaction times ranging from 120 to 30 seconds on each face. Two slabs were produced using the laboratory roller compactor, at an initial rolling temperature of 140°C, and six cores were again obtained from the sampled truckload after placement on site.

RLA testing was carried out on those specimens which were most similar in air void content, under the same conditions as used for the 100 Pen DBM roadbase. The results of these tests are presented in Table 7.7 and plots of the test data are shown in Figures 7.24 to 7.27.

Table 7.7 RLA Test Results for 28mm HDM Roadbase

Specimen Source	Specimen Reference	V_V (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain/cycle)	Min. Strain Rate (microstrain/cycle)
Road Cores	MYB R1	3.4	1.85	26	1.1
	MYB R2	4.3	1.76	24	1.1
	MYB R3	1.3	3.12	50	1.7
	MYB R4	1.2	3.54	57	1.6
	MYB R5	1.5	4.31	56	2.6
	MYB R6	1.8	4.24	53	2.3
PRD	MYB P3	1.4	4.03	38	3.0
	MYB P5	1.5	2.83	35	1.9
	MYB P6	1.2	2.51	34	1.3
	MYB P7	3.4	1.22	16	0.7
	MYB P8	2.4	2.61	34	1.1
	MYB P9	3.5	1.41	21	0.5
Slab Cores	MYB SA2	1.3	3.74	46	2.1
	MYB SB3	2.0	5.49	64	3.2
	MYB SC3	1.4	3.71	31	3.0
	MYB SD2	1.1	3.65	41	1.6
	MYB SG3	1.2	3.91	42	1.6
Laboratory Roller Compactor	MYB XS1C	4.4	3.07	37	3.5
	MYB XS2B	1.2	5.20	48	3.3
	MYB XS2C	1.3	3.54	40	2.0
	MYB XS2D	1.5	5.27	51	2.9

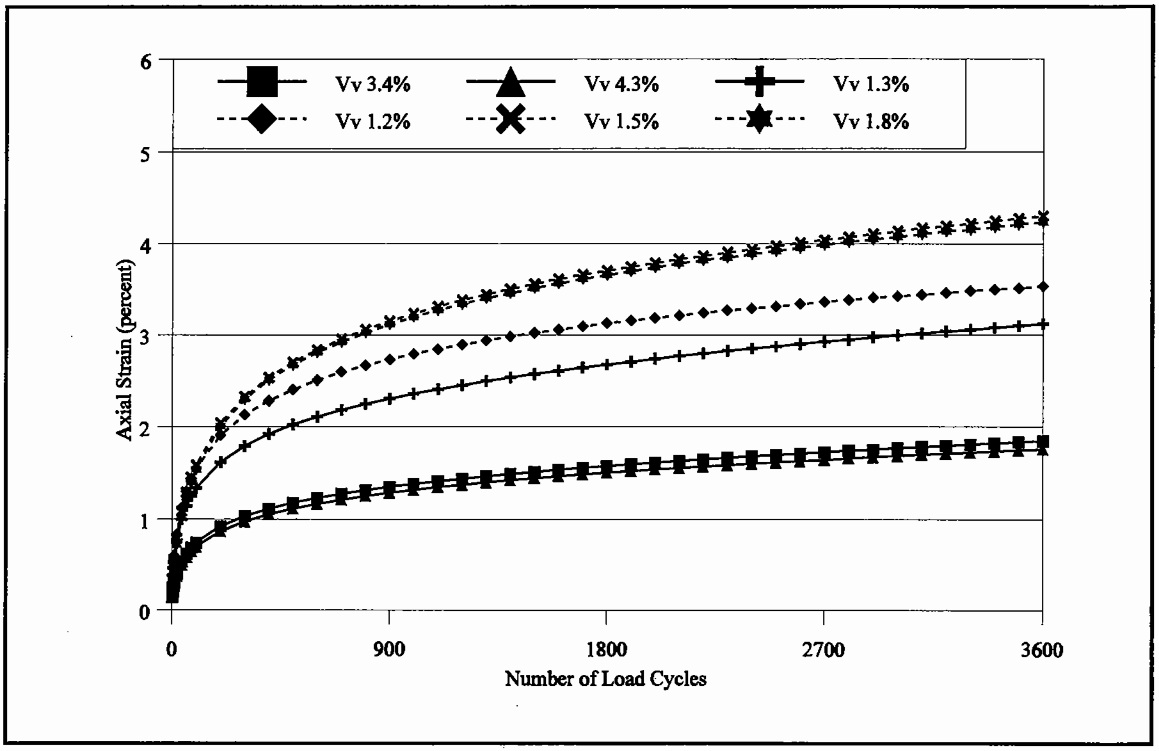


Figure 7.24 28mm HDM Roadbase : Road Cores

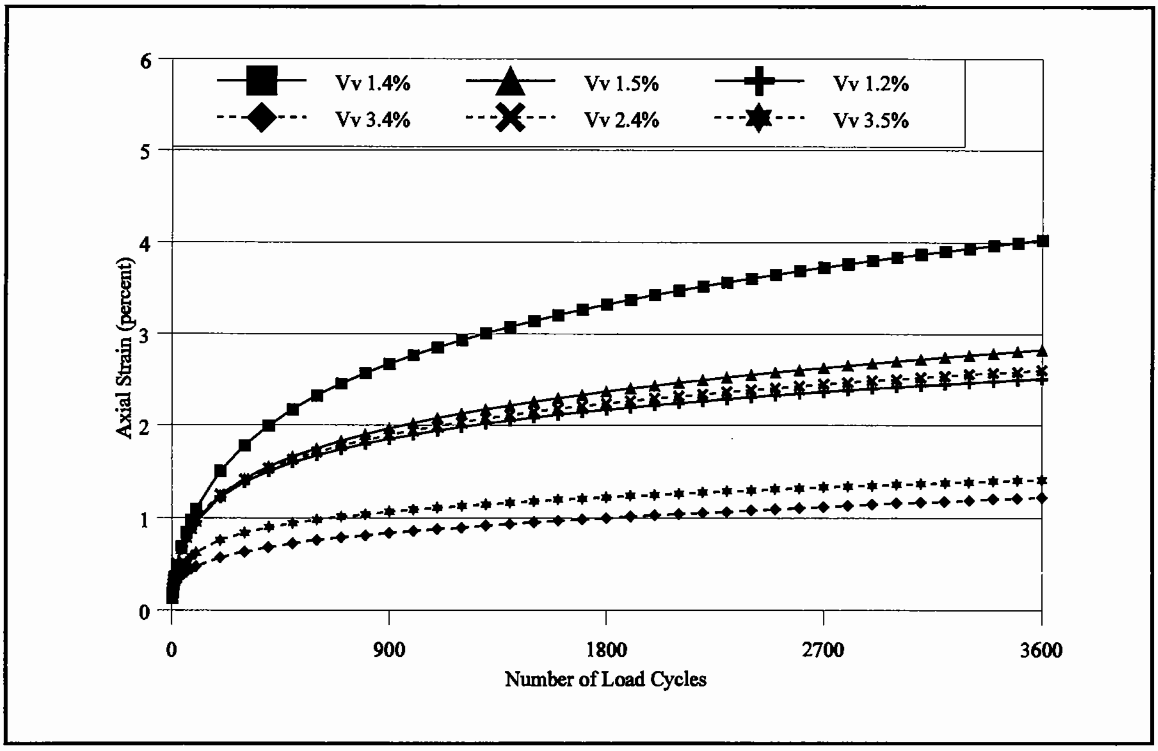


Figure 7.25 28mm HDM Roadbase : PRD Specimens

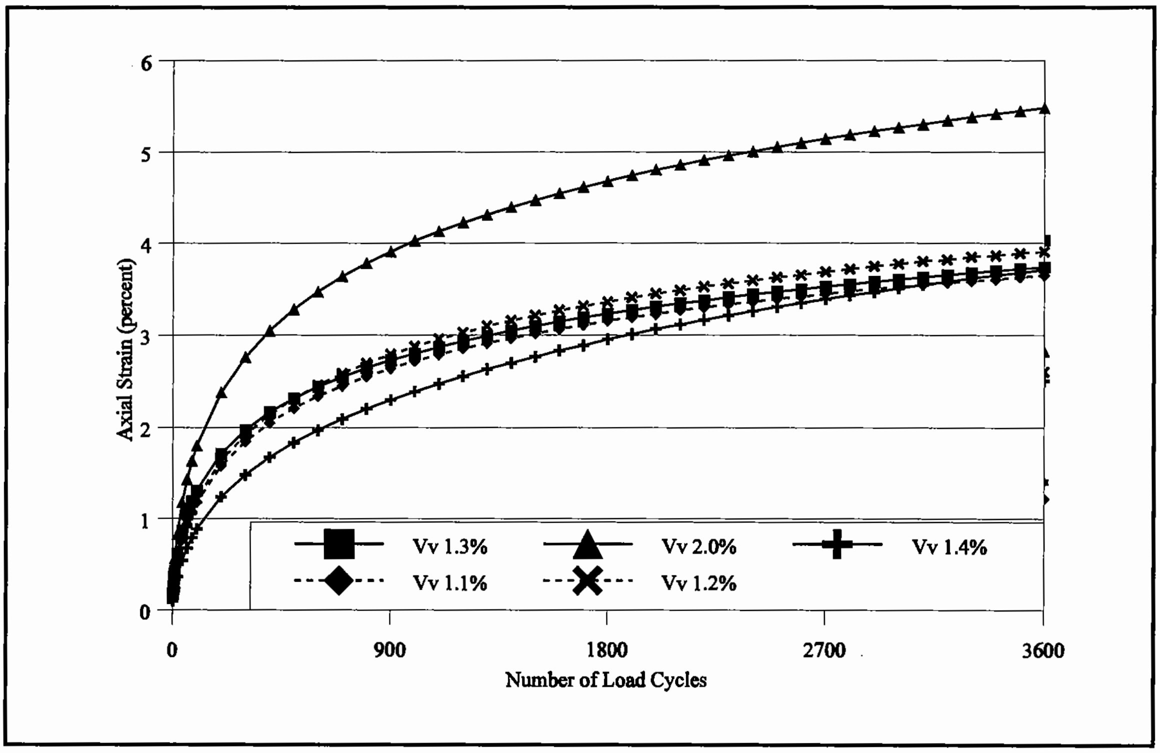


Figure 7.26 28mm HDM Roadbase : Slab Cores

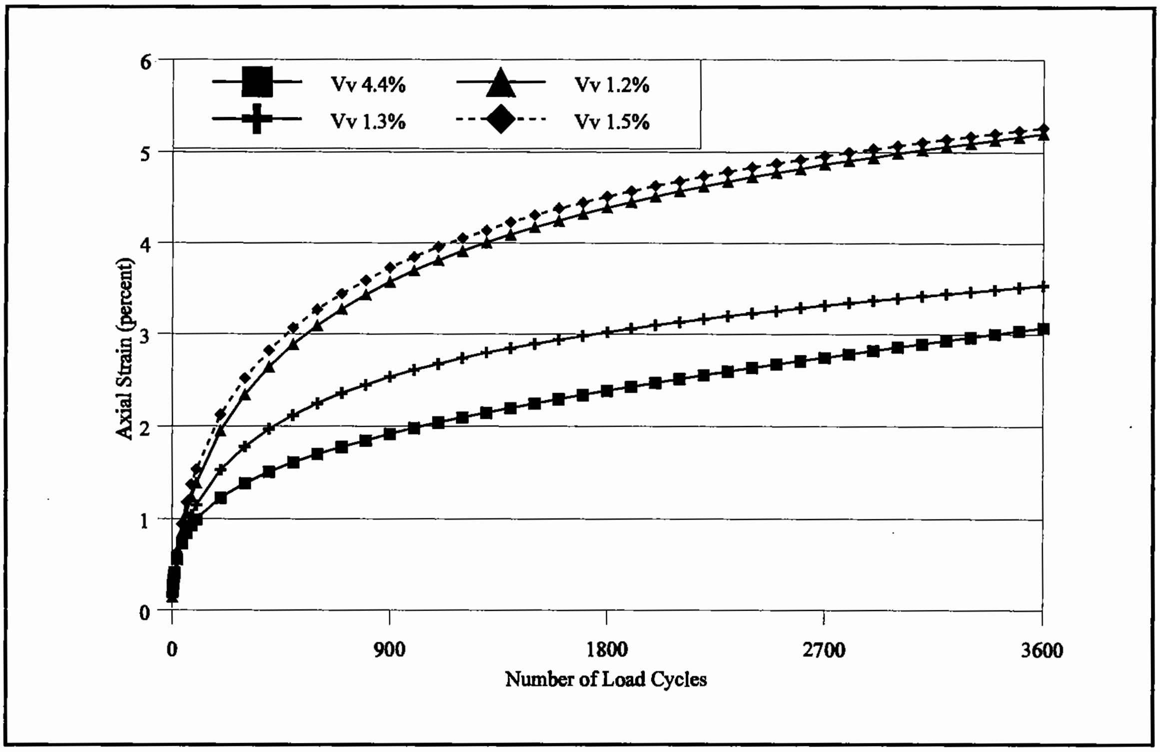


Figure 7.27 28mm HDM Roadbase : Roller Compactor Cores

It is difficult to discern, reliably, any difference in performance between the different compaction methods because of the degree of scatter in the data. However, it is clear that better performance was obtained in all cases from those specimens with an air void content in excess of 3%. This would, perhaps, indicate that the state of compaction, in the majority of specimens tested, is higher than the optimum for resistance to permanent deformation and this has contributed to the generally poor and varied performance.

7.7.3 Evaluation of 60/28 HRA Roadbase

The range of materials evaluated in the compaction study was governed by the supply to sites which were suitable for this exercise. In the event, although the range covered HDM and DBM, basecourse and roadbase and binders from 50 pen to 200 Pen, all the field trials described above were on continuously graded materials. The inclusion of a gap graded material in the study was considered important, since this would provide information on a different generic type of material. A third trial was, therefore, undertaken at Coryton on a 60/28 HRA roadbase which again was being produced at Purfleet for a motorway reconstruction contract. The compaction of the slab in the formwork and the manufacture of nine PRD specimens and two roller compactor slabs were all performed by Mobil personnel. Foster Yeoman drilled the six cores taken after the material had been placed and compacted in the carriageway.

The results of the RLA testing, which was carried out under the same conditions as for the previous two exercises at Coryton, are presented in Table 7.8. Direct comparison of the results from the various methods of compaction is complicated by the variation in void content of the specimens. While it is evident that material with high voids, ie in excess of 6%, generally performed poorly, it is difficult from the data to assess the significance of differences in air voids of the order of 1% to 2% .

Table 7.8 RLA Test Results for 60/28 HRA Roadbase

Specimen Source	Specimen Reference	V_V (%)	Ultimate Strain (%)	Mean Strain Rate (microstrain/cycle)	Min. Strain Rate (microstrain/cycle)
Road Cores	MYC RA1	2.1	1.77	25	0.9
	MYC RA2	2.0	2.81	36	2.0
	MYC RB1	2.4	3.75	44	3.0
	MYC RB2	3.1	2.87	39	1.9
	MYC RC1	3.3	4.25	46	4.5
	MYC RC2	2.2	2.44	33	1.2
PRD	MYC P1	3.8	6.23	47	10.8
	MYC P2	3.6	3.5	34	3.9
	MYC P3	3.4	3.73	43	2.8
	MYC P4	3.7	2.3	28	1.7
	MYC P6	5.5	6.49	51	9.7
	MYC P7	6.7	F	82	13.9
	MYC P8	6.9	4.89	25	7.4
	MYC P9	5.8	F	68	13.3
Slab Cores	MYC SA2	4.3	F	141	31.4
	MYC SA4	4.2	F	141	36.4
	MYC SB4	4.7	F	114	28.6
	MYC SC4	5.1	F	164	42.9
	MYC SD3	4.5	F	150	39.9
	MYC SD4	5.0	F	67	21.3
	MYC SE2	5.2	F	171	49.1
	MYC SE4	5.8	F	173	35.6
	MYC SF3	7.1	F	174	41.7
	MYC SG2	7.7	F	510	171.2
Laboratory Roller Compactor	MYC XS11	6.2	F	266	76.7
	MYC XS12	6.0	F	236	91.3
	MYC XS13	4.9	F	304	96.3
	MYC XS14	5.0	F	215	75.3
	MYC XS21	10.2	F	367	233.6
	MYC XS22	9.2	F	306	160.4
	MYC XS23	11.3	F	386	247.6
	MYC XS24	9.3	F	245	124.4

F denotes failure of the specimen under test

The road cores had the lowest void contents, lower even than the PRD specimens nominally compacted to refusal. Figure 7.28 shows the test plots for the road cores, while Figures 7.29 and 7.30 show, respectively, the plots for the specimens with lowest void contents from the PRD and slab compaction. The PRD specimens, though having void contents typically 1% to 1.5% higher gave, with one exception, similar performance to the road cores. The slab cores however, with void contents around 1% higher than the PRD's, showed much worse performance.

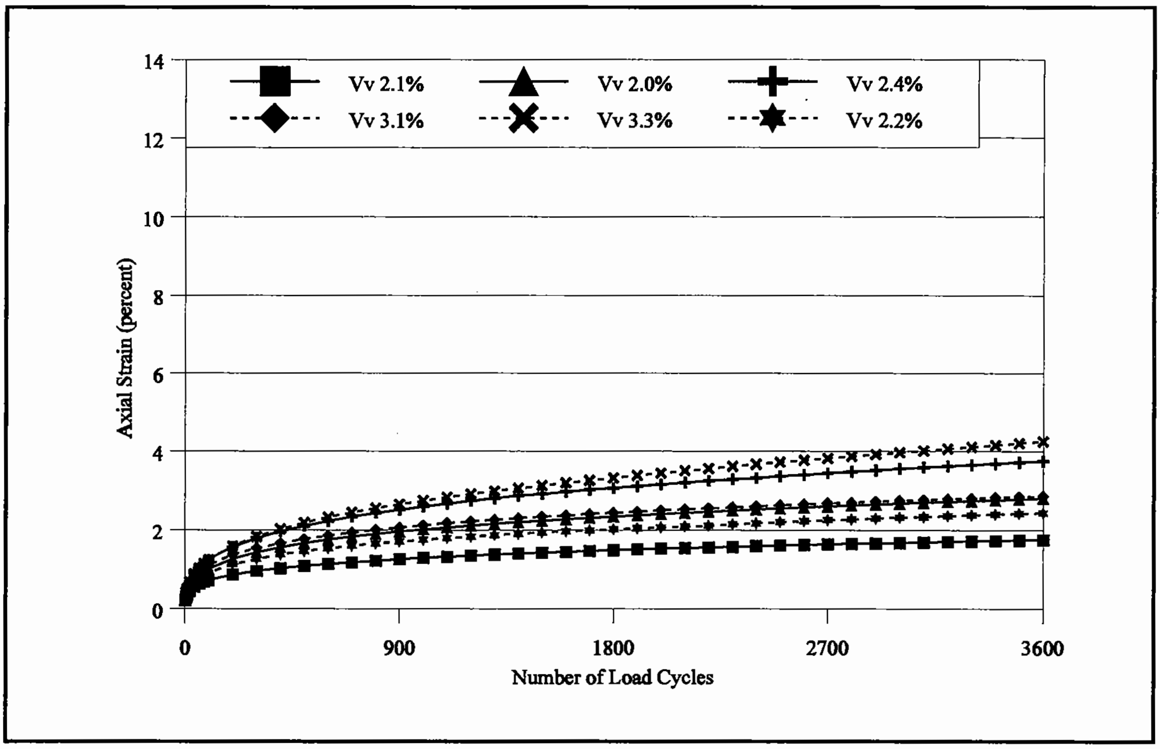


Figure 7.28 60/28 HRA Roadbase : Road Cores

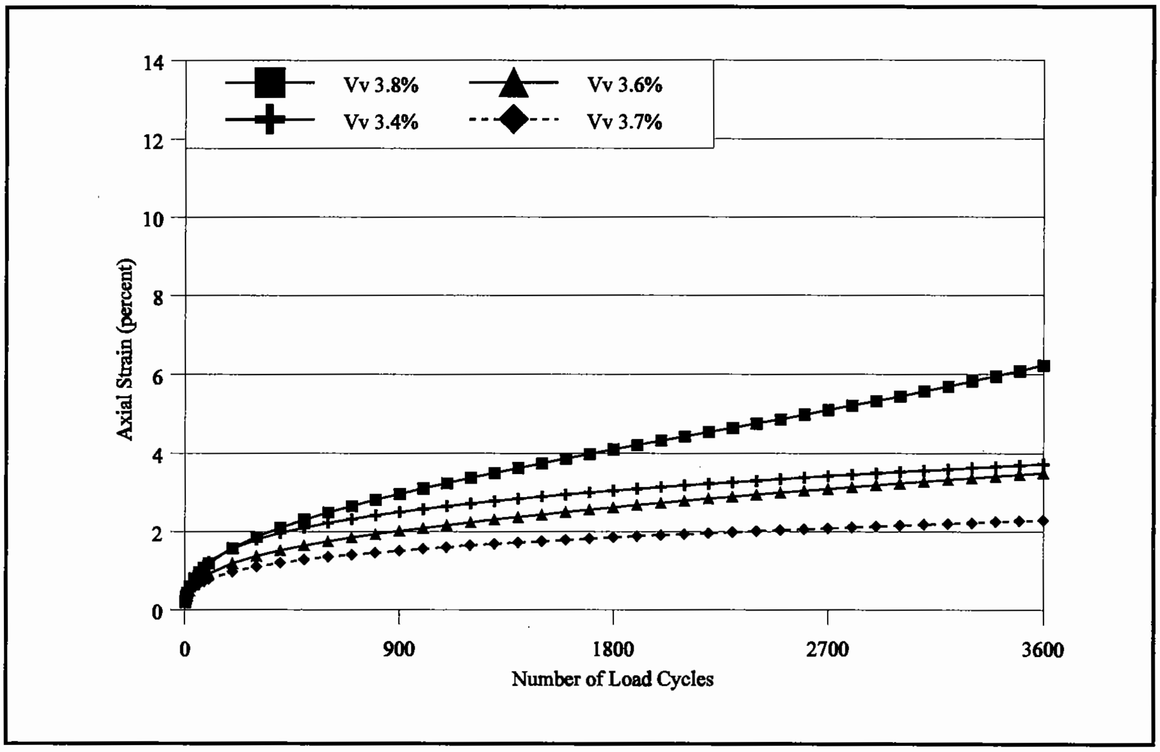


Figure 7.29 60/28 HRA Roadbase : PRD Specimens (Low Voids)

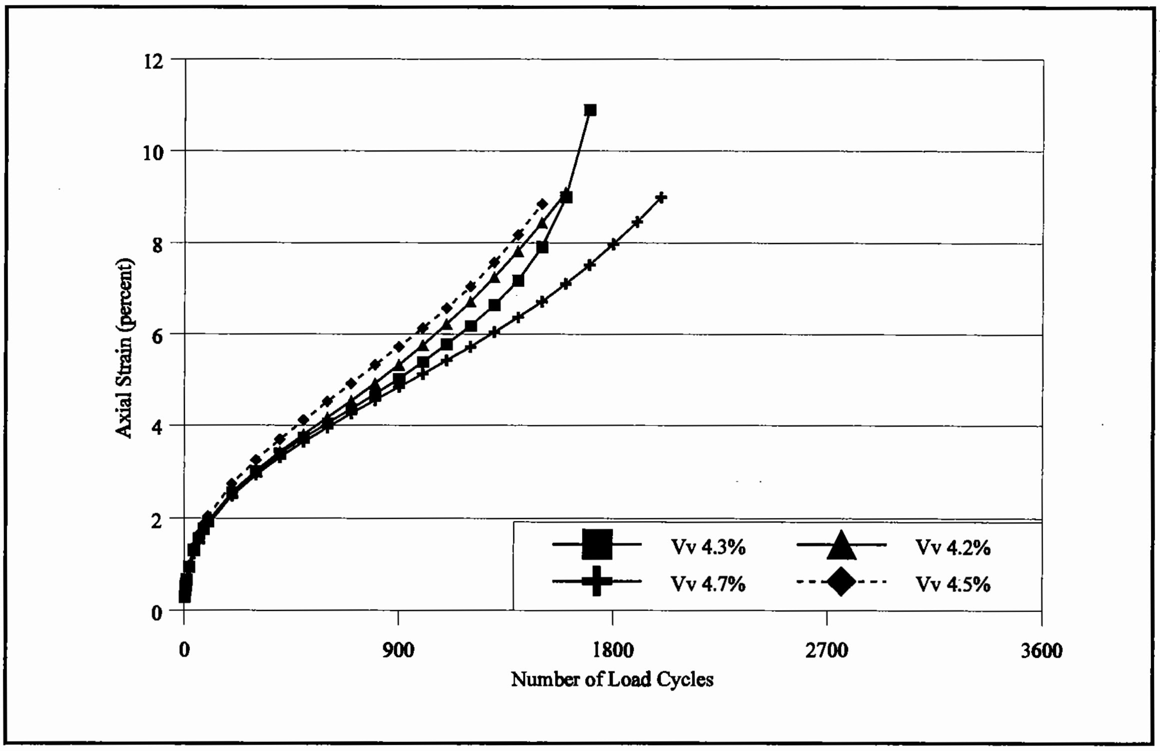


Figure 7.30 60/28 HRA Roadbase : Slab Cores (Low Voids)

Figures 7.31, 7.32 and 7.33 show, respectively, the test plots for the PRD specimens, slab cores and laboratory roller compactor cores with void contents in the range 5% to 6%. It is evident that the PRD specimens showed better performance than the slab cores which were, in turn, better than the roller compactor cores. It is perhaps significant that the laboratory roller compactor is the only compaction method which does not impart vibration.

It would appear that, notwithstanding the variation in void contents and the high values of void content in some specimens, the method of compaction does have an effect on performance in the RLA test for this gap graded material. A further point to note is that the values of minimum strain rate for the HRA are generally high, particularly when compared with the DBM and HDM specimens compacted by the same means, indicating the continued development of strain, which may be explained by the absence of a continuous coarse aggregate skeleton which would "lock-up" on the mobilisation of high strains.

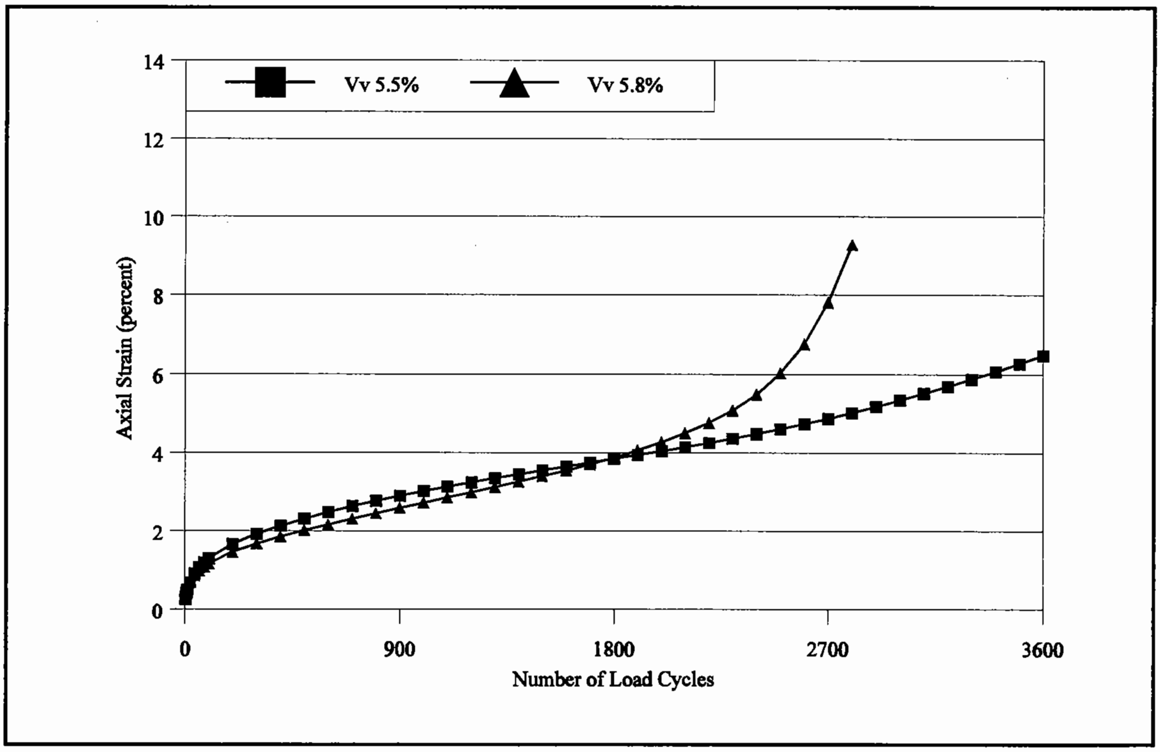


Figure 7.31 60/28 HRA Roadbase : PRD Specimens (Medium Voids)

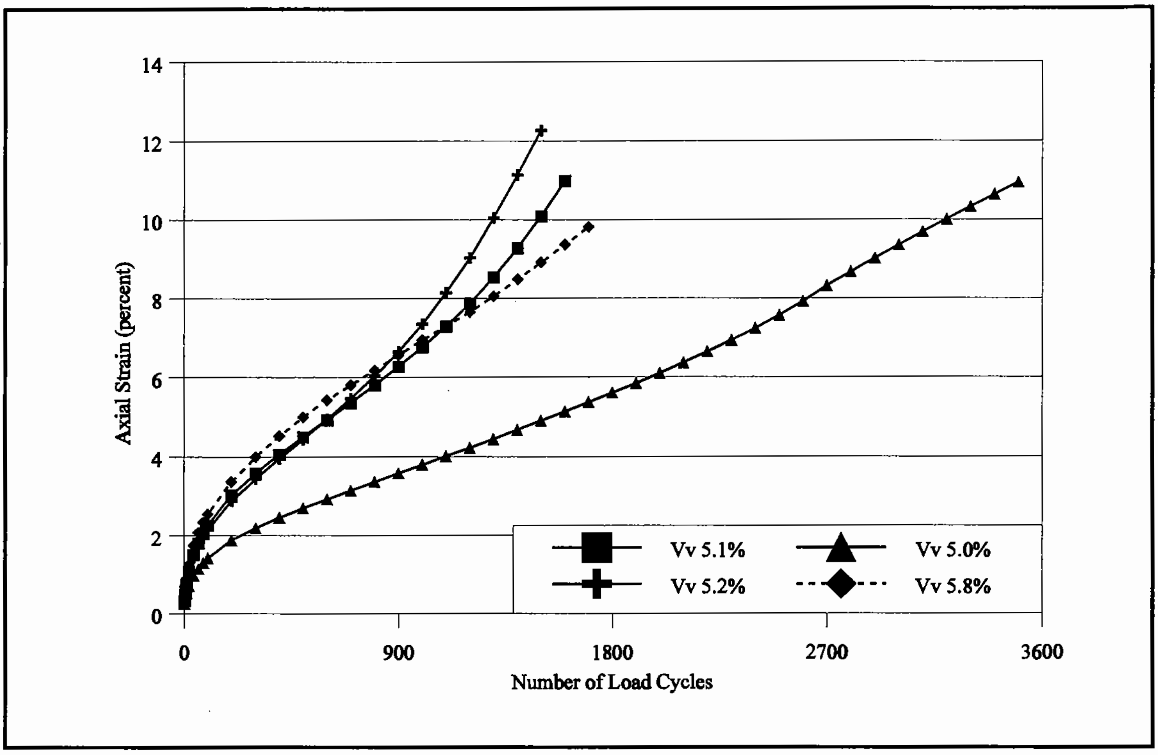


Figure 7.32 60/28 HRA Roadbase : Slab Cores (Medium Voids)

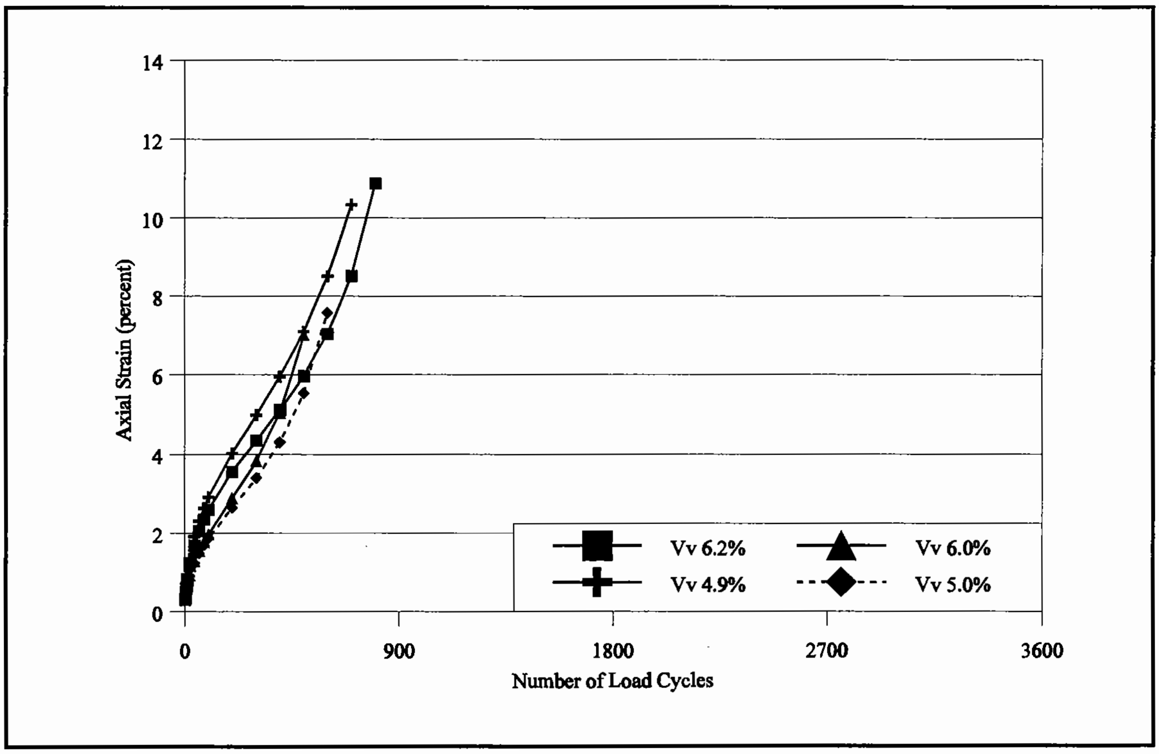


Figure 7.33 60/28 HRA Roadbase : Roller Compactor Cores (Medium Voids)

7.8 Contributions to the Study by the Industrial Sponsors

Several of the industrial organisations carried out compaction trials independently, in general accordance with the experiment guidelines which had been issued, as their contributions to the LINK Bitutest project, and in each case differences in performance in the RLA test for specimens compacted by different means were reported (143). Figure 7.34 shows the results of a trial undertaken jointly by Tarmac Roadstone Southern (TRS) and Kent County Council, in which a comparison was made between field cores, PRD specimens manufactured from mixture samples taken at the plant and cores drilled from a slab placed and compacted on a purpose-built test pad, also at the mixing plant. Compaction of the slab was by means of a deadweight site roller. RLA testing was carried out by both TRS and Kent CC and though there was a slight difference in the values of ultimate strain reported by the two, their results were consistent in rating the performance of the specimens compacted by the different methods. Poorest performance was exhibited by the cores from the slab laid on the test pad while the results from the PRD specimens were much closer to, though slightly better than, those from the field cores. The effect of compaction method on performance was greater than the effect of difference in void content on the response of the PRD specimens.

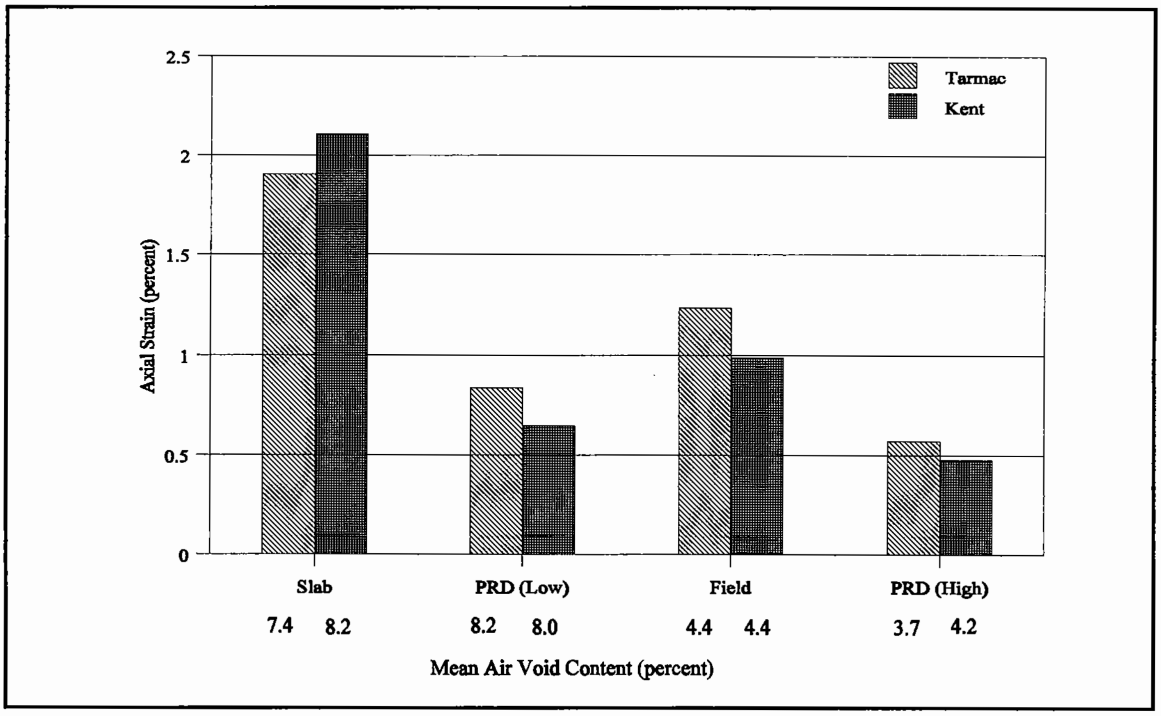


Figure 7.34 Effect of Compaction Method on RLA Performance : TRS & Kent CC Data

7.9 Discussion

The first observation to be made on the results of these trials is that the method of specimen compaction can have a considerable effect on resistance to permanent deformation as measured in the RLA test. Recognition of the potential effects of different compaction techniques on properties is, therefore, essential if a reliable evaluation of mixture performance is to be made. It is not, however, considered prudent to attempt a correlation between the different compaction methods based on the data obtained, since it is likely that the magnitude of any effect will vary according to mixture type and aggregate source.

The second observation to be made is that the PRD compacted specimens have generally produced results which, while not exactly the same, are closer to those from the road cores than the other methods. The performance of the slab compacted cores, which it had been assumed at the outset would be representative of site compacted material, has generally been rather poorer than that of the PRD specimens and not as close to that of the road cores as had been anticipated. On the basis of this study, it would appear, therefore, that use of the PRD equipment for routine specimen preparation is acceptable, at least for the purposes of mixture design. The apparently more representative methods, which are more time consuming, did not prove to offer a more reliable or superior approach. The ability of the PRD equipment to produce specimens which match the generally very high densities of site compacted material is another important consideration (see Chapter Two).

Control of the air void content in individual specimens is extremely difficult, if not impossible, with the PRD method. Also, difficulties were reported by some of the industrial sponsors in the use of this method for 40mm material where distribution of large aggregate was influenced by the size of the mould (150mm diameter), though no such problems were encountered with 28mm mixtures. However, it is a simple and quick procedure which can be used to manufacture a number of specimens over a wide range of density by varying compaction time and temperature. In this context, use of the guidelines for compaction times and temperatures developed in the earlier mixture design research (142) was not entirely successful and difficulty was experienced in achieving the lower levels of density. In practice the use of compaction temperatures lower than those recommended was found to be necessary.

Gyratory compaction was not included in this study since there were no such devices available for use while the research was being carried out. However, gyratory compactors are used in several of the mixture design methods reviewed in Chapter Two, though of the comparative studies described in Section 7.2 only that performed in the development of the AAMAS (44) concluded that this method produced specimens most representative of site compacted material. The particular advantages offered by gyratory compactors are the facilities both to control the density of the specimen and to assess mixture workability. Further investigations of the type reported here but including an evaluation of gyratory compaction would, therefore, be of great value.

Variation in the void contents of the specimens compacted by different methods frequently complicated direct comparison of the RLA test results. It proved difficult to achieve comparable air voids by each method of compaction since it was not known, in advance, what the density of the site compacted material would be and it was not feasible in practice to control the mass of material placed in the formwork for purposes of compaction to a specified volume. Furthermore, there was little merit in standardising compactive effort, in terms of roller passes, applied to each slab since different mixtures had different workabilities, while binder viscosities during compaction would have varied according to grade and rolling temperature.

There is, however, an indication from the data for the continuously graded material that within an "optimum range" of void content for a particular material, the actual value of air voids has little effect on performance. Figure 7.35 shows a plot of mean strain rate against void content for the PRD specimens of 100 Pen 28mm DBM roadbase manufactured at Coryton. It is apparent that mean strain rate is insensitive to voids below about 8% to 10% but, above this level, there is a marked reduction in performance. A similar plot is shown in Figure 7.36 for the 28mm HDM Roadbase PRD specimens made at Coryton. These data show higher strain rates in those specimens with void contents below 3% while the results for voids in the range 3% to 8% are consistent and lower. The results for the 200 Pen 20mm DBM basecourse supplied by Wimpey and the 28mm DBM 50 obtained by Staffordshire CC are consistent with this observation.

It would appear that performance at very low densities is influenced by air void content, simply reflecting poor compaction. Air void content is also significant at very high densities because there is insufficient void space to accommodate the binder/filler mortar (see Chapter Two). Within a notional optimum void content range, the effect of void content on resistance to permanent deformation is not so significant and this range should be targetted for mixture design purposes.

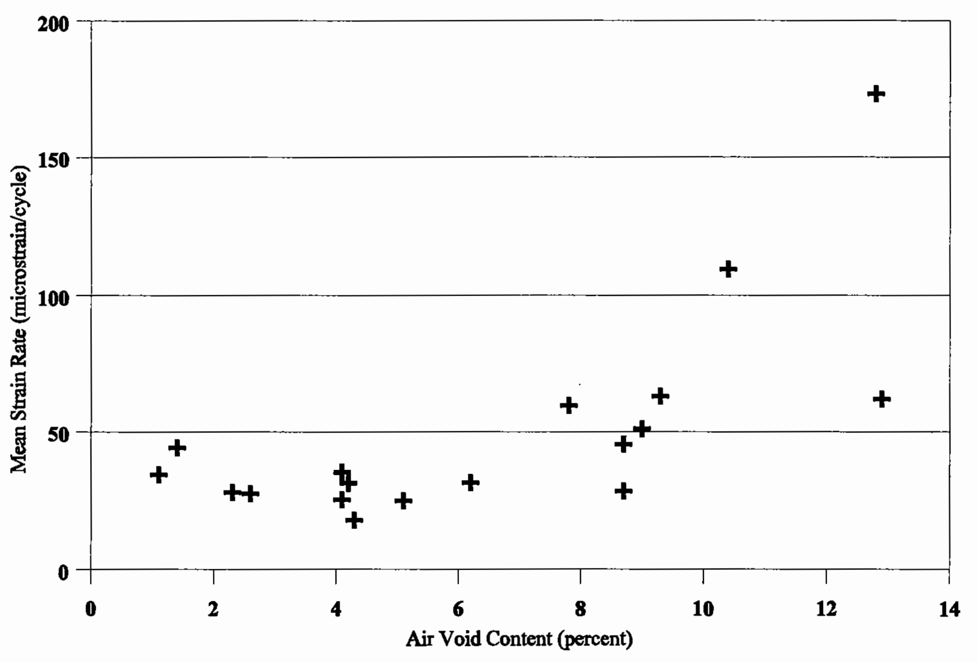


Figure 7.35 28mm DBM Roadbase : Effect of Void Content on Mean Strain Rate

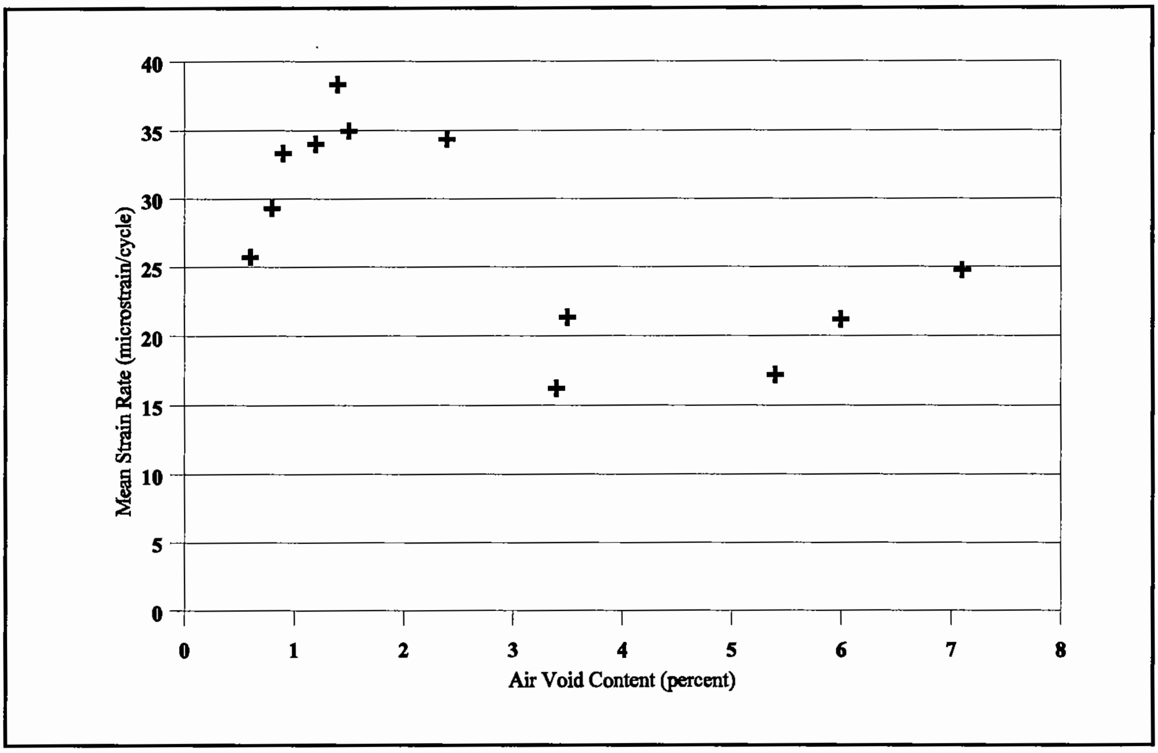


Figure 7.36 28mm HDM Roadbase : Effect of Void Content on Mean strain Rate

CHAPTER EIGHT
FINAL DISCUSSION AND CONCLUSIONS

8.1 Introduction

Simple uniaxial tests are used for the assessment of resistance to permanent deformation in half of the methods for the design of bituminous paving mixtures which were reviewed and presented in Chapter Two. The attractions of these tests include straightforward procedures and the facility to test cores or specimens moulded by any of several means. Equally important are the simplicity and relatively low cost of the equipment required, even for repeated load tests since the introduction of pneumatic systems (32,112). However, while a repeated load axial test showed good correspondence with a wheeltracking test (Chapter Three), over which it would be preferred for mixture design for reasons of simplicity, it has been found that performance in the test is influenced by both the test configuration and the method of specimen compaction. These findings are of concern if such a test is to be used to make a quantitative assessment of mixture performance, particularly since it appears that the degree of influence of test configuration may vary with both mixture and aggregate type. Furthermore the results presented in Chapter Five, which showed that a gap graded 50 Pen HRA outperformed a continuously graded 50 Pen DBM at test temperatures of 20°C and 30°C but that at 40°C the rating was reversed, indicate that test configuration is also an important consideration in mixture design where the aim is simply to rank and compare the performance of different mixture formulations.

8.2 Effect of Test Configuration

The occurrence of plastic strain, due to the relative displacement of aggregate particles, is an important component of the total permanent strain which develops in mixtures with a contiguous aggregate skeleton (116,144). It has been demonstrated, both in this research and by others (86,114), that this effect is emphasised by repeated loading, though it would appear that the application of a confining stress is beneficial in constraining such displacement, as is increased binder stiffness which presumably binds the aggregate more rigidly in the matrix. Conversely, static loading would be more onerous for mixtures which rely on the binder or binder/filler mortar due to the very low binder stiffness which ensues from the long load duration. The effect of low binder stiffness was clearly illustrated by the failure of the ceramic aggregate specimens subjected to static loading in the triaxial test programme (Chapter Five). The relative effects of test configuration on the two principal mechanisms for resistance to permanent deformation may be summarised as shown in Table 8.1.

Table 8.1 Effects of Axial Test Configuration

<table>
<tr><th rowspan="3">Test Configuration</th><th colspan="3">Effect of Test Configuration</th></tr>
<tr><th colspan="2">Mechanism of Resistance To Permanent Deformation</th><th rowspan="2">Remarks</th></tr>
<tr><th>Contiguous Aggregate Skeleton</th><th>Binder or Binder/Filler Mortar</th></tr>
<tr><td>Static Loading</td><td>Beneficial</td><td>Adverse</td><td>Low binder stiffness. Deformation principally due to viscous flow in the binder.
Little or no plastic strain induced in the aggregate skeleton.</td></tr>
<tr><td>Repeated Loading (short duration)</td><td>Adverse</td><td>Beneficial</td><td>Relatively high binder stiffness. Promotes plastic strains in aggregate skeleton.</td></tr>
<tr><td>High Temperature</td><td>Adverse</td><td>Adverse</td><td>Low binder stiffness. Effect likely to be greatest on binder/filler mechanism at very high temperatures.</td></tr>
<tr><td>Confinement</td><td>Beneficial</td><td>Neutral</td><td>Constrains relative displacement of aggregate particles.</td></tr>
</table>

The results presented in Chapters Four and Five indicate that the effects of confinement and variation in binder stiffness on materials which rely principally on aggregate friction and interlock are most pronounced in mixtures with poor aggregate shape and texture.

On this basis it would appear that, since a particular test configuration may favour one mechanism of resistance more than another it may be difficult to establish a single set of test conditions which would permit a true and fair comparison of the relative performance of all mixtures. However, the case for the use of both repeated loading and confinement is compelling.

Repeated loading is to be preferred to static loading as, besides being intuitively more representative of vehicle loading, its importance in the development of strains in an aggregate structure has already been discussed. The application of a confining stress is also considered desirable since it is more representative of in situ loading than the unconfined condition and should produce no adverse effect on any mixture type.

The other parameter which has a major effect on mixture performance, at least in unmodified materials, is the stiffness of the binder. In mechanical testing to assess resistance to permanent deformation binder stiffness is, in part, controlled by the temperature at which the test is carried out. The currently proposed value in the draft British Standard (134) for uniaxial testing of basecourse and roadbase is 30°C, which it is understood was recommended by TRL on the basis of data which shows that the temperature of roadbase and basecourse in pavements in the UK is unlikely to exceed this value, which is, therefore, considered "representative".

However, binder stiffness is also controlled by loading time. In the RLA test the load pulse duration is one second and, using equation 4.2 (10), reproduced below, this can be shown to be equivalent to the loading time at a depth of 100mm in the pavement for a vehicle travelling at 0.7 km/hr, which would not generally be considered a "representative" traffic speed.

$$\log t = 0.5 \times 10 - 4 \times h - 0.2 - 0.94 \log v \qquad (4.2)$$

where t is loading time (seconds)

h is depth (mm)

v is vehicle speed (km/hr)

As an example, the stiffness of the 50 Pen binder used in the NAT test programme described in Chapter Five has been estimated, using the Shell program BANDS-PC, as 56 kPa under the conditions of the RLA test at 40°C. If the test temperature were reduced to 30°C the stiffness of the binder would increase to 385 kPa. However, at 40°C and with the loading time, at a depth of 100mm, equivalent to a vehicle travelling at 50 km/hr the stiffness of the binder is estimated at 1460 kPa, while under a vehicle travelling at 80 km/hr the binder stiffness would be 2040 kPa. This illustrates the relative effects on binder stiffness of temperature and loading time, and gives a perspective on the selection of test temperature based upon what is deemed a "realistic" condition. It also shows that vehicle speed is an important factor in the performance of bituminous mixtures with respect to permanent deformation, as has also been demonstrated by other researchers (145), but nonetheless appears to indicate that the use of static loading would perhaps only be justified to emulate conditions where stationary vehicles are expected since the loading time in the RLA test already corresponds to a very low traffic speed.

8.3 Importance of Binder Properties

It has been suggested on the basis of previous research into the deformation resistance of DBM's (20) that the grade, and hence the stiffness, of the bitumen is of little or no consequence for such materials and that aggregate-related parameters are more important. This may be due to the fact that, largely for reasons of accelerating the test procedure, deformation tests are generally carried out under conditions of very low binder stiffness arising from the use of high temperatures, long loading times or a combination of the two. Under such conditions the effect of the binder might reasonably be expected to be diminished while that of the aggregate is emphasised. There is, however, clear evidence from this research which illustrates the significance of the binder on performance, and binder stiffness has been found to correlate with rut depth in medium scale wheeltracking tests on asphaltic concretes (12).

The testing of the trial pavement reported in Chapter Four showed the binder to be the dominant parameter despite the use of two very different aggregates in a material with a continuous grading. It is not possible, however, to be certain that the superior performance of the SBR modified mixtures in this exercise was entirely attributable to greater stiffness in the binder as a result of the reduced temperature susceptibility which it is known that this additive causes (119,120,121). The effect of reducing the temperature susceptibility is to increase the viscosity of the bitumen, at a given temperature and rate of loading, which should also reduce the viscous strain in the binder, and it has been shown by TRL that the wheeltracking performance of a 30/14 HRA wearing course (gap graded with 30% stone content), which would rely on the binder/filler/mortar mechanism, correlated with viscosity for a range of conventional and modified binders (146). Nahas et al (147) have found in work on conventional and modified binders that Softening Point, which is a pseudo-viscosity measure, is a good indicator of deformation resistance but gives no indication as to the mechanism by which a modifier may enhance performance. However, Decker and Goodrich (148), while recognising that increased binder viscosity will result in increased mixture stiffness which will be beneficial for deformation resistance, have suggested that elastic behaviour in the binder may be more important than viscosity in producing mixtures which are resistant to permanent deformation. The effect of many modifiers is to enhance the elastic properties of the binder, and Valkering et al (111) have attributed the superior deformation resistance which they observed in SBS (styrene-butadiene-styrene) modified mixtures compared to those with conventional and multigrade binders to the greater recovery properties of that material. In 1993 Reese and Goodrich (149) reported the findings of analysis of materials from a test road constructed in California in which varying levels of rutting had developed in sections where the controlling variable was determined to be the binder. Poorest performance in the field was from a section containing one of the highest viscosity binders while binders with lower viscosities had not rutted. Rheological analysis of the binders, which included modified materials, indicated that the most deformation resistant had greater elastic properties.

Though there is obviously an effect of binder stiffness on the deformation resistance of a mixture it is not clear by what other mechanism, or mechanisms, binders, modified binders in particular, contribute to or enhance this property. Indeed, it is likely that the mechanism varies with the binder/modifier type or combination. It would appear, therefore, that further investigation is merited in this area, and suggestions as to how this may be pursued are presented in Chapter Nine.

8.4 Application of the Repeated Load Axial Test

The RLA test provides a simple and convenient means of assessing resistance to permanent deformation. Comparisons with wheeltracking tests have shown that it is able to discriminate effectively which give it an obvious application in mixture design where the aim is to rank and compare the performance of different mixture formulations. However, an understanding of the deformation mechanisms which the test emphasises and how these are dependent upon the test conditions is essential if a proper assessment of a material is to be made. This would be particularly important when comparing mixtures which operate on different mechanisms.

The potential for test configuration to create a bias against a particular mechanism or exaggerate differences in performance between mixtures, say with different aggregate types, to a degree which would not be apparent in service poses a difficulty in making a quantitative assessment of either the relative or absolute difference in performance between mixtures. For the latter, the situation is complicated further by the observed effect of method of specimen compaction. Moreover, it may be that differences in service loading conditions may make the choice of the most appropriate material vary from one application to another.

It may, therefore, be better for the purposes of specification of material performance to use the test as a tool to determine mixtures most likely to fail under extreme conditions and to set the specification in terms of the acceptable risk for the intended application, eg class of road. The principle of this approach is illustrated and discussed below with particular reference to the selection of a standard test temperature.

The results of the tests on the 200 Pen mixtures in the NAT test programme reported in Chapter Five demonstrated that the reduction in binder stiffness caused by increasing the test temperature from 30°C to 40°C exaggerated the difference in performance between the two materials. Bearing this in mind, it is possible to envisage a situation where two mixtures perform similarly under the majority of operating conditions but show very different levels of resistance to permanent deformation under extreme conditions, such as may arise with a combination of high temperatures and slow, heavy traffic in contraflow. It is under such conditions that the potential for, possibly rapid, failure exists.

Consideration of the philosophy behind the use of the LCPC wheeltracking test (101) is worthwhile in this context. This test is carried out at a temperature of either 50°C for basecourse or 60°C for wearing course (97), both of which are extreme, though the climate of southern France does mean that high pavement temperatures do have to be considered. However, this test is not intended to characterise the anticipated performance of the material in situ but is used as a "specification test" to assess whether the material is capable of providing an acceptable level of resistance to permanent deformation under the conditions of the test. Moreover, the test is principally used when a mixture is being proposed for use on a site which has a potentially high risk of rutting. This test is biased towards mixtures which derive their mechanical integrity from the aggregate structure, since at 60°C the stiffness of the binder will be low. Mixtures which rely largely on the binder or binder/fines/filler mortar are likely to perform disproportionately badly with regard to their observed performance on the road. A 30/14 HRA tested by Esso for the Bitutest project failed to produce a reportable result. However, where an aggregate structure which is capable of withstanding the conditions exists the test can, reputedly, discriminate between the performance of different binders.

Given that the RLA test conditions are not realistic, in terms of bitumen stiffness, at either 30°C or 40°C, (Section 8.2) it is recommended that 40°C be used routinely as this will accelerate the test, make it more discriminating and give a greater degree of certainty of performance under extreme conditions. It is recognised that this may discriminate against materials which would perform adequately under the majority of conditions, so the option should remain to test at a lower temperature if the application for the material is likely to give a low risk of rutting.

8.5 Implementation of the RLA Test

8.5.1 Standardisation of Test Conditions

Ideally the test configuration should be tailored for the anticipated environment and loading conditions of a particular application in order that the most appropriate material could be selected. Obviously this is not a practical proposition for routine testing and the absence of standardised conditions for the test would be likely to hinder its acceptance for general use. However, if the test is used in the manner described in Section 8.4, and with an understanding of its effects then the standardisation of test conditions is feasible.

On the basis of the results of the parametric study and following the discussion presented in the preceding Sections, the test conditions which it is recommended be adopted for the RLA test performed in the NAT are as detailed below:

Test temperature	:	40°C
Compressive stress	:	100 kPa
Load pulse duration	:	1 second
Load pulse frequency:		0.5 H_Z
Confining stress	:	70 kPa (static)
Test duration	:	3600 cycles
Pre-test conditioning	:	2 minutes @ 5 kPa (static)

The magnitude of the confining stress has been selected because this value has been found to produce a significant effect in continuously graded mixtures, and also because it is should be easily achievable through the use of a vacuum pump of the type currently used in many laboratories for binder recovery.

The test duration of 3600 cycles was chosen as a balance between running the test for a sufficient length of time to be able reliably to characterise the material and achieving an acceptable rate of testing. Although each test will require a little over two hours of machine time the input from the operator is much less. It may be that with increasing experience of use of the test and the parameter minimum strain rate, which should identify trends in the latter stages of the test, it may be possible to reduce the duration.

It is recommended that the results reported from the test should be both mean and minimum strain rate, since the use of these parameters together allows a more complete characterisation than could be obtained independently from either. They also have the advantage over ultimate strain that a numeric result is obtained even if failure of the specimen occurs. However, plotting of the test data is always advisable, not least for reasons of identifying any obvious problems in the execution of the test.

8.5.2 Considerations for Specification

For the test to be implemented in a performance-based specification, criteria for acceptance or classification of mixture performance will need to be established in terms of absolute values of the reported results. Given the potential effect of method of specimen compaction, demonstrated in the investigation reported in Chapter Seven, it would appear that specimen preparation will also have to be standardised where mixture evaluation is to be carried out on material other than that placed and compacted in the road during construction. Such standardisation would need to take account of the possibility that different versions of the same generic type of laboratory compaction device may also produce differences in performance (69).

The use of the PRD equipment as a means of specimen preparation, as proposed in the Nottingham mixture design method (33), has been shown to be feasible in terms of producing specimens representative of site compacted material. A further merit of this method, for which reason it has also been incorporated in the revision to the DOT specification (24), is that it is capable of achieving very high levels of density. This is essential for checking basic volumetric composition, the importance of which was discussed in the review of mixture design methods presented in Chapter Two.

The possible effects of compaction method and also the differences in test conditions employed for the various investigations which were carried out mean that the RLA test database which has been established during this research does not form a sound basis for setting criteria for performance in this test. To attempt to do so would, therefore, be premature.

A further limitation is the scope of the data which presently exists. Although during the course of the research RLA tests have been carried out on asphaltic concretes and typical UK mixtures for wearing courses, basecourses and roadbases, including HRA's, DBM's, and HDM's with various aggregates and binders, these materials are intended primarily for use on heavily trafficked roads. This is certainly the case for the UK mixtures which are produced in accordance particular national specifications designed to ensure adequate performance on the trunk road network. Such materials do not necessarily represent the best solutions, either structurally or economically, for use in less heavily trafficked situations where the whole pavement structure may be very different from that found in a trunk road. If proper use is to be made of performance-based specifications then materials must be designed for a particular application which means contemplating designing for low as well as high levels of performance where appropriate. In this context, the data does not exist over a sufficiently wide range of performance levels to establish sensible criteria.

The availability of data for a wider range of materials may also allay or put into perspective concerns over the ability of the test to discriminate which have largely been raised by trials carried out and reported by the TRL (150). These trials, which formed part of a research project sponsored jointly by the Highways Agency, the Refined Bitumen Association (RBA) and British Aggregates Construction Materials Industries (BACMI), involved use of the RLA test to assess DBM basecourse and roadbase mixtures used on two motorway strengthening projects. TRL reported that no link could be established between composition and performance. This is, however, not surprising since in each case the mixtures conformed to the same specification grading (21) and the absolute variations in results were small, as might have been perceived if the comparison had been made between more fundamentally different materials. Assessment of the reproducibility and repeatability of the RLA test was specifically excluded from this research since certain of the sponsors of the LINK Bitutest project were members of the RBA and BACMI and they were keen to avoid duplication of effort with the TRL project. It would appear, however, that such an exercise would be worthwhile if it were to include materials which would give a wide range in expected performance.

8.6 Conclusions

The following conclusions may be drawn from the research presented in this thesis :

1. The repeated load axial (RLA) test is able to discriminate between the performance of mixtures with respect to their resistance to permanent deformation.
2. Care must be taken in the assessment of mixture performance due to the influence of test configuration. Binder stiffness and the application of confinement are of particular significance.
3. Further work is required to establish the mechanisms by which binder modifiers enhance resistance to permanent deformation, and how these mechanisms are examined by the RLA test.
4. Method of specimen compaction may affect the reported result and should, therefore, be standardised.
5. The Percentage Refusal Density (PRD) equipment has been shown to be at least as effective as a number of other means of compaction in producing RLA test specimens representative of site compacted material. However, evaluation of gyratory compaction is recommended.
6. Recommendations have been made for a test configuration suitable for routine use.
7. Two strain rate parameters, mean and minimum strain rate, have been proposed for the characterisation of performance in the test.
8. Establishment of performance criteria would be premature. Further data are required once test configuration and method of specimen preparation have been standardised.

CHAPTER NINE
RECOMMENDATIONS FOR FURTHER WORK

The importance of binder properties for resistance to permanent deformation was clearly demonstrated by the testing of the mixtures in the trial pavement (Chapter Four). Unfortunately it was not possible to determine from this data by what mechanism the superior performance of the SBR modified mixtures was achieved. It may have been due to increased viscosity which would inhibit viscous strain in the binder, increased stiffness which would inhibit plastic strain in the aggregate structure, greater elastic recovery properties, a combination of these or some other mechanism. There is conflicting evidence in the literature as to the effect of viscosity, and recent rheological analysis has tended to favour the significance of enhanced elastic properties. It is not clear, however, whether the effect improved elastic behaviour is restricted to strains which occur in the binder or whether greater resilience in the aggregate structure also results. Goodrich (151) has concluded from rheology on mixture samples that the effects of binder properties apparent in rheological analysis of the binders alone are greatly diminished in the mixture. However, the value the mixture rheology as performed by Goodrich must be questionable for permanent deformation, which is a large strain phenomenon, due to the size of the specimens (50×12.5×2.5mm) and also the magnitude of the strain applied in testing (0.01%).

It would seem that a more satisfactory approach would be offered by the use of image analysis techniques to determine the permanent and recoverable strains in an aggregate structure with a range of binders. The use of an artificial aggregate such as that used in the triaxial test programme would be suitable for such an investigation as it would provide a relatively uniform aggregate structure which should enhance repeatability, has been shown to be sensitive to binder properties and would facilitate monitoring the movement of individual particles.

In order to obtain more quantitative data it is suggested that a programme of mixture testing, along the lines of that reported in Chapter Five for the continuous and gap graded mixtures at a range of temperatures, be repeated with a wider range of binders and other aggregate gradings. The testing should be augmented by rheological analysis of the binders under loading conditions as close as possible to those of the mixture tests. This would be helpful in determining the approximate value of stiffness at which aggregate response starts to dominate and whether this is consistent for all types of binder since it is possible that, as discussed above, the use of modifiers may alter the mechanism by which the mixture resists permanent strain. However, careful consideration will have to be given to the loading conditions as preliminary investigations have shown that it may not be possible to obtain stiffness measurements from rheometry when using a uni-directional square pulsed load, to emulate the RLA test, but the use of sinusoidal loading in mixture tests has been shown to produce a different response than pulsed loading (115).

One of the significant findings from this research is the effect of method of specimen compaction on performance in the RLA test. Given that gyratory compaction is used in a number of mixture design methods and considerable interest in this technique has been generated in the UK, largely through the development of harmonised European standards and the dissemination of the SHRP research, a compaction study similar to that described in Chapter Seven which included such a device would be of great interest and value.

For the RLA test to be implemented in performance specification, test criteria must be established. In order to achieve this, test data which are compatible both in terms of the test conditions and method of specimen preparation must be acquired for as wide a range of mixtures as possible. Ideally the performance of these mixtures should also be assessed by realistic, ie medium or large scale, wheeltracking tests under harsh environmental and loading conditions to provide an assessment of their relative probabilities of failure to provide adequate performance.

REFERENCES

1 Savage, C.I. "An economic history of transport". Hutchinson & Co. London, 1961.

2 "Road fact '94". British Road Federation. London, 1994.

3 Gillespie, H.M. "A century of progress. The history of hot mix asphalt". National Asphalt Pavement Association. Maryland, 1992.

4 "Statistical year book 1994". British Aggregate Construction Materials Industries. London, 1994.

5 Claessen, A.I.M., Edwards, J.M., Sommer, P. and Ugé, P. "Asphalt pavement design - the Shell method". Proceedings, Fourth International Conference on the Structural design of Asphalt Pavements. Volume 1, pp 39 - 74. Ann Arbor, 1977.

6 Pell, P.S. and Brown, S.F. "The characteristics of materials for the design of flexible pavement structures". Proceedings, Third International Conference on the Structural design of Asphalt Pavements. Volume 1, pp 326 - 342. London, 1972.

7 "Shell Pavement Design Manual - asphalt pavements and overlays for road traffic". Shell International Petroleum Company Ltd. London, 1978 with Addendum 1985.

8 Powell, W.D., Potter, J.F., Mayhew, H.C. and Nunn, M.E. "The structural design of bituminous roads". Laboratory Report 1132, Transport and Road Research Laboratory. Crowthorne, 1984.

9 Pell, P.S. and Taylor, I.F. "Asphaltic road materials in fatigue". Journal of the Association of Asphalt Paving Technologists, Volume 44, pp 1 - 37. The Association of Asphalt Paving Technologists, 1975.

10 Brown, S.F. and Brunton, J.M. "An Introduction to the analytical design of bituminous pavements". Third edition. University of Nottingham, 1986.

11 Brown, S.F. and Brunton, J.M. "Improvements to pavement subgrade strain criterion". Journal of Transportation Engineering, Volume 110, No. 6. American Society of Civil Engineers. November 1984.

12 Hofstra, A. and Klomp, A.J.G. "Permanent deformation of flexible pavements under simulated traffic conditions". Proceedings, Third International Conference on the Structural design of Asphalt Pavements. Volume 1, pp 613 - 621. London, 1972.

13 Eisenmann, J. and Hilmer, A. "Influence of wheel load and inflation pressure on the rutting effect at asphalt pavements - experiments and theoretical investigations". Proceedings, Sixth International Conference on the Structural design of Asphalt Pavements. Volume 1, pp 392 - 403. Ann Arbor, 1987.

14 Brown, S.F. "Stiffness and fatigue requirements for structural performance of asphaltic mixes". Eurobitume Seminar, pp 141 - 145. London, 1978.

15 Cooper, K.E. and Pell, P.S. "The effect of mix variables on the fatigue strength of bituminous materials". Laboratory Report 633, Transport and Road Research Laboratory. Crowthorne, 1974.

16 van der Poel, C.J. "A general system describing the visco-elastic properties of bitumens and its relation to routine test data". Journal of Applied Chemistry. Volume 4, pp 221 - 236, 1954.

17 "BS 2000. Methods of test for petroleum and its products. Part 49. Determination of needle penetration of bituminous material". British Standards Institution. London, 1993.

18 "BS 2000. Methods of test for petroleum and its products. Part 58. Determination of softening point of bitumen - Ring and ball method". British Standards Institution. London, 1993.

19 Sousa, J.B, Craus, J. and Monismith, C.L "Summary report on permanent deformation in asphalt concrete " Report SHRP-A/IR-91-104. Institute of Transportation Studies, University of California Berkeley, 1991.

20 Brown, S.F. and Cooper, K.E. "The mechanical properties of bituminous materials for roadbases and basecourses". Journal of the Association of Asphalt Paving Technologists, Volume 53, pp 415 - 436. The Association of Asphalt Paving Technologists, 1984.

21 "BS 4987. Coated macadam for roads and other paved areas. Part 1. Specification for constituent materials and for mixtures". British Standards Institution. London, 1993.

22 "BS 594. Hot rolled asphalt for roads and other paved areas. Part 1. Specification for constituent materials and asphalt mixtures". British Standards Institution. London, 1992.

23 Haydon, M.J.H. "Design of dense bitumen macadam and the development of performance specifications". Msc dissertation. University of Nottingham, 1994.

24 Department of Transport Specification for Highway Works (with Related Notes for Guidance). Various editions. Her Majesty's Stationery Office, London.

25 Kerali, H.G.R and Snaith, M.S. "A system for assigning priorities to maintenance projects in developing countries". Proceedings, Second International Conference on The Bearing Capacity of Roads and Airfields. Plymouth, 1986.

26 Bowskill, G.J. and Abell, R. "Whole life costing of road pavements - the way ahead". Proceedings, Seventh International Symposium on Concrete Roads. Vienna, 1994.

27 Department of Transport Design Manual for Roads and Bridges. Volume 7 : Pavement design and maintenance. Her Majesty's Stationery Office. London, 1994 with amendments at August 1995.

28 "DBFO bids call for 40 year road surfaces". New Civil Engineer, issue of 13 July 1995, pp 6. Thomas Telford Services Ltd. London, 1995.

29 Nunn, M.E. and Bowskill, G.J. "Towards a performance specification for bituminous roadbase". Proceedings, 7th International Conference on Asphalt Pavements, Volume 3, pp 266 - 295. International Society for Asphalt Pavements. Nottingham, 1992.

30 van der Plas, J.J "European standardisation of asphalt - a state of the art". Blacktop Developments in Europe and at Home. ACMA Seminar. Birmingham, 1992.

31 Hodgson, G.J. "Design and build - effects of contractor design on highway schemes". Proceedings of the Institution of Civil Engineers. Volume 108, Issue 2, pp 64 - 76. Institution of Civil Engineers. London, May 1995.

32 Cooper, K.E. and Brown, S.F. "Development of simple apparatus for the measurement of the mechanical properties of asphalt mixes". Proceedings, 4th Eurobitume Symposium, Volume 1, pp 494 - 498. Madrdid, 1989.

33 Brown, S.F., Preston, J.N. and Cooper, K.E. "Application of new concepts in asphalt mix design". Journal of the Association of Asphalt Paving Technologists, Volume 60, pp 264 - 286. The Association of Asphalt Paving Technologists, 1991.

34 Vallerga, B.A. and Lovering, W.R. "Evolution of the Hveem Stabilometer method of designing asphalt paving mixtures". Journal of the Association of Asphalt Paving

Technologists, Volume 54, pp 243 - 265. The Association of Asphalt Paving Technologists, 1985.

35 Hills, J.F. "The creep of asphalt mixes". Journal of the Institute of Petroleum, Volume 59, No. 570, pp 247 - 262. November 1973.

36 Kalcheff, I.V. and Tunnicliff, D.G. "Effects of crushed stone aggregate size and shape on properties of asphalt concrete". Journal of the Association of Asphalt Paving Technologists, Volume 51. The Association of Asphalt Paving Technologists, 1982.

37 Marks, V.J., Monroe, R.W. and Adam, J.F. "Effects of Crushed Particles in Asphalt Mixtures". Transportation Research Record 1259, pp 91 - 106. Transportation Research Board, National Research Council. Washington D.C., 1990.

38 Kandahl, P.S., Cross, S.A. and Brown, E.R. "Evaluation of heavy duty asphalt pavements for rutting". Proceedings, 7th International Conference on Asphalt Pavements, Volume 4, pp 64 - 78. International Society for Asphalt Pavements. Nottingham, 1992.

39 Button, J.W, Perdomo, D. and Lytton, R.L. "Influence of Aggregate on Rutting in Asphalt Concrete Pavements". Transportation Research Record 1259, pp 141- 152. Transportation Research Board, National Research Council. Washington D.C., 1990.

40 McHale, M.J. "Optimising the composition of HRA wearing courses". Journal of the Institution of Highways and Transportation, pp 11 - 13. March 1995.

41 Brien, D. "A design method for gap-graded asphaltic mixes". Shell Bitumen Review No.38, pp 11 - 15, 1972.

42 Code de bonne pratique pour la formulation des enrobés bitumineux denses. Recommendations C.R.R. - R 61/87. Centre De Recherches Routières, Bruxelles, 1987.

43 LAMBS - The design and use of large aggregate mixes for bases. Manual 13, Sabita Ltd. Roggebai, October 1993.

44 von Quintus, H.L, Scherocman, J.A., Hughes, C.S and Kennedy, T.W. "Asphalt-aggregate mixture analysis system AAMAS". National Cooperative Highway Research Program Report 338. Transportation Research Board, National Research Council. Washington D.C., March 1991.

45 Cominsky, R., Huber, G.A., Kennedy, T.W. and Anderson, M. "The Superpave mix design manual for new construction and overlays". Report SHRP-A-407. Strategic Highway Research Program, National Research Council. Washington DC, 1994.

46 Cominsky, R., Leahy, R.B. and Harrigan, E.T. " Level one mix design ; materials selection, compaction and conditioning". Report SHRP-A-408. Strategic Highway Research Program, National Research Council. Washington DC, 1994.

47 Mix design methods for asphalt concrete and other hot-mix types. Manual Series No.2. The Asphalt Institute. Kentucky, 1988 with amendments at January 1992.

48 Fuller, W.B. and Thompson, S.E. "The laws of proportioning concrete". Transactions of the American Society of Civil Engineers, Volume 59, pp 67 - 172. 1907.

49 Cooper, K.E., Brown, S.F. and Pooley, G.R. "The design of aggregate gradings for asphalt basecourses". Journal of the Association of Asphalt Paving Technologists, Volume 54, pp 324 - 347. The Association of Asphalt Paving Technologists, 1985.

50 SUPERPAVE™ Asphalt Mixture Design. National Asphalt Training Center Demonstration Project 101. Pilot Workshop Edition. Federal Highway Administration Office of Technology Applications, Washington, DC & Asphalt Institute Research Center, Lexington, Kentucky. February 1994.

51 International Bank for Reconstruction and Development on behalf of The Hashemite Kingdom of Jordan, "Road Pavement Study". Draft Final Report, Transport Research Laboratory. Crowthorne, November 1992.

52 Saarela, A. "Asphalt Pavements. Design". ASTO, The Finnish Asphalt Research Programme. VTT Technical Research Centre of Finland. Espoo, 1993.

53 "ASTM Standard D2041-90. Theoretical specific gravity of bituminous paving mixtures". American Society for Testing and Materials. Philadelphia, 1990.

54 McLeod, N.W. "Void requirements for dense-graded bituminous paving mixtures". ASTM STP 252, Bituminous Paving Materials, pp 81 - 112. American Society for Testing and Materials. Philadelphia, 1959.

55 Ford, M.C. "Pavement densification related to asphalt mix characteristics".Transportation Research Record 1178, pp 9 - 15. Transportation Research Board, National Research Council. Washington D.C., 1988.

56 Brown, E.R. and Cross, S.A. "Comparison of laboratory and field density of asphalt paving mixtures". Transportation Research Record 1300, pp 1 - 12. Transportation Research Board, National Research Council. Washington D.C., 1991.

57 Vallerga, B.A., Tayebali, A.A. and Monismith, C.L. "Early ruttting of asphalt concrete pavement under heavy axle loads in hot desert environment - a case history". Paper presented at the 1995 annual meeting of the Transportation Research Board.

58 Sousa, J.B. "Asphalt-aggregate mix design using the repetitive simple shear test (constant height)". Journal of the Association of Asphalt Paving Technologists, Volume 63, pp 298 - 345. The Association of Asphalt Paving Technologists, 1994

59 Finn, F.N., Monismith, C.L. and Markevich, N.J. "Pavement performance and asphalt concrete mix design". Journal of the Association of Asphalt Paving Technologists, Volume 52, pp 121 - 150. The Association of Asphalt Paving Technologists, 1983.

60 Francken, L. and Vanelstraete, A. "New developments in analytical asphalt mix design". Proceedings of the 5th Eurobitume Symposium. Stockholm, 1993.

61 O'Donnell, E., Fordyce, D, and Khweir, K. "Guidance on the design of fine dense bituminous material". Proceedings, 7th International Conference on Asphalt Pavements, Volume 1, pp 186 - 200. International Society for Asphalt Pavements. Nottingham, 1992.

62 "BS 598. Sampling and examination of bituminous mixtures for roads and other paved areas. Part 104. Methods of test for the determination of density and compaction". British Standards Institution. London,1989.

63 Lees, G. and Salehi, M. "Orientation of particles with special reference to bituminous paving materials". Highways Research Record No. 273, Highways Research Board. 1969.

64 Paterson, W.D.O. "Consideration of particle orientation in the compaction of asphalt concrete". Transportation Research Record No. 515, pp 151-156 (Abridgment). Transportation Research Board, National Research Council. Washington D.C., 1974.

65 Harvey, J., Eriksen, K., Sousa, J. and Monismith, C.L. "Effects of laboratory specimen preparation on asphalt-aggregate structure, air void content measurement, and simple shear test results". Paper submitted for presentation at the 1994 annual meeting of the Transportation Research Board.

66 Nevitt, H.G. "Some sources of stability measurement variations". Journal of the Association of Asphalt Paving Technologists, Volume 59. The Association of Asphalt Paving Technologists, 1959.

67 van de Loo, P.J. "The creep test : a key tool in asphalt mix design and in the prediction of pavement rutting". Journal of the Association of Asphalt Paving Technologists, Volume 47, pp 522-544. The Association of Asphalt Paving Technologists, 1978.

68 Brown, S.F. and Cooper, K.E. "A fundamental study of the stress-strain characteristics of a bituminous material". Journal of the Association of Asphalt Paving Technologists, Volume 49. The Association of Asphalt Paving Technologists, 1980.

69 Sousa, J.B., Deacon, J.A. and Monismith, C.L. "Effect of laboratory compaction method on permanent deformation characteristics of asphalt-aggregate mixtures". Journal of the Association of Asphalt Paving Technologists, Volume 60, pp 533 - 585. The Association of Asphalt Paving Technologists, 1991.

70 Hopman, P.C., Valkering, C.P. and van der Heide, J.P.J. "Mixes and five design procedures : search for a performance related mix design procedure". Journal of the Association of Asphalt Paving Technologists, Volume 61, pp 188 - 216. The Association of Asphalt Paving Technologists, 1992.

71 Nunn, M.E. "Deformation testing of dense coated macadam : effect of method of specimen compaction". Laboratory Report 870, Transport and Road Research Laboratory. Crowthorne, 1978.

72 Harvey, J., Monismith, C.L. and Sousa, J. "An investigation of field- and laboratory-compacted asphalt-rubber, SMA, recycled and conventional asphalt-concrete mixes using SHRP A-003A equipment". Paper submitted for presentation at the 1994 annual meeting of the Association of Asphalt Paving Technologists.

73 Acott, M. "Today's traffic calls for heavy duty asphalt mixes". Roads and Bridges, Volume 26, No. 1, pp 39 - 45. January 1988.

74 Brown, E.R and Bassett, C.E. "Effects of maximum aggregate size on rutting potential and other properties of asphalt - aggregate mixtures". Transportation Research Record 1259, pp 107 - 119. Transportation Research Board, National Research Council. Washington D.C., 1990.

75 Mahboub, K. and Allen, D.L. "Characterization of rutting potential of large-stone asphalt mixes in Kentucky". Transportation Research Record 1259, pp 133 - 140. Transportation Research Board, National Research Council. Washington D.C., 1990.

76 Kandhal, P.S. "Design of large-stone asphalt mixes to minimise rutting". Transportation Research Record 1259, pp 153 - 162. Transportation Research Board, National Research Council. Washington D.C., 1990.

77 Lister, N.W. and Powell, W.D. "The compaction of bituminous base and base-course materials and its relation to pavement performance". Journal of the Association of Asphalt Paving Technologists, Volume 44, pp 75 - 118. The Association of Asphalt Paving Technologists, 1975.

78 Leech, D, and Selves, N.W. "Deformation resistance of dense coated macadams : effect of compaction and binder content". Supplementary Report 626, Transport and Road Research Laboratory. Crowthorne, 1980.

79 Powell, W.D., Lister, N.W. and Leech, D. "Improved compaction of dense graded bituminous macadams". Journal of the Association of Asphalt Paving Technologists, Volume 50. The Association of Asphalt Paving Technologists, 1981.

80 White, T.D. "Marshall procedures for design and quality control of asphalt mixtures". Journal of the Association of Asphalt Paving Technologists, Volume 54, pp 265 - 284. The Association of Asphalt Paving Technologists, 1985.

81 Oliver, J.W.H. "The AUSTROADS asphalt mix design method - version 1". Research Report ARR 259. Australian Road Research Board Ltd. Victoria, Australia, November 1994.

82 Shuler, T.S. and Huber, G.A. "Effect of aggregate size and other factors on refusal density of asphalt concrete by vibratory compaction". ASTM STP 1147, Effects of Aggregates and Mineral Fillers on Asphalt Mixture Performance. American Society for Testing and Materials. Philadelphia, 1992.

83 Tuller, G. and van der Heide, J.P.J. "A study of the effects of a number of laboratory compaction devices on density level of asphalt concrete - summary". VBW - Asfalt. January 1994.

84 Jacobs, F.A. "A33 Winchester By-Pass : The performance of rolled asphalts designed by the Marshall test". Laboratory Report 1082, Transport and Road Research Laboratory. Crowthorne, 1983.

85 Choyce, P.W "Relationships between wheel-tracking and Marshall test results for 30% stone content rolled asphalt". Journal of the Institute of Asphalt Technology, September 1986.

86 van de Loo, P.J. "Creep testing, a simple tool to judge asphalt mix stability". Journal of the Association of Asphalt Paving Technologists, Volume 43, pp 253 - 284. The Association of Asphalt Paving Technologists, 1974.

87 Anani, B.A., Balghunaim, F.A. and Al-Hazza, A.S. "Laboratory and field study of pavement rutting in Suadi Arabia". Transportation Research Record 1259, pp 79 - 90. Transportation Research Board, National Research Council. Washington D.C., 1990.

88 Leahy, R.B., Monismith, C.L., Finn, F.N. and Hicks, R.G. "Recommended mix testing and analysis systems resulting from SHRP contract A-003A". Proceedings, 6th Conference on Asphalt Pavements for Southern Africa, Volume 2, pp IV-119 - IV-144. Cape Town, October 1994.

89 "Permanent deformation response of asphalt aggregate mixes". Report SHRP-A-415, Strategic Highway Research Program, National Research Council. Washington DC, 1994.

90 Decker, D., Ferragut, E., Holt, D. and Huber, G. "An industry discussion on Superpave implementation". Special Report 174, National Asphalt Pavement Association. Maryland, January 1995.

91 Sousa, J.B. "Asphalt - aggregate mix design using the repetitive simple shear test (constant height)". Journal of the Association of Asphalt Paving Technologists, Volume 63. The Association of Asphalt Paving Technologists, 1994.

92 Sousa, J.B., Harvey, J.T. and Partl, M.N. "Permanent deformation considerations in asphalt-aggregate mix design". Proceedings, 6th Conference on Asphalt Pavements for Southern Africa, Volume 1, pp III-229 - III-247. Cape Town, October 1994.

93 van der Heide, J.P.J. Discussion of Symposium on Asphalt Mix Design. Journal of the Association of Asphalt Paving Technologists, Volume 54, pp 315. The Association of Asphalt Paving Technologists, 1985.

94 Delorme, J.L. "French bituminous mix design method". Revue Générale des Routes et des Aérodromes. La Route Française. 1992.

95 Goacolou, H., Dimitri, A., Prudhomme, F. and George, L.A. "Use of subnormal local aggregates in high modulus asphalt mixes". Proceedings, 7th International Conference on Asphalt Pavements, Volume 3, pp 132 - 147. International Society for Asphalt Pavements. Nottingham, 1992.

96 Bonnot, J. "The partnership between contractors and administration as regards innovation for asphalt pavements in France". Blacktop Developments in Europe and at Home. ACMA Seminar, Birmingham, 1992.

97 Grimaux, J.P. and Hiernaux, R. "Utilisation de l'orniéreur type LCPC". Bulletin de Liaison des Laboratoires des Ponts et Chaussées, Numéro Spécial V, pp 165 - 172. LCPC, Paris, 1977.

98 Nievelt, G. and Thamfald, J. "Evaluation of the resistance to deformation of different road structures and asphalt mixtures determined in the pavement rutting tester". Journal of the Association of Asphalt Paving Technologists, Volume 57, pp 320 - 345. The Association of Asphalt Paving Technologists, 1988.

99 Sosnovske, D., Leahy, R.B. and Hicks, R.G. "Evaluation of rutting potential of Oregon surface mixtures". Journal of the Association of Asphalt Paving Technologists, Volume 63. The Association of Asphalt Paving Technologists, 1994.

100 Aschenbrener, T.B. "Comparison of results obtained from the LCPC rutting tester with pavements of known field performance". Paper presented at the 1994 annual meeting of the Transportation Research Board.

101 Norme Française NF P 98-253-1 "Deformation Permanente des Mélanges Hydrocarbonés. Partie 1 : Essai d'Orniérage ". Association Française de Normalisation, 1991.

102 Hopman, P.C., Kunst, P.A.J.C., Pronk, A.C., Molenaar, J.M.M. and Molenaar, A.A.A. "Asphalt research in the Netherlands". Proceedings, VTI/TRB Conference. Gothenburg, Sweden, 1991.

103 Collins, R., Watson, D.W. and Campbell, B. "Development and use of the Georgia loaded wheel tester". Paper presented at the 1995 annual meeting of the Transportation Research Board.

104 "The wheel-tracking test". Leaflet 50, Road Research Laboratory. Crowthorne, 1971.

105 Pfeiffer, J. and van Doormaal, P.M. "The rheological properties of asphaltic bitumens". Journal of the Institute of Petroleum. 1936.

106 Pagen, C.A. "Dynamic structural properties of asphalt pavement mixtures". Proceedings, Third International Conference on the Structural design of Asphalt Pavements. Volume 1, pp 290 - 316. London, 1972.

107 Lai, J.S. and Lee, T. "Use of a loaded-wheel testing machine to evaluate rutting of asphalt mixes". Transportation Research Record 1269, pp 116 - 124. Transportation Research Board, National Research Council. Washington, D.C., 1990.

108 de Hilster, E and van de Loo, P.J. "The creep test : influence of test parameters". Proceedings, Colloquium 77 Plastic Deformability of Bituminous Mixes, pp 173 - 215. Institut für Strassen-, Eisenbahn- und Felsbau an der Eidgenössischen Technischen Hochschule. Zürich, 1977.

109 Hills, J.F., Brien, D. and van de Loo, P.J. "The correlation of creep and rutting". Institute of Petroleum, Paper IP 74-001. 1974.

110 Goetz, W.H., McLaughlin, J.F. and Wood, L.E. "Load-deformation characteristics of bituminous mixtures under various conditions of loading". Journal of the Association of Asphalt Paving Technologists, Volume 26, pp 237 - 296. The Association of Asphalt Paving Technologists, 1957.

111 Valkering, C.P., Lancon, D.J.L., de Hilster, E. and Stoker, D.A. "Rutting resistance of asphalt mixes containing non-conventional and polymer modified binders". Journal of the Association of Asphalt Paving Technologists, Volume 59, pp 590 - 609. The Association of Asphalt Paving Technologists, 1990.

112 Cooper, K.E. and Brown, S.F. "Assessment of the mechanical properties of asphaltic mixes on a routine basis using simple equipment". Proceedings, Eurobitume Congress, Volume 1B, pp 872 - 876. Stockholm, 1993.

113 Huschek, S. "The deformation behaviour of asphaltic concrete under triaxial compression". Journal of the Association of Asphalt Paving Technologists, Volume 54, pp 407 - 431. The Association of Asphalt Paving Technologists, 1985.

114 Monismith, C.L. and Tayebali, A.A. "Permanent deformation (rutting) considerations in asphalt concrete pavement sections". Journal of the Association of Asphalt Paving Technologists, Volume 57, pp 414 - 463. The Association of Asphalt Paving Technologists, 1988.

115 Brown, S.F. and Cooper, K.E. "Permanent deformation of dense bitumen macadam under creep and repeated loading". Proceedings, Colloquium 77 Plastic Deformability of Bituminous Mixes, pp 139 - 156. Institut für Strassen-, Eisenbahn- und Felsbau an der Eidgenössischen Technischen Hochschule. Zürich, 1977.

116 Sousa, J.B. and Weissman, S.L. "Modelling permanent deformation of asphalt - aggregate mixes". Journal of the Association of Asphalt Paving Technologists, Volume 64. The Association of Asphalt Paving Technologists, 1994.

117 Lai, J.S. and Hufferd, W.L. "Predicting permanent deformation of asphalt concrete from creep tests". Transportation Research Record 616, pp 41 - 43 (Abridgment). Transportation Research Board, National Research Council. Washington, D.C., 1976.

118 "BS 3690. Part 1. Specification for bitumens for roads and other paved areas". British Standards Institution. London, 1989.

119 Cooper, K E and Brown, S F, "Asphapol Cushion Course as an Overlay Material". University of Nottingham, 1984.

120 Mechanical properties of polyphalt bituminous roadbase: laboratory evaluation". SWK Pavement Engineering. Nottingham,1986.

121 de Beer, M. "Heavy vehicle simulator (HVS) testing on national route 2 (N2) section 23 with a styrene butadiene latex rubber (SBR) modified asphalt surfacing". Report No. DPVT/C-128.1. Pavement Engineering Technology Programme, Division of Roads and Transport Technology, CSIR. Pretoria, 1990.

122 Brodrick, B V, "The development and performance of a wheeltracking facility and in-situ instruments for pavement experiments", MPhil thesis. University of Nottingham, 1977.

123 Clegg, B "An impact testing device for in-situ basecourse evaluation", Proceedings, 8th Australian Road Research Board Conference, University of Western Australia, 1976.

124 Brown, S.F. and Snaith, M.S. "The permanent deformation characteristics of a dense bitumen macadam subjected to repeated loading". Journal of the Association of Asphalt Paving Technologists, Volume 43, pp 224 - 252. The Association of Asphalt Paving Technologists, 1974.

125 Bonnot, J. "Discussion paper on uniaxial and triaxial repeated load testing of bituminous mixtures". Paper presented to CEN Committee TC227 (WG1 TG2 - Test Methods). Vienna, 1994.

126 Brown, S.F. and Cooper, K.E. "Simplified methods for determination of fundamental properties of asphalt mixes". Proceedings, Conference on Strategic Highway Research Program and Traffic Safety on Two Continents. No. 1A, Part 3, pp 124 - 136. The Hague, 1993.

127 Lee, X. and Dass, W.C. "An experimental study of granular packing structure changes under load". Powders and Grains '93, pp 17 - 22. Balkema. Rotterdam, 1993.

128 Potter, D. "Test procedures and specification guidelines", Asphalt Review, Volume 11, No.4, pp 11-13. Australian Asphalt Pavement Association, 1992.

129 "Determination of the permanent compressive strain characteristics of hot mix asphalt by the dynamic creep test". Australian Standard AUST-002, Revision D04. August 1992.

130 "Recommendation for the performance of unconfined static creep test on asphalt specimens". Proceedings, Colloquium 77 Plastic Deformability of Bituminous Mixes, pp 355 - 357. Institut für Strassen-, Eisenbahn- und Felsbau an der Eidgenössischen Technischen Hochschule. Zürich, 1977

131 Morris, J., Haas, R.G., Reilly, P. and Hignell, E.T. " Permanent deformation in asphalt pavements can be predicted". Journal of the Association of Asphalt Paving Technologists, Volume 43. The Association of Asphalt Paving Technologists, 1974.

132 Preston, J.N. " The Design of Bituminous Concrete Mixes ". Ph.D thesis. University of Nottingham, 1991.

133 Snaith, M.S. "Deformation characteristics of dense bitumen macadam subjected to dynamic loading". Ph.D thesis. University of Nottingham, 1973.

134 Proposed revision to "DD185. Method for assessment of resistance to permanent deformation of bitumen aggregate mixtures subject to unconfined uniaxial loading". British Standards Institution. London, 1993.

135 Oliver, J.W.H. "Identification of some factors affecting the precision of the creep test". Working Document No. WD RI93/012. Australian Road Research Board Ltd. Victoria, Australia, November 1993.

136 Nevitt, H.G. "Compaction fundamentals". Journal of the Association of Asphalt Paving Technologists, Volume 26, pp 201 - 206. The Association of Asphalt Paving Technologists, 1957.

137 Van de Loo, P.J. "The creep test : a key tool in asphalt mix design and in the prediction of pavement rutting". Journal of the Association of Asphalt Paving Technologists, Volume 47, pp 522 - 544. The Association of Asphalt Paving Technologists, 1978.

138 Aussedat, G. "L'essai de fluage dynamique dans la formation des enrobés et le dimensionnement des chaussées". Bulletin de Liaison des Laboratoires des Ponts et Chaussées, Numéro Spécial V, pp 139 - 151. LCPC, Paris, 1977.

139 "Design and testing of bituminous mixtures". Volume 2 : Research Report, pp 8.14 - 8.24. Report submitted to DOT/EPSRC LINK Programme on Transport Infrastructure and Operations. University of Nottingham, 1995.

140 Button, J.W., Little, D.N., Jagadam, J. and Pendleton, O.J. "Correlation of selected laboratory compaction methods with field compaction". Paper submitted for presentation at the 1994 annual meeting of the Transportation Research Board.

141 Harvey, J., Monismith, C.L. and Sousa, J.B. "An investigation of field- and laboratory-compacted asphalt-rubber, SMA, recycled and conventional asphalt-concrete mixes using SHRP A-003A equipment". Paper submitted for presentation at the 1994 annual meeting of the Association of Asphalt Paving Technologists.

142 Cooper, K.E., Brown, S.F., Preston, J.N. and Akeroyd, F.M.L. "Development of a practical method for design of hot-mix asphalt". Transportation Research Record 1317, pp 42 - 51. Transportation Research Board. National Research Council, Washington D.C., 1991.

143 "Design and testing of bituminous mixtures". Volume 4 : Case Studies. Report submitted to DOT/EPSRC LINK Programme on Transport Infrastructure and

144 Perl, M., Uzan, J. and Sides, A. "Visco-elasto-plastic constitutive law for a bituminous mixture under repeated loading". Transportation Research Record 911, pp 20 - 27. Transportation Research Board. National Research Council, Washington, D.C., 1983.

145 Faure, M., Simoncelli, J-P. and Bertaux, J-M. "Rutting of bituminous mixes - the effect of heavy goods vehicles". Proceedings, 5th Eurobitume Congress. Stockholm, 1993.

146 Denning, J.H. and Carswell, J. "Improvements in rolled asphalt surfacings by the addition of organic polymers". Laboratory Report 989. Transport and Road Research Laboratory. Crowthorne, 1981.

147 Nahas, N.C., Bardet, J., Eckmann, B. and Siano, D.B. "Polymer modified asphalts for high performance hot mix pavement binders". Journal of the Association of Asphalt Paving Technologists, Volume 59. The Association of Asphalt Paving Technologists, 1990.

148 Decker, D.S. and Goodrich, J.L. "Asphalt cement properties related to pavement performance". Journal of the Association of Asphalt Paving Technologists, Volume 58. The Association of Asphalt Paving Technologists, 1989.

149 Reese, R.E. and Goodrich, J.L. "California desert test road - a step closer to performance based specifications". Journal of the Association of Asphalt Paving Technologists, Volume 62, pp 247 - 313. The Association of Asphalt Paving Technologists, 1993.

150 Nunn, M.E. and Smith, T. "Evaluation of a performance specification in road construction". Project Report 55, Transport Research Laboratory. Crowthorne, 1994.

151 Goodrich, J.L. "Asphaltic binder rheology, asphaltic concrete rheology and asphalt mix properties". Journal of the Association of Asphalt Paving Technologists, Volume 60, pp 80 - 120. The Association of Asphalt Paving Technologists, 1991.

152 Kennedy, T.W., Huber, G.A., Harrigan, E.T., Cominsky, R.J., Hughes, C.S., von Quintus, H. and Moulthrop, J.S. "Superior performing asphalt pavements (Superpave) : the product of the SHRP asphalt research program". Report SHRP-A-410, Strategic Highway Research Program, National Research Council. Washington DC, 1994.

153 "Design and testing of bituminous mixtures". Executive Summary. Report No. PGR95026, submitted to DOT/EPSRC LINK Programme on Transport Infrastructure and Operations. University of Nottingham, 1995.

154 Duthie. J.L. "The Marshall test in the design of hot rolled asphalt wearing course". Mix Design of Asphalt Now and in the Future. Joint Institution of Highway Engineers and Society of chemical Industry symposium. January 1974.

155 Raybould, M.J. "The response of silt-clay mixtures to cyclic loading". Ph.D thesis. University of Nottingham, 1992.

APPENDIX A

MIXTURE DESIGN PROCEDURES

A.1 The Objective of Mixture Design

Mixture requirements, in terms of mechanical properties, will generally vary according to environmental and loading conditions, the location and function of the material within the pavement structure and also the type of pavement structure. Environmental conditions in the UK are such that the extremely low temperatures which can cause thermal cracking in more severe climates do not generally occur, hence this form of distress is not a principal consideration in this country. Durability is essential for wearing courses exposed to sunlight, rainfall, ice and de-icing salts, but would be of less concern for roadbase positioned deeper in the pavement structure. Flexible pavements designed to carry high traffic loadings generally have large thicknesses of bituminous materials which, as the main structural component, should have high stiffness to distribute the load transmitted to the pavement foundation. In pavements designed to carry very low traffic loadings, however, bituminous material may be used principally as a surfacing over an unbound structural base in which case the mixture would have to be relatively flexible to accommodate large transient strains from vehicle loading.

Mixture requirements often conflict. For example, in climates which have extremes of low and high temperatures both thermal cracking and rutting are likely to be of concern. Thermal cracking can be alleviated through use of high contents of soft binder while low contents of hard binder are generally beneficial for resistance to permanent deformation. A further consideration is the required level of performance ; rut depths deemed acceptable for lightly trafficked, low speed roads may not be tolerated on major routes with high levels of high speed traffic, for reasons of safety and serviceability. The process of mixture design is, therefore, generally one of optimisation to determine an economic composition with an appropriate balance of mechanical properties to suit the application.

A.2 Mixture Design Procedures

The Marshall procedure, which is a long established and widely used method of mixture design (47,80), provides a means for the optimisation of binder content for a particular aggregate grading. However, one of the chief limitations of this method is that the parameters considered in the optimisation do not reliably relate to the mechanical properties of the mixture (20, 70). Also, while the Marshall procedure may be amended by individual specifying authorities, once established the procedure and criteria are generally only varied to accommodate different levels of traffic. Thus any bias of the procedure which would favour a particular mechanical property (33), or properties, cannot readily be changed to suit applications where other properties may be of greater concern.

Recent developments in mixture design have tended to concentrate on assessing parameters which directly relate to performance. The Superpave method (45), is one such procedure which uses both performance-based and performance-related properties. Performance-based properties are defined as those which "*directly govern the response of the pavement to load*" and from which performance can be predicted, while performance-related properties "*affect performance, but do not, in themselves control it*". On this basis, resistance to deformation measured in a mechanical test would be a performance-based property while a volumetric criterion for, say, minimum air void content would be a performance-related property. The concept of the Superpave mixture design system is shown in the flow chart presented in Figure A.1. though, in effect, the overall procedure indicated serves as a general illustration of this approach to mixture design.

The differences between various methods of design which use performance-based and performance-related parameters are largely in the methods of test used to assess performance, and in the degree of control which is placed on the composition in order to achieve specified performance criteria. As shown on Figure A.1, at the levels of design which require mechanical testing the Superpave system uses tests to measure resistance to permanent deformation, fatigue cracking and low temperature cracking. Table A.1 shows the testing regime and general procedure for the French method of mixture design (94), illustrating both the mechanical properties assessed and the variation in complexity of the procedure with the level of mixture assessment required. It can be seen that the full procedure, which incorporates direct assessment of fatigue resistance, is reserved for only completely new designs but assessment of rutting resistance is included at the mixture adaptation level. Details of the test methods employed in both the Superpave and French methods are well documented (45, 94).

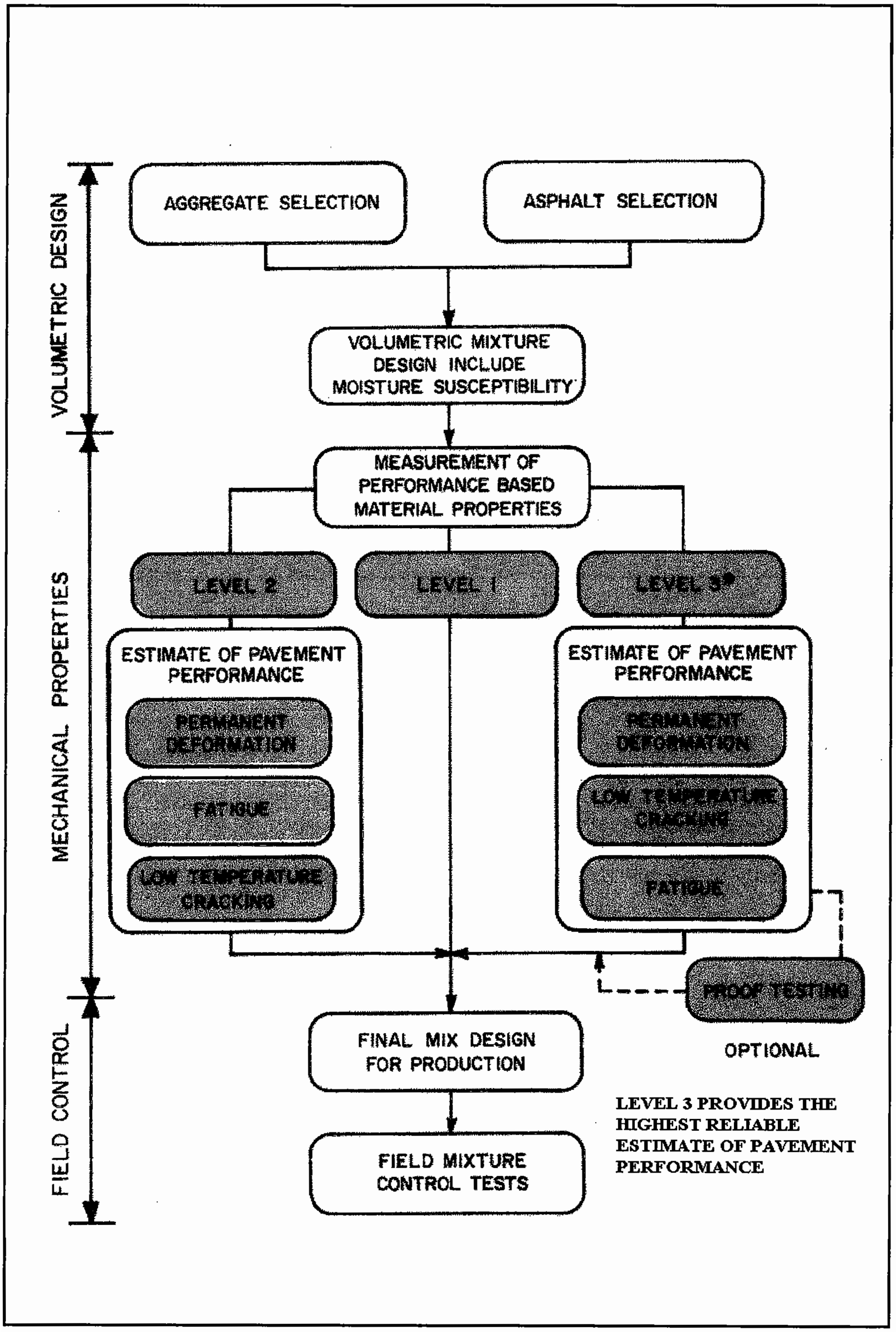


Figure A.1 Structure of the Superpave Mixture Design System
(reproduced from reference 45)

Table A.1 Principle of the French Mixture Design Method
(Adapted from reference 94)

Verification of a Mixture Design Already Completed and Applied	Adaptation of a Mixture Design Where One or More Constituents Have Changed	New Mixture Design
Gyratory compaction test to assess workability/compactibility	Gyratory compaction test on several candidate formulations to assess workability/compactibility	Gyratory compaction test on several candidate formulations to assess workability/compactibility
⇓	⇓	⇓
	One formulation selected	Several formulations selected
	⇓	⇓
Duriez test at 18°C to assess water susceptibility	Duriez test at 18°C to assess water susceptibility	Duriez test at 18°C to assess water susceptibility
	⇓	⇓
	Rutting test	Rutting test [1]
		⇓
		One formulation selected
		⇓
		Characterisation test on mechanical performance of mixture [2]

[1] Tests to be carried out where the risk of rutting is high, for example :
- Slow, channelised traffic
- Use of rounded sand etc.

[2] Tests to be carried out where the mechanical properties of the mixture are used for pavement design. The tests will be from among :
- Stiffness test , direct tension
- Stiffness test (complex modulus), two-point bending
- Fatigue test

The differences in the degree of control placed on mixture composition between the Superpave system and the French method of mixture design were discussed in Chapter Two of this thesis. It is worth noting that developments to the Nottingham mixture design method, which arose from the research carried out under the Link Bitutest Project (153), included relaxation of such controls which were felt to be at variance with a performance-oriented approach to mixture design. Figure A.2 shows a flow chart for the original version of the Nottingham method (33), similar in principle to that for the Superpave system, demonstrating the specific requirements for target aggregate gradings and binder content. The format of the method has now been revised to permit assessment of a wide range of mixture types using the same basic principle of approach, but retaining the target gradings and binder contents of the original method for guidance where required (139).

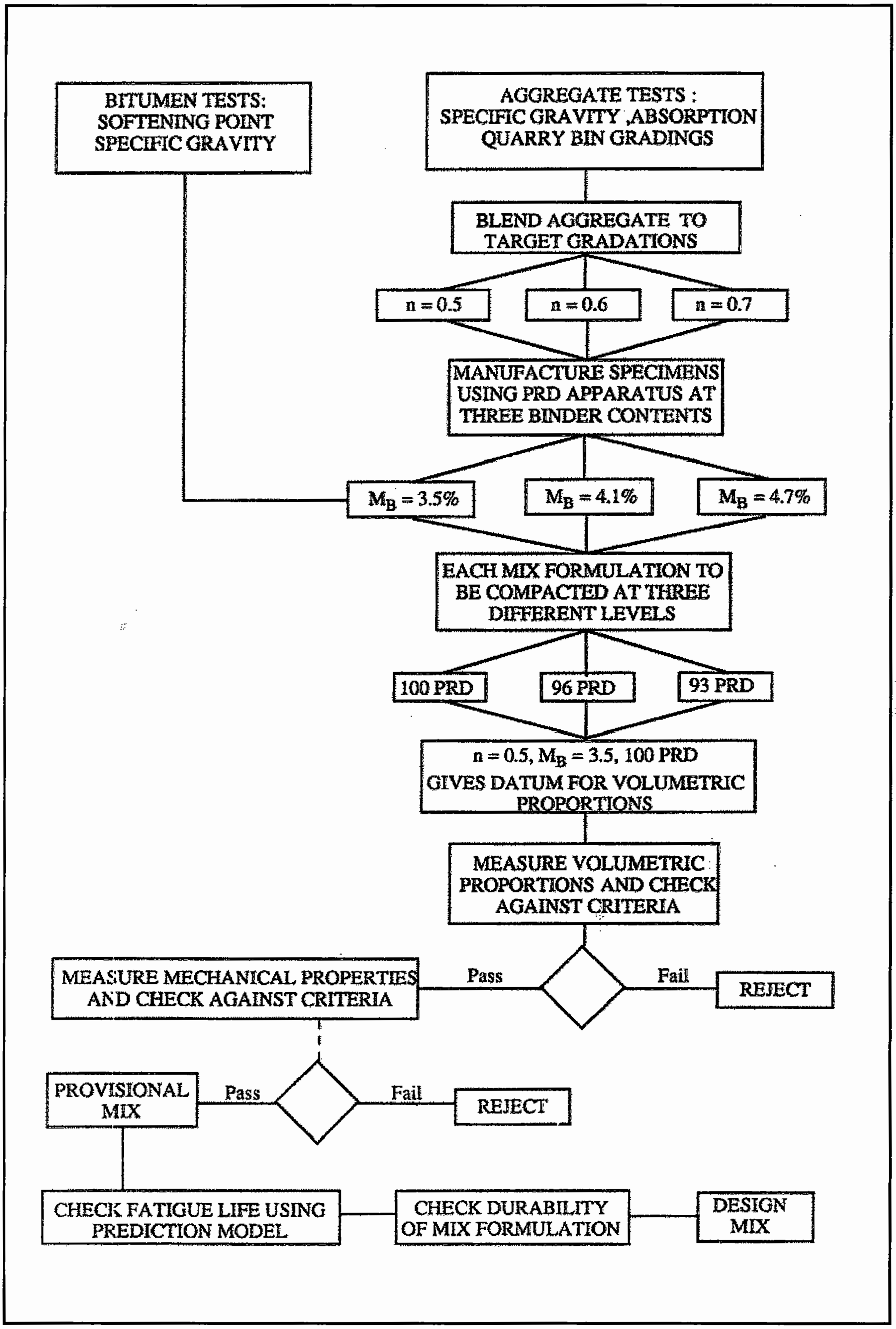


Figure A.2 Flow Chart of the Nottingham Mixture Design Procedure
(reproduced from reference 33)

APPENDIX B

CONSTRUCTION AND TESTING OF THE TRIAL PAVEMENT :

ADDITIONAL DATA

B.1 Tests for Uniformity on the Pavement Foundation

B.1.1 Subgrade

Moisture Content: A single sample was taken from the upper 100mm of the clay in each of the four sections of the trial pavement and determinations of moisture content were made on the upper, middle and lower portions of each sample. The results, presented in Table B1, show moisture contents in sections B and D to be marginally higher than those in A and C, and also indicate an increase in moisture content with depth in each section.

Table B1 Subgrade Moisture Contents (%)

	Section A	Section B	Section C	Section D
Top	11.2	12.3	11.1	12.3
Middle	12.3	14.5	12	13.8
Bottom	12.4	13.6	13	14.7

California Bearing Ratio: Table B2 contains the results of CBR tests which were carried out in situ using the loading frame of the PTF to provide reaction, with load applied by a hydraulic jack. All the values are high for a clay subgrade and indicate a strong formation.

Table B2 In situ subgrade CBR's (%)

	Section A	Section B	Section C	Section D
Test 1	15	20	23	15
Test 2	19	14	26	16

Clegg Impact Value: A series of three drops of the Clegg Hammer was performed at each test location, and the Clegg Impact Value (CIV) from the final drop was recorded. These values are presented in Table B3.

Table B3 Clegg Impact Values Recorded on the Subgrade

	Section A	Section B	Section C	Section D
Test 1	20	13	18	13
Test 2	17	12	20	30
Test 3	21	16	22	14
Test 4	18	15	19	14
Test 5	17	19	21	12
Test 6	12	17	21	15
Mean	18	15	20	16

The testing on the subgrade revealed little difference between any of the sections, notwithstanding the low number of repeat determinations of CBR and moisture content, and the overall indication was of a stiff and uniform formation.

B.1.2 Sub-base

The Clegg Hammer was used, following the same procedure as used on the subgrade, on the final compacted surface of the sub-base to assess uniformity. The CIV's presented in Table B4 show relatively low variation between the four sections.

Table B4 Clegg Impact Values Recorded on the Sub-base

	Section A	Section B	Section C	Section D
Test 1	48	35	32	42
Test 2	31	34	40	67
Test 3	49	36	44	39
Test 4	40	37	60	41
Test 5	47	44	40	48
Test 6	44	38	53	48
Mean	43	37	45	48

B.2 Permanent Strain Profiles

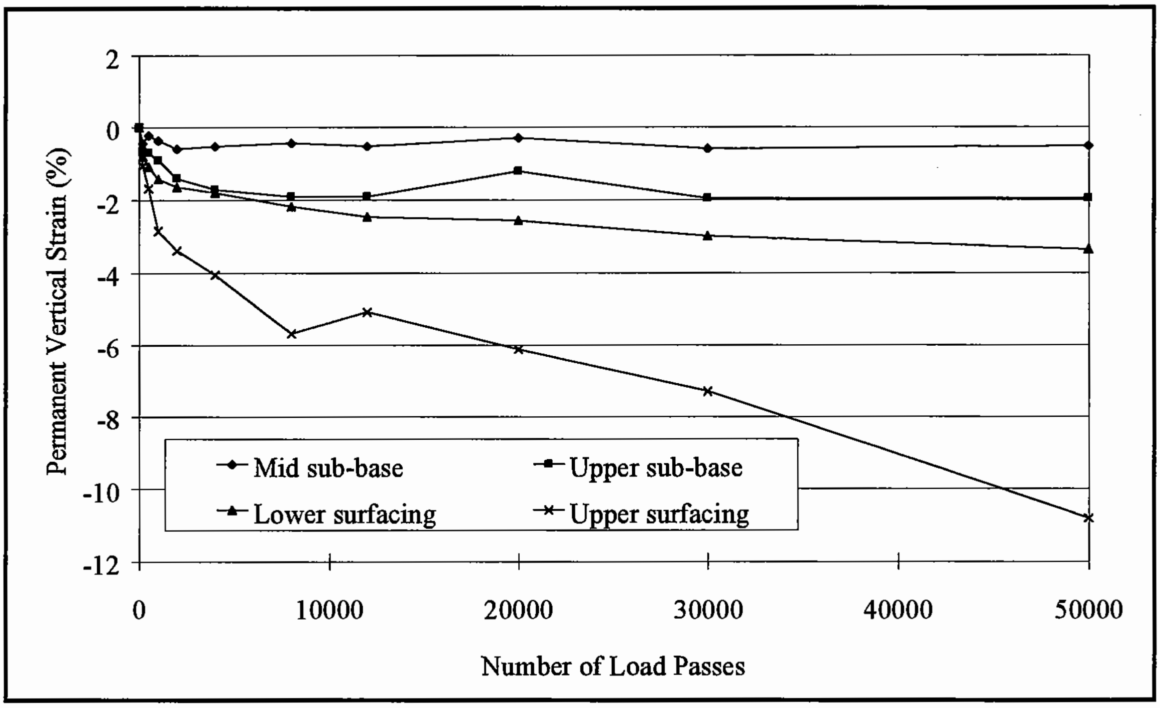


Figure B1 Development and Distribution of Permanent Vertical Strain : Section A Coil Stack 3

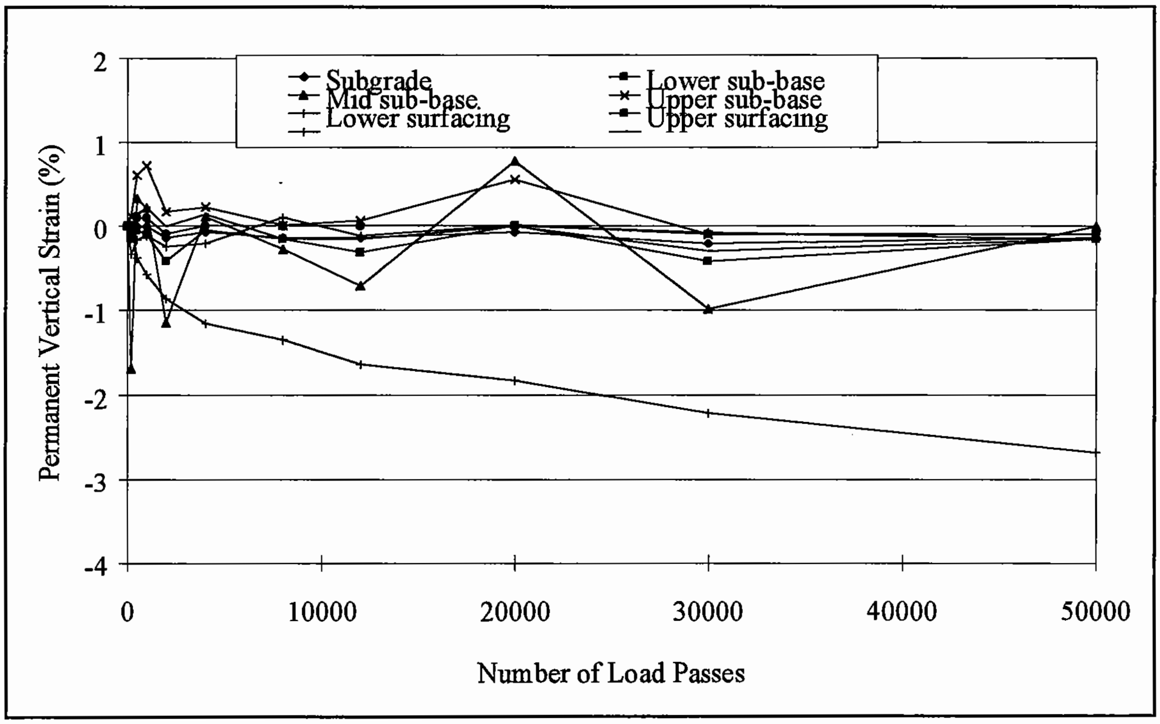


Figure B2 Development and Distribution of Permanent Vertical Strain : Section B Coil Stack 2

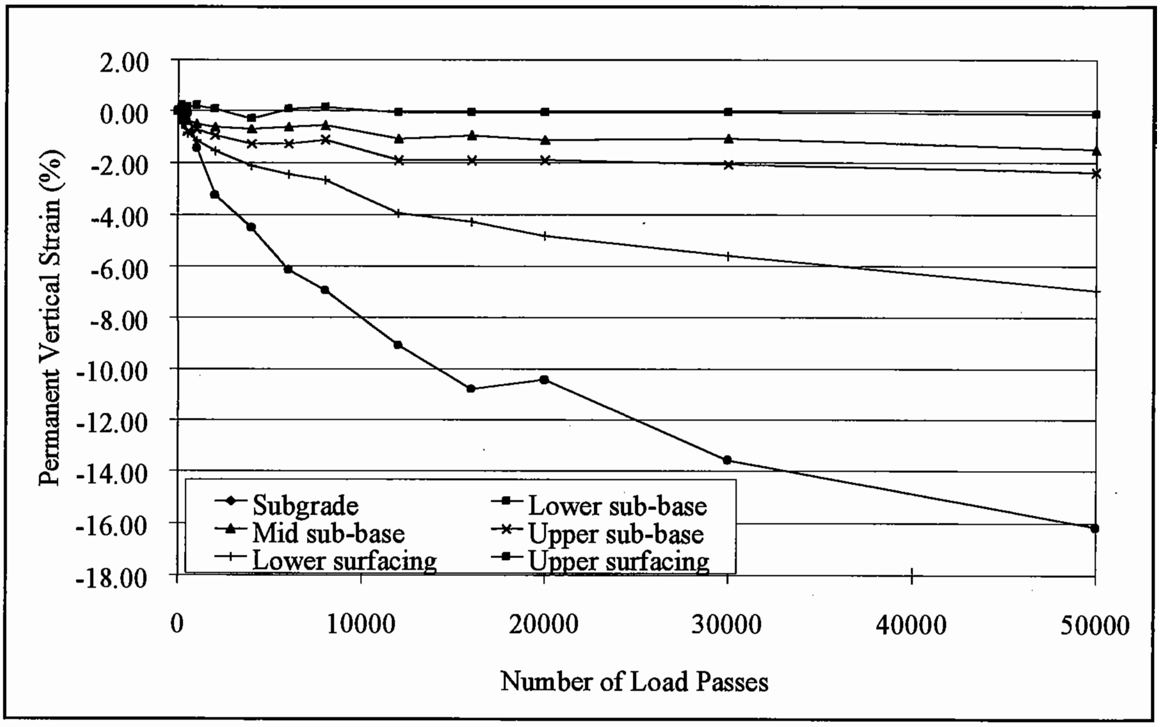


Figure B3 Development and Distribution of Permanent Vertical Strain : Section C Coil Stack 7

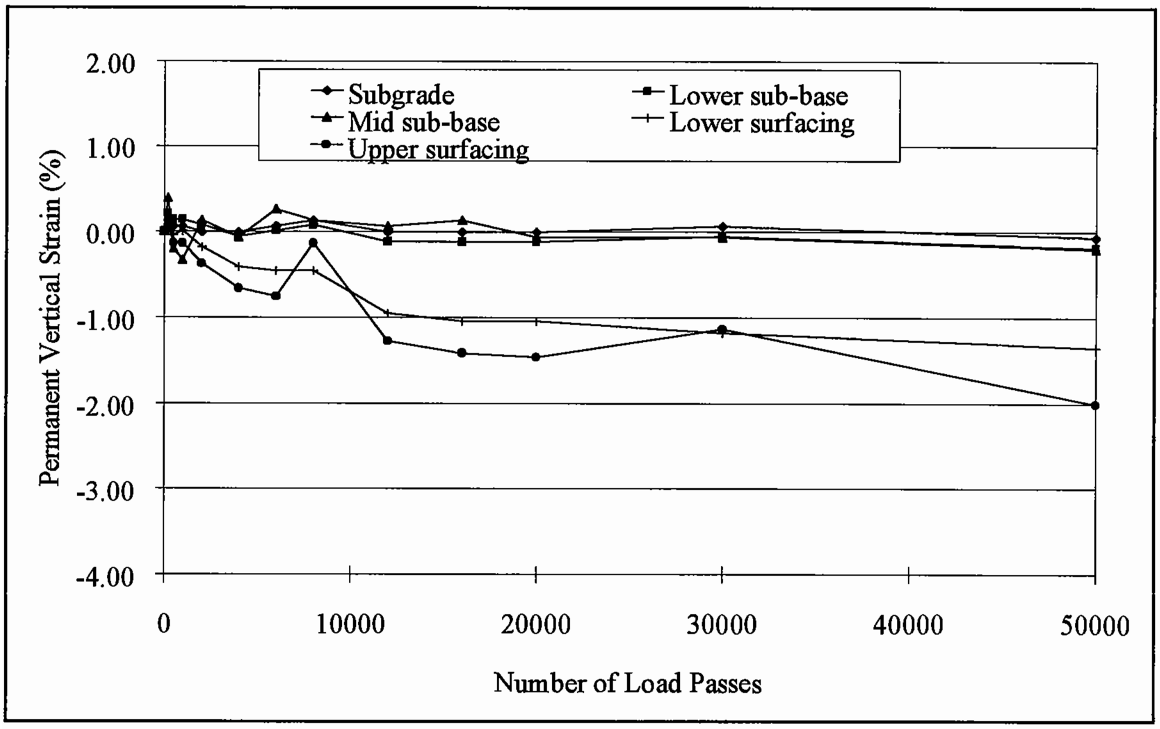


Figure B4 Development and Distribution of Permanent Vertical Strain : Section D Coil Stack 6

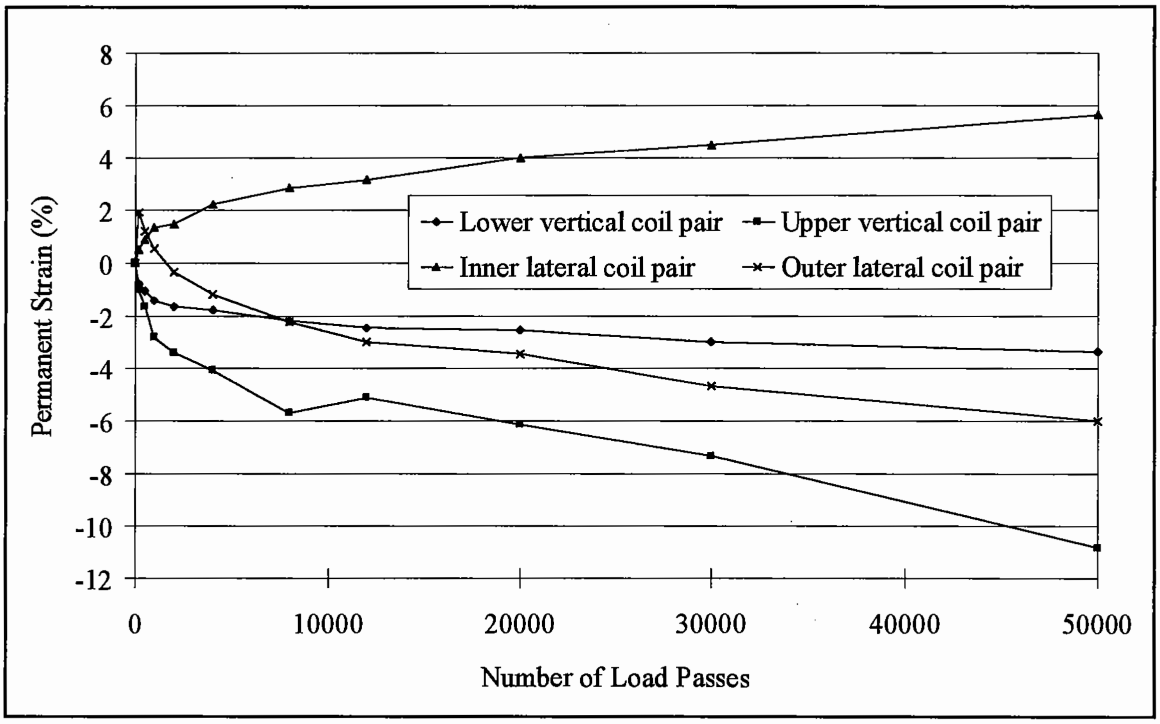


Figure B5 Development and Distribution of Strain within the Bituminous Surfacing Section A, Coil Stack 3

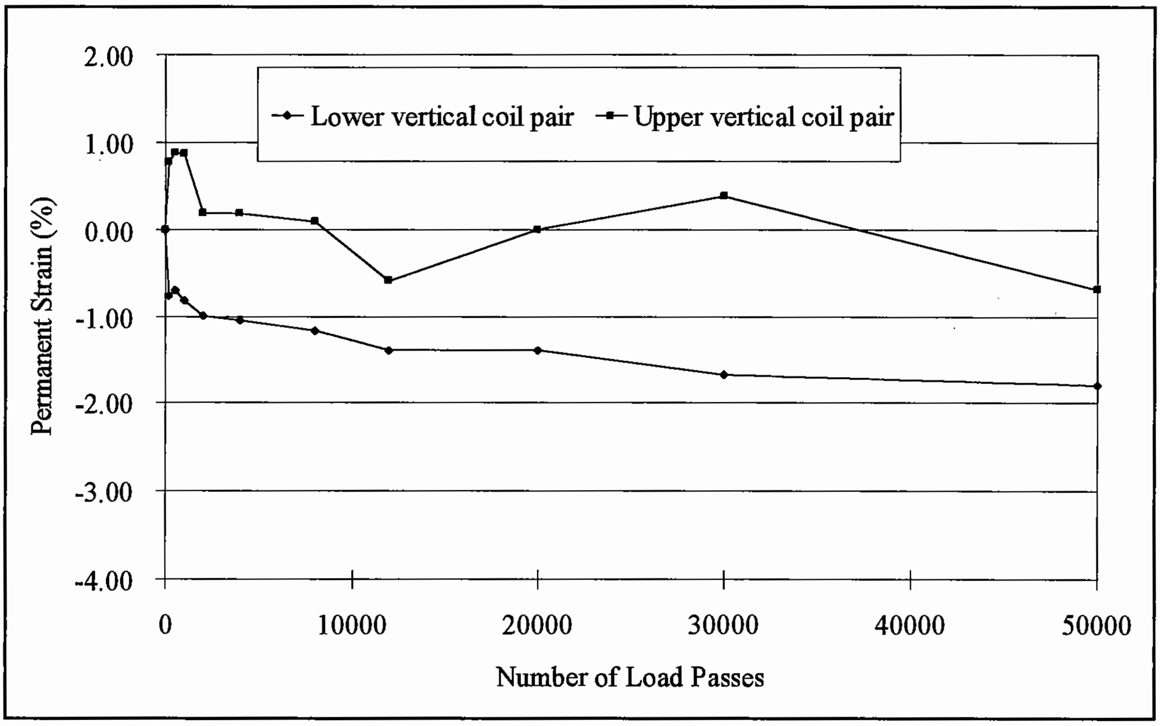


Figure B6 Development and Distribution of Strain within the Bituminous Surfacing Section B, Coil Stack 1

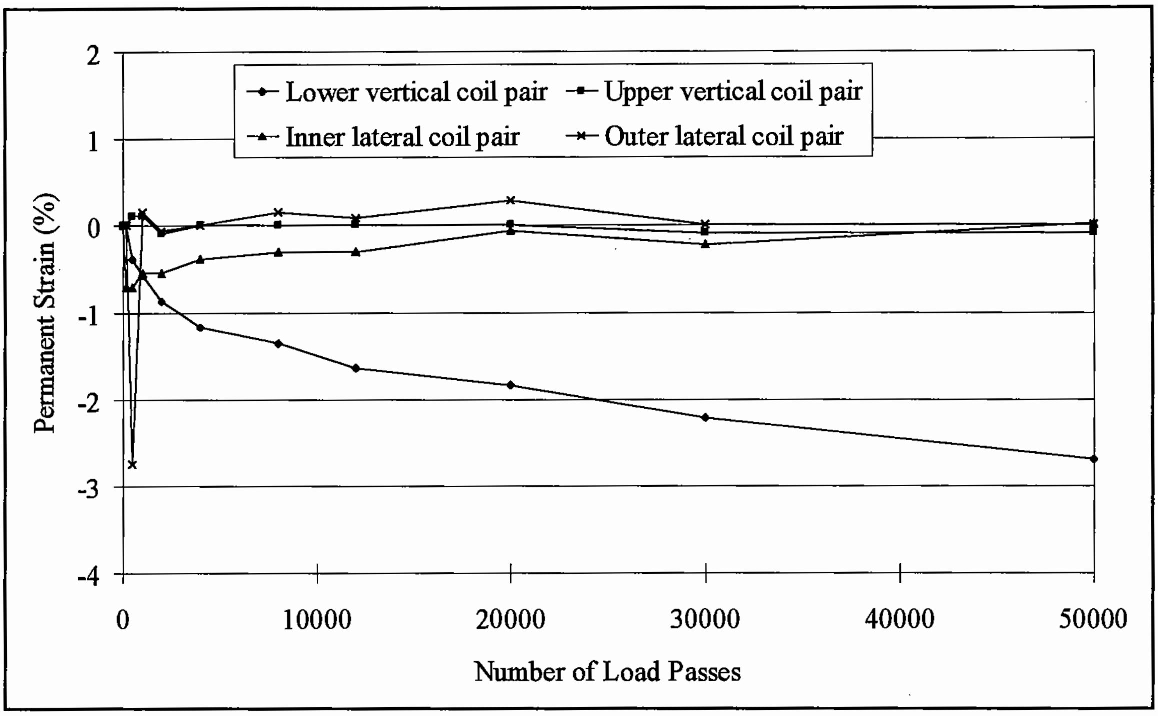


Figure B7 Development and Distribution of Strain within the Bituminous Surfacing Section B, Coil Stack 2

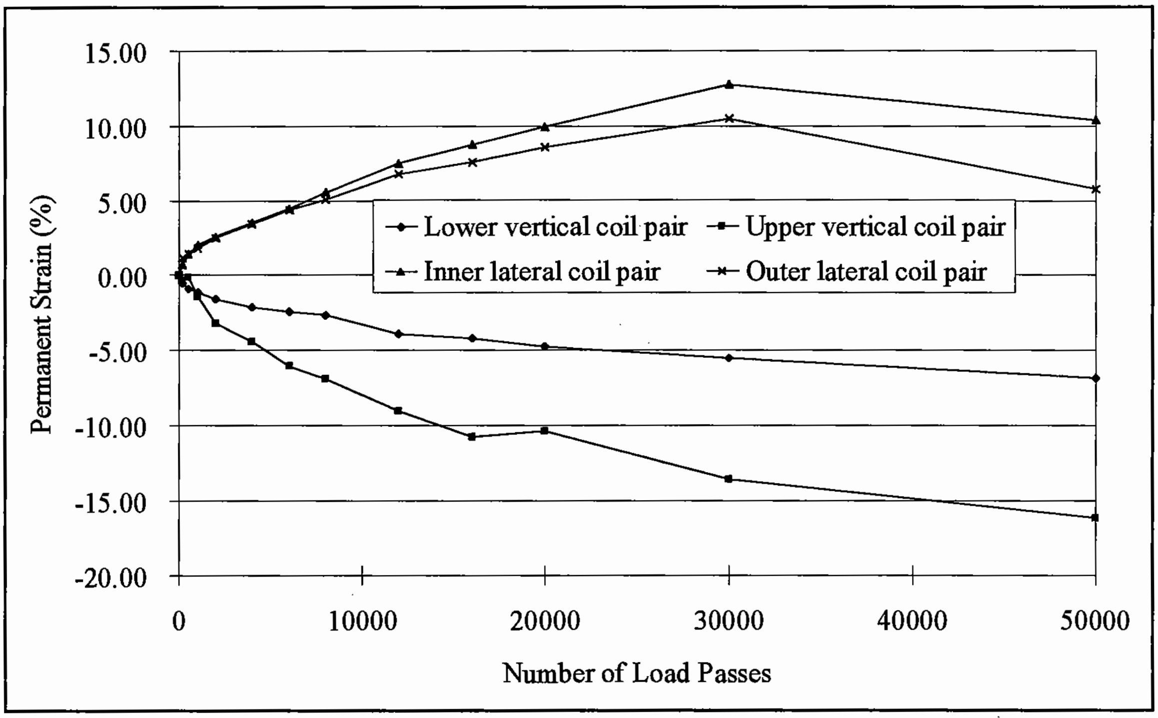


Figure B8 Development and Distribution of Strain within the Bituminous Surfacing Section C, Coil Stack 7

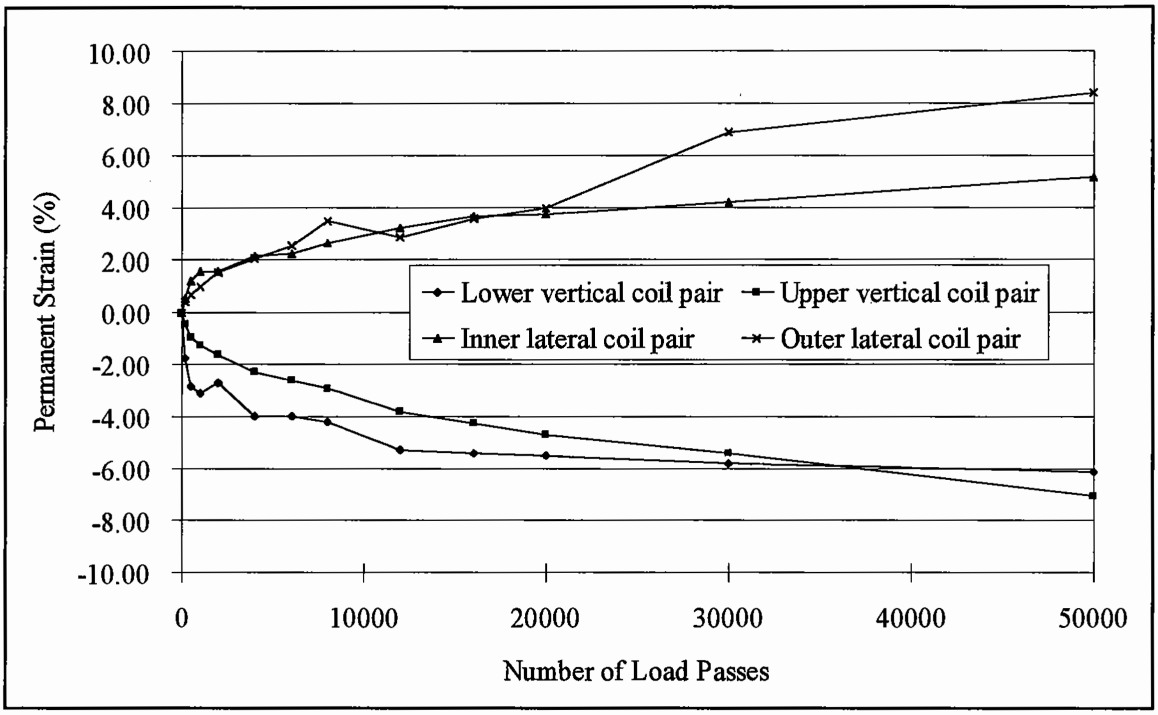


Figure B9 Development and Distribution of Strain within the Bituminous Surfacing Section C, Coil Stack 8

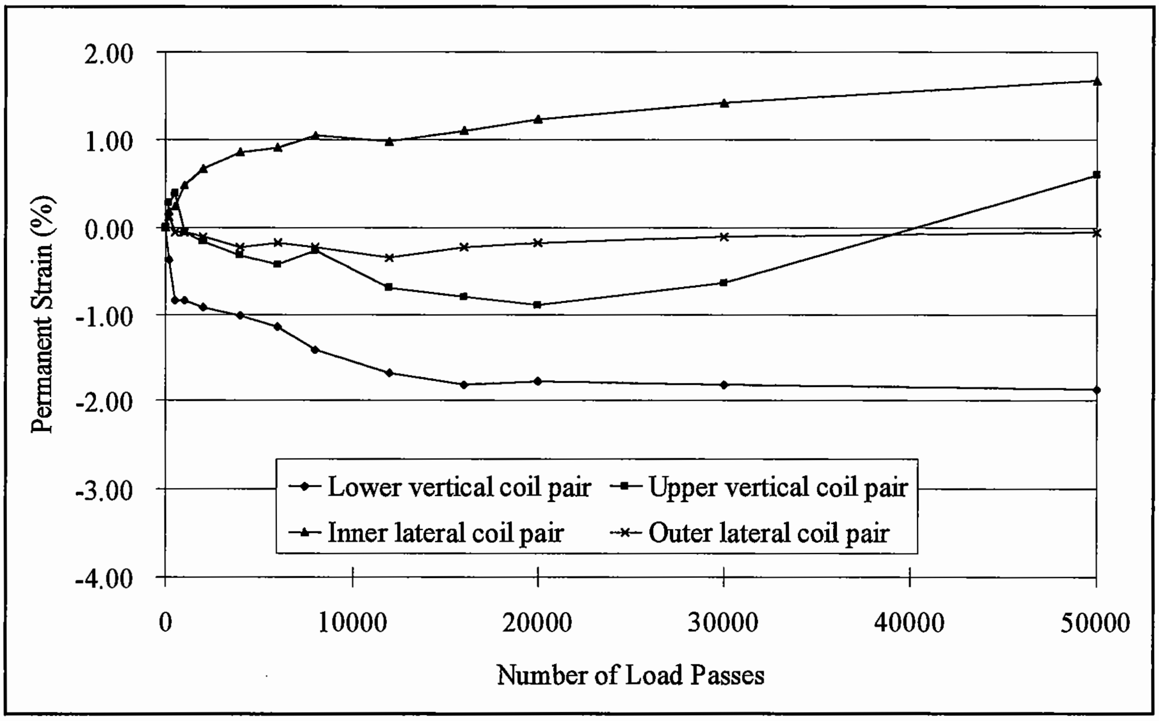


Figure B10 Development and Distribution of Strain within the Bituminous Surfacing Section D, Coil Stack 5

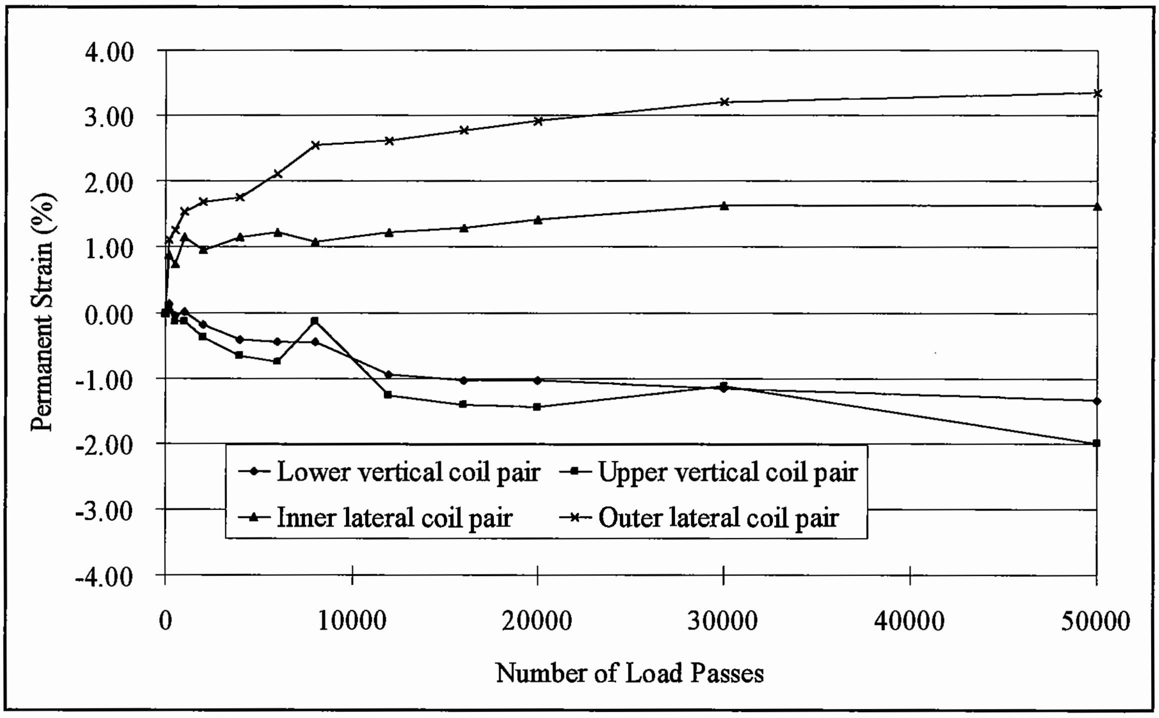


Figure B11 Development and Distribution of Strain within the Bituminous Surfacing Section D, Coil Stack 6

B.3 Digitised Rut Profiles

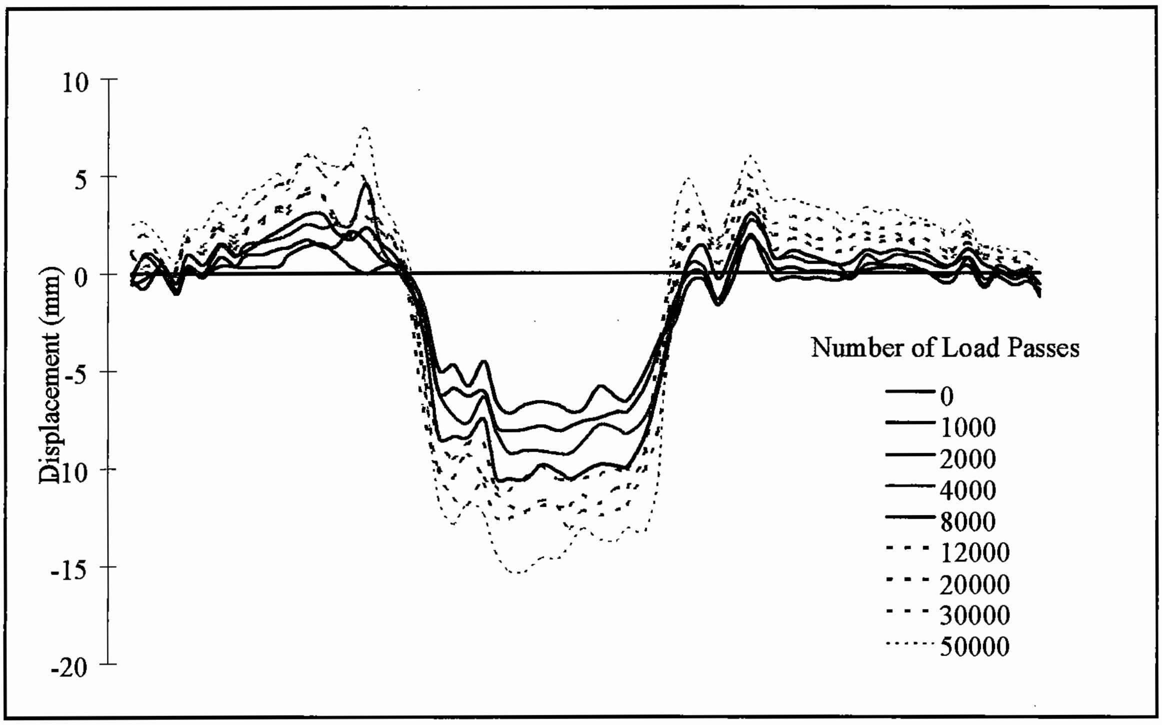


Figure B12 Digitised Rut Profile : Section A, Profile 1

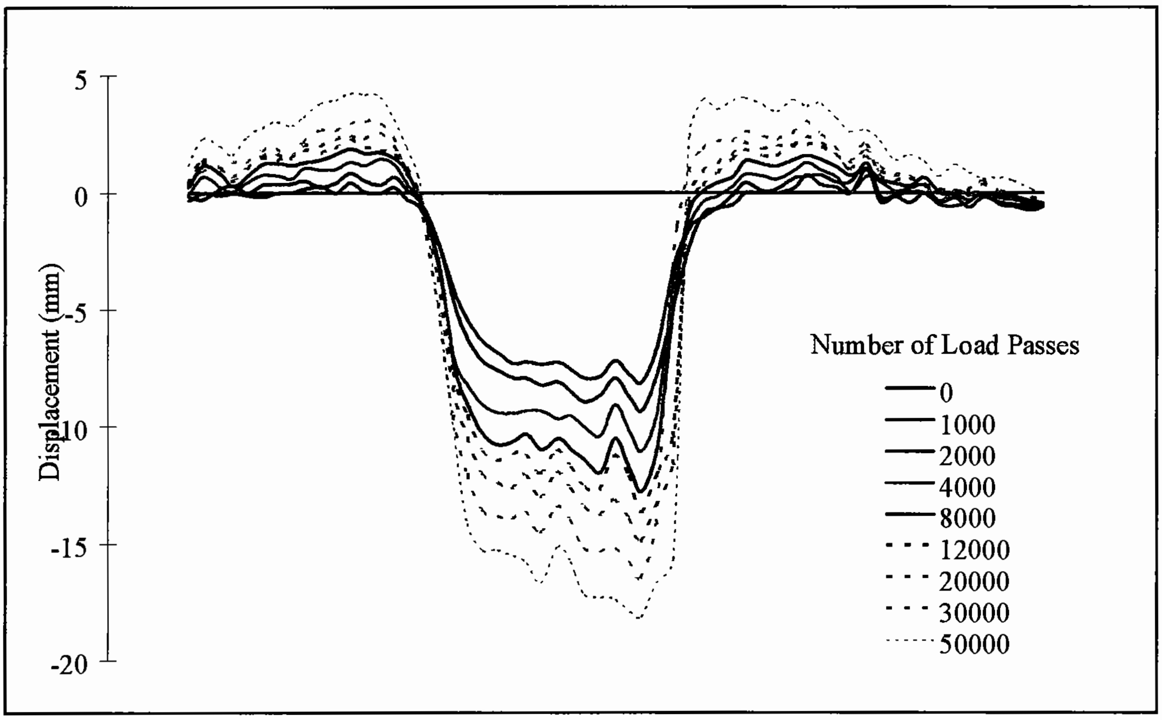


Figure B13 Digitised Rut Profile : Section A, Profile 3

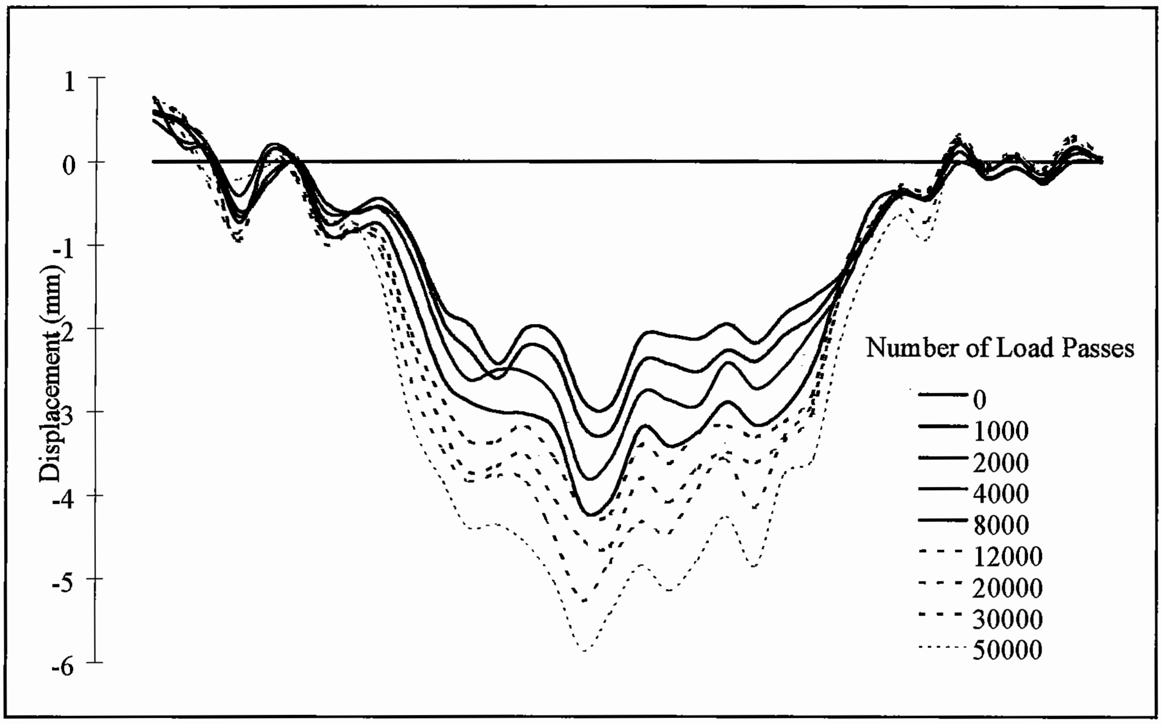


Figure B14 Digitised Rut Profile : Section B, Profile 1

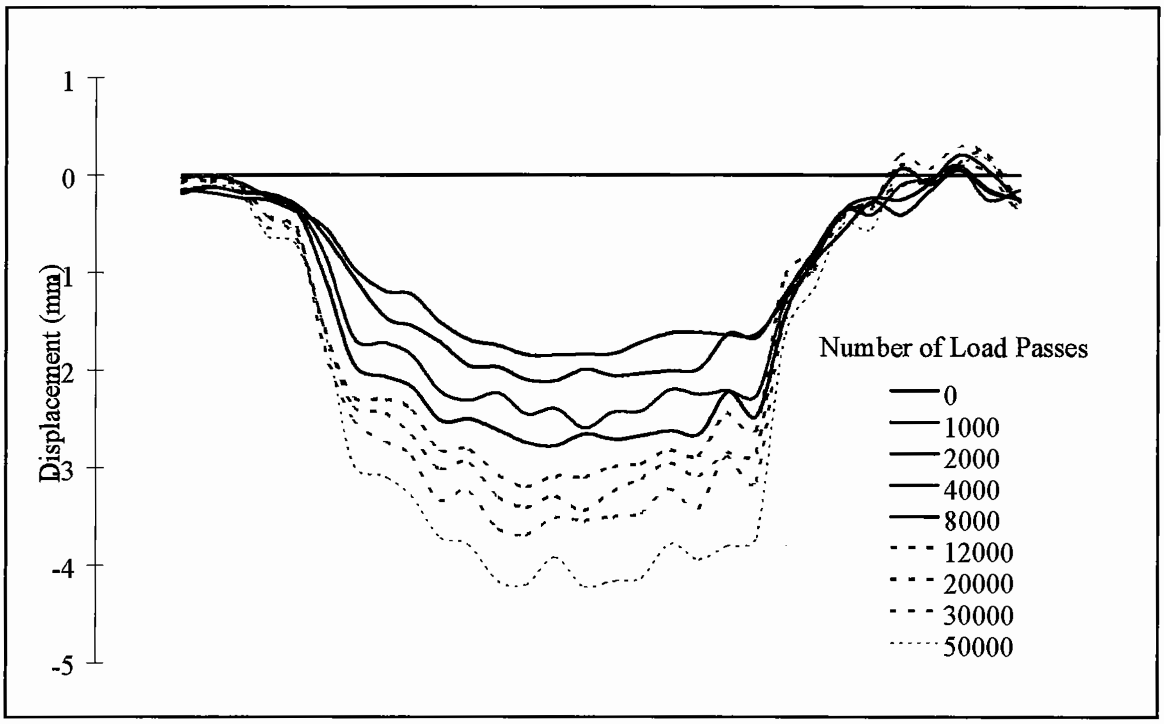


Figure B15 Digitised Rut Profile : Section B, Profile 3

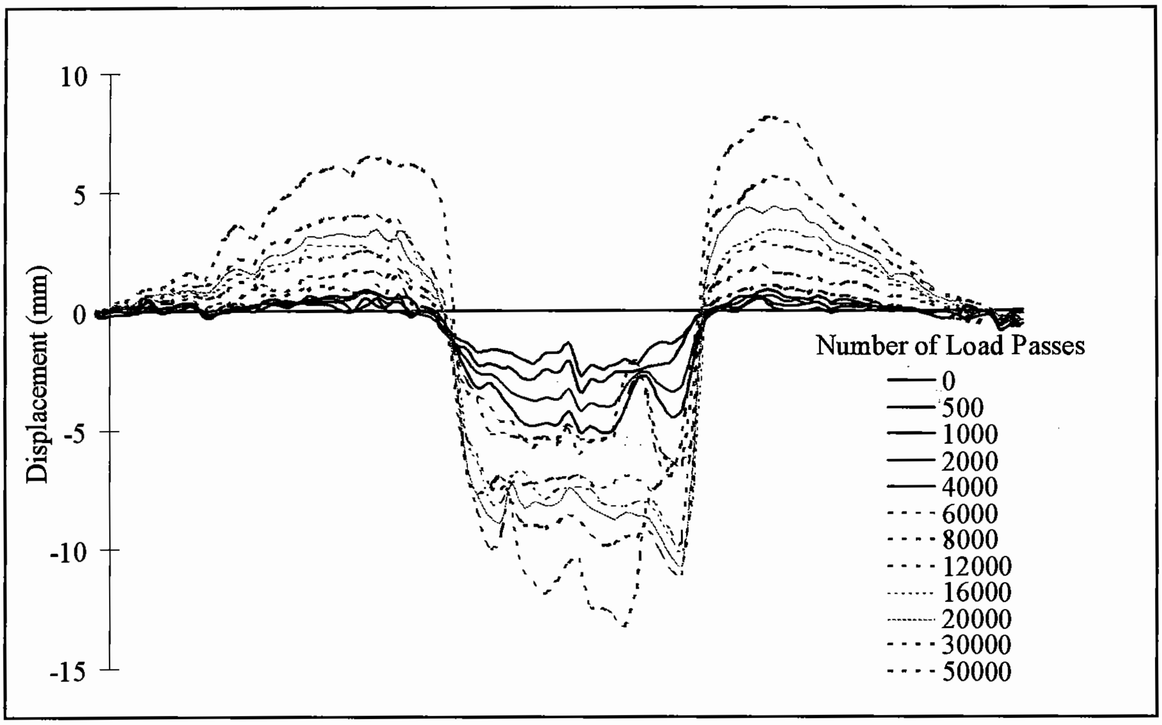


Figure B16 Digitised Rut Profile : Section C, Profile 1

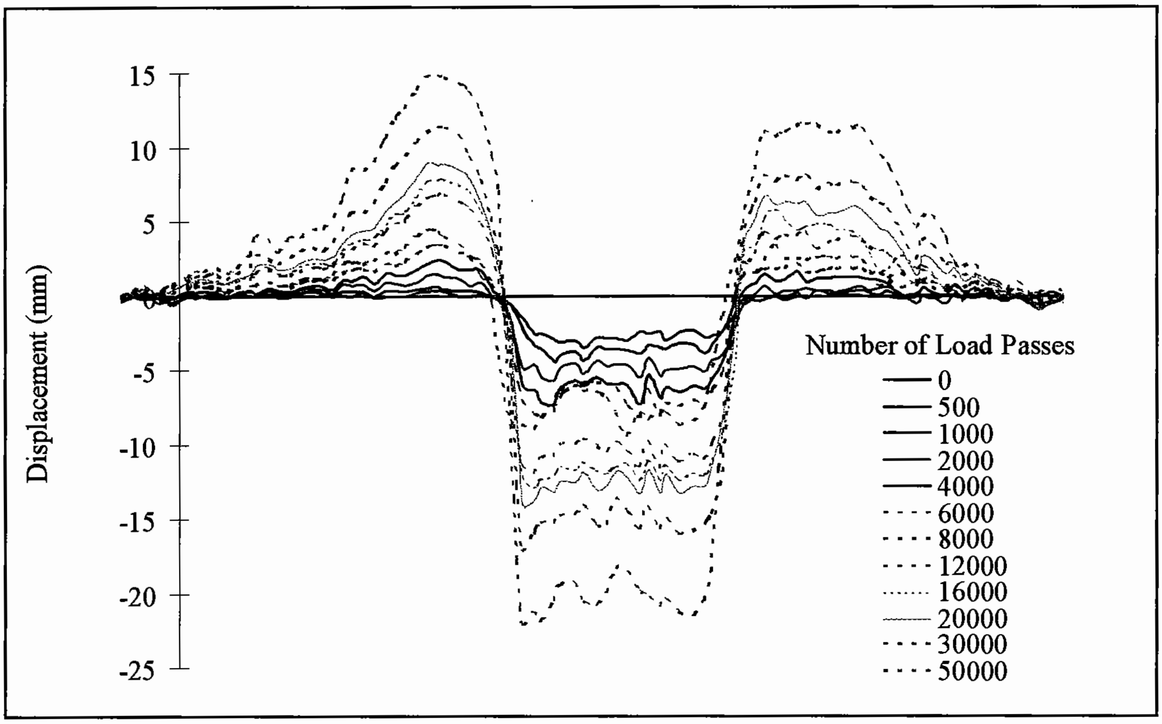


Figure B17 Digitised Rut Profile : Section C, Profile 3

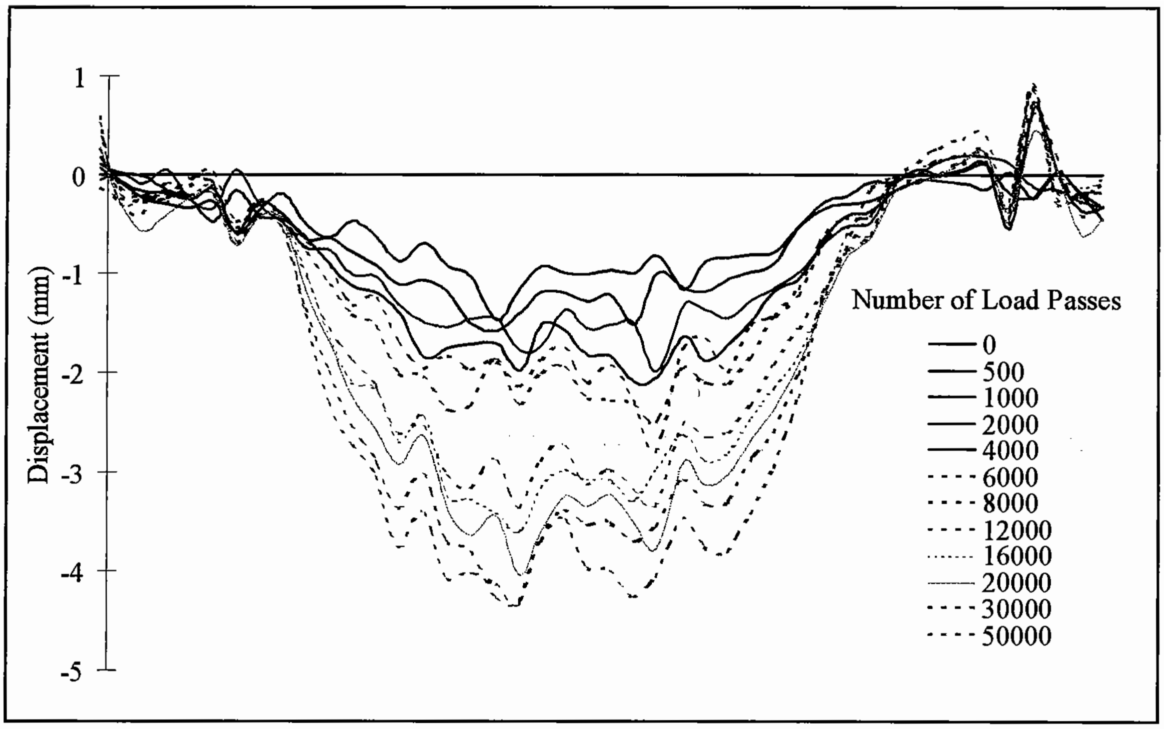


Figure B18 Digitised Rut Profile : Section D, Profile 1

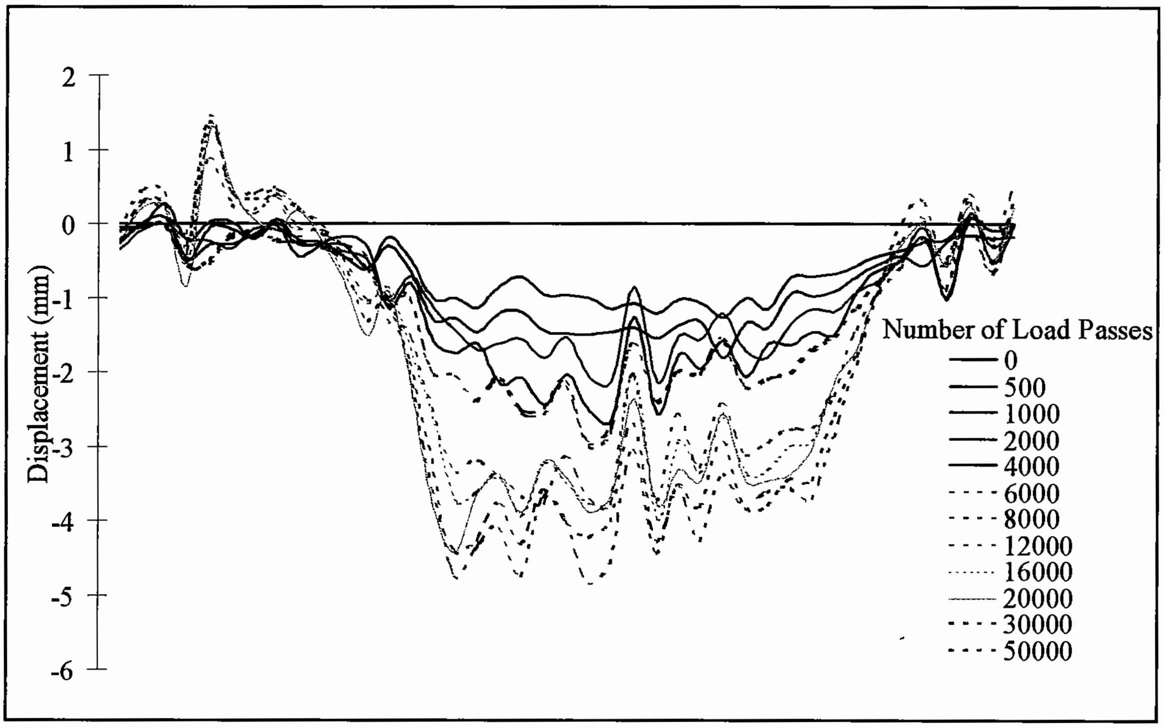


Figure B19 Digitised Rut Profile : Section D, Profile 2

APPENDIX C

TRIAXIAL LOADING FACILITY

C.1 Introduction

The triaxial equipment used in the experiment on "artificial" mixtures described in Chapter Five is a servo-hydraulic loading facility developed and used primarily for the testing of 75mm diameter soil samples. The equipment, which has evolved through a succession of research projects at Nottingham University, has most recently been described by Raybould (155) who implemented a digital data acquisition system.

The basic elements of the system are shown diagrammatically in figure C.1.

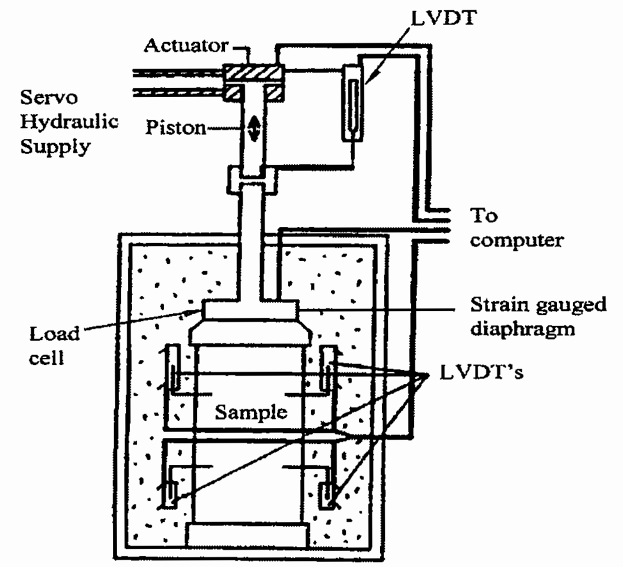


Figure C.1 Triaxial Loading and Axial Deformation Measurement System
(reproduced from reference 155)

C.2 Loading System

The only development to the loading system made during this project was the upgrading of the system control to IBM PC compatible format through the installation of new software obtained from the system manufacturer.

Due to the differences in response between the bound and unbound specimens, and even the different aggregates and grades of binder, dummy specimens were required for each material for the purposes of gain setting prior to testing. Monitoring of the load signal during testing showed the form of the square wave used in the repeated load testing to be excellent.

The cell pressure, which was maintained constant throughout each test, was applied using compressed air.

C.3 Data Acquisition

The only data signals recorded were the output from the strain-gauged load cell and the axial deformation LVDT's. The system of using four LVDT's to record axial deformation "on-sample", as shown in Figure C.1, was replaced with a simpler system using only two LVDT's to monitor deformation "off-sample". The principal reasons for this are given below.

i) The set-up of the test specimens was much simpler and quicker. This was an important consideration given the number of specimens tested.

ii) There was no satisfactory method for attaching the pips necessary for "on-sample" measurement to the single-sized materials in a manner which would accommodate large strains and be suitable for both bound and unbound specimens.

iii) The "off-sample" arrangement was similar to that used in the NAT.

iv) The "off-sample" arrangement provides a degree of mechanical averaging of the deformation response (113). It was found that "on-sample" measurements made using the strain collar on the NAT (Chapter Six) gave an irregular deformation vs load cycles plot when testing at temperatures below 30°C (as in this test programme) whereas as "off-sample" measurements made simultaneously on the same specimen gave a much smoother response.

v) Use of only two LVDT's significantly reduced the amount of data to be collected and manipulated.

The deformation measurement system is shown in Figures C.2 and C.3.

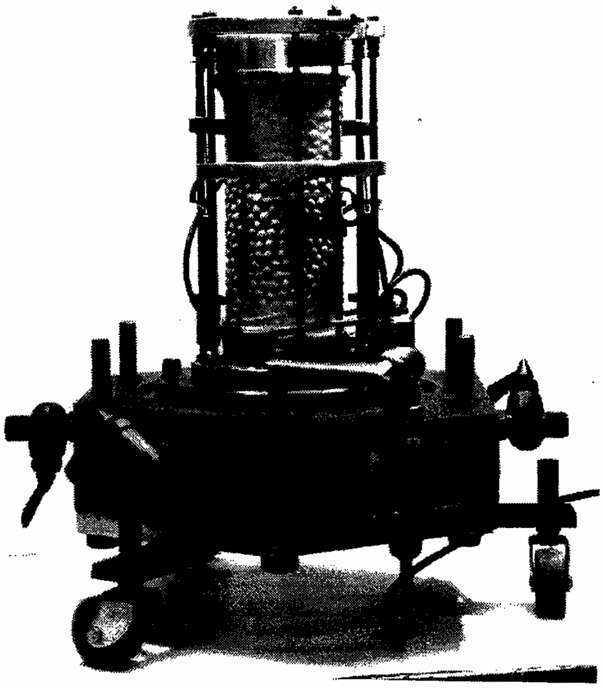


Figure C.2 Unbound Denstone Specimen in Preparation for Testing

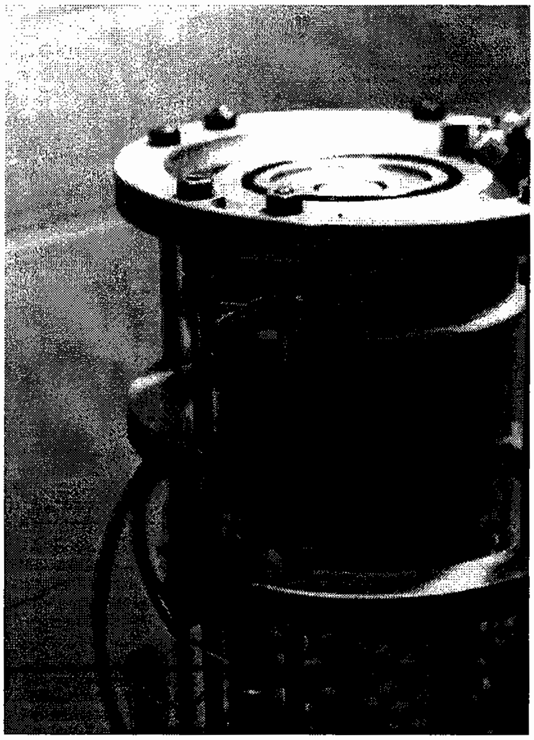


Figure C.3 Detail of Axial Deformation Measurement System : LVDT Core Attached to Upper Loading Platen

APPENDIX D

GUIDELINES FOR INVESTIGATION OF THE EFFECT OF COMPACTION TECHNIQUE ON MECHANICAL PROPERTIES

(Issued to industrial sponsors in July 1992)

Effect of Compaction Technique on Mechanical Properties

The facility to prepare samples of material in the laboratory which have properties representative of the same material in situ is desirable for the purposes of mixture design, but is essential if performance tests are ultimately to be carried out on material sampled prior to laying. The purpose of this investigation is, in the first instance, to assess the suitability of the PRD apparatus for specimen manufacture by obtaining data which will enable a direct comparison of the mechanical properties of PRD and site compacted material. Where possible the investigation could be extended to include other means of specimen preparation, including gyratory when the new apparatus is available.

The following guidelines have been drawn up with the aims of ensuring that adequate data is obtained from each individual investigation and that a consistent approach is adopted by all parties involved.

1. Sufficient loose material for the manufacture of 9 PRD specimens should be obtained (this will normally require approximately 40 kg of material). Three specimens should then be made at each of three target levels of PRD, using the guidance given in the Mix Design Manual, chosen to straddle the typical value for the site compacted material.

2. Ideally samples should be taken from the augers of the paver, with the location of the sample, and extent of the load from which it came, to be recorded and physically marked on site. Where possible the sample should be taken in conjunction with sampling for compositional analysis.

3. If it is not possible to manufacture PRD specimens on site, then the material could be sampled from the wagon at the quarry. When the load is placed on site, its precise location should be recorded and physically marked.

4. When the compacted mat has cooled, a minimum of three cores should be taken at the location of sampling for PRD specimens.

5. Where circumstances permit, larger samples could be obtained direct from the batch plant or hot bin at the quarry for assessment of other compaction techniques to be carried out by the University of Nottingham.

6. All specimens should be prepared and tested in accordance with the protocol already issued. Where possible, coring should be carried out to permit RLIT testing of the cores within 7 days of the material being laid.

7. It is envisaged that this work will normally be carried out on dense roadbase or basecourse mixtures produced either in accordance with a standard recipe or to a mixture design. PRD compaction is unlikely to be suitable for rolled asphalt wearing course, though a similar exercise could be undertaken to assess the suitability of Marshall compaction for this material.

8. All sampling should be in accordance with BS 598 : Part 100 : 1987.