The University of Nottingham

SCHOOL OF CIVIL ENGINEERING

The Role of Bitumen and Bitumen/Filler Mortar in Bituminous Mixture Fatigue

By

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To my parents,

To Amani

ABSTRACT

Fatigue of bituminous mixtures is one of the main mechanisms of flexible pavement failure. This causes pavement failure and safety problems, hence a substantial maintenance cost. The results of many research works have shown that cracking of an asphalt mixture does not occur through the aggregate, but rather within the bitumen and bitumen-fines fractions. However, the relationship between cracking of bitumen and cracking of an asphalt mixture is not well understood, and there is no standard testing methodology to study the behaviour of binder fatigue and its properties. The purpose of this research was to investigate the role of bitumen and bitumen-filler mortar on asphalt mixture fatigue.

This thesis presents extensive sets of fracture and fatigue test data obtained from the Direct Tension Test (DTT) and a new specially designed binder fatigue apparatus. The DTT tests were carried out in the temperature range –10 to +10°C and over a range of strain rates 5.9%/min to 147.9%/min. In these tests, a single straight run binder has been used, and in order to investigate the effect of filler content as well as filler type, tests have also been carried out on a wide range of binder/filler mortars. Data from a series of fatigue tests on bitumen and bitumen/filler mortar are also presented. These tests enabled a study of the behaviour of bitumen and bitumen/filler mortar cracking under repeated load. Two test variations using flat and hemi-spherically shaped end 'platens', were designed and used for fatigue testing of bitumen and bitumen/filler mortar. All the fatigue tests were carried out at a temperature of +10°C and a frequency of 10Hz. Further fatigue tests have also been carried out on realistic asphalt mixture employing a 4-point bend test and Indirect Tensile Fatigue Test (ITFT) using the same binder, filler and bitumen/filler combinations.

The stress at failure of most of the mortars tested in the DTT was about 3MPa for low filler concentration and up to 9MPa for high filler concentration. Time-temperature superposition was used to produce master curves. The results have been evaluated in the context of the DTT derived master curves. It is seen that the DTT strength relating to brittle fracture corresponds well in every case to the stress necessary to induce failure at a single load application, projected from fatigue data. Fatigue life was increased with inclusion of increasing quantities of filler particles. If the bitumen/filler mortar fatigue is determined, it is a performance indicator of the

effect of binder (including filler) on mixture fatigue for a given mixture. It was concluded that the new binder and mortar fatigue equipment worked successfully, and the results correlate well with the mixture fatigue data. The testing methods developed in this study could be considered suitable and useful tools to study the fatigue behaviour of any bitumen and mortar.

A tentative deduction from this study is that pure bitumen and mortar specimen may possibly reach their fracture point without propagation of a crack, as a result of progressive weakening of the intermolecular bond or cohesion structure of the mortar. Mixture failure might be explained by postulating that micro-cracks tend to form immediately within the bitumen/fines mortar at particle contacts where the strains are high, in areas of high stress concentration. However, the crack cannot propagate far before it reaches a zone of much reduced strain, after which the cracks propagate around the particles. The results show that mixture fatigue is primarily strain dependent whereas mortar fatigue is stress dependent. It was also found that the strength ratio of mortar to binder is approximately similar to that of mixture to mortar, on a stress basis.

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All praise and thanks are due to 'Allah'. Almighty who sustained me throughout and enabled me to pursue this study.

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Declaration

The research described in this thesis was conducted at the University of Nottingham, Department of Civil Engineering, NCPE, between October 2001 and October 2004. I declare that the work is my own and has not been submitted for a degree at another university.

Sami A. Osman
Nottingham
Oct 2004

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Notation

DTT	Direct Tension Test
BBR	Bending Beam Rheometer
DSR	Dynamic Shear Rheometer
SHRP	Strategic Highway Research Program
NAT	Nottingham Asphalt Tester
DBM	Dense Bitumen Macadam
ITFT	Indirect Tensile Fatigue Test
TDC	Top-Down Cracking
Max	Maximum
Min	Minimum
Ref.	Reference
B/F	Bitumen-filler
PI	Penetration index
SP	Softening point
L_{eff}	Effective length
BS	British Standard
σ	Stress
ρ	Density
ε	Strain
$\dot{\varepsilon}$	Strain rate
δ	Phase angle
G^*	Complex modulus
τ	Shear stress
T	Temperature
ν	Poisson's ratio
P	Penetration
S	Stiffness
W	Energy dissipated
Hz	Hertz
°C	Degree Celsius
θ	Average angle between particle surface and stress direction

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Introduction

1.1 General

Bitumens are very widely used civil engineering materials, and have been used for thousand of years in construction, the properties and behaviour of which still remain a mystery to many of the large number of people involved in their application [1,2]. The complexity of bitumen properties raises questions of considerable interest in understanding its behaviour. They are thermoplastic materials that behave like glass at low temperature, in that they are very elastic and brittle, and like a fluid at very high temperature, in that they posses the ability to flow when subjected to shear loading. At intermediate temperature (e.g. for 50 pen bitumen between –10°C and 50°C), they behave in a viscoelastic manner, possessing both elastic and viscous properties [1]. Thus the study of bitumens' behaviour has become an important aspect and many researchers have studied their rheology.

Bituminous mixtures are mixtures of bitumen, mineral aggregate with or without filler. In-situ, bituminous layers distribute traffic stresses both through the aggregate skeleton and through the fine aggregate/sand/filler-bitumen mortar. Therefore, to resist deformation under the imposed stresses, the mortar must have high stiffness. The use of filler in the bituminous mixture increases the stiffness of the mixture [3].

Fatigue of the bituminous mixture is one of the main failure mechanisms of flexible pavements, and as a consequence, the assessment of fatigue behaviour is the basis of pavement design methodology used in many countries.

Fatigue cracking of a bituminous layer arises from repeated tensile strains due to traffic loading, which are found at the bottom of the bituminous layer [1]. When the crack is initiated at the bottom of the layer, it propagates upwards, causing gradual weakening of the structure [4, 5]. Recently, some researchers [6] have suggested that cracks in flexible pavements can also be initiated at the top surface, and propagate downwards to the bottom of the bound layers, particularly in thick pavements.

Research [7,8,9] has revealed that cracks in an asphalt mixture tend to run through the binder. In practice, the binder consists of a mixture of bitumen and fine aggregate or filler. Figure 1.1 shows a picture of specimens that had failed in an indirect tensile fatigue test. It illustrates that cracks run almost entirely through the binder. It is generally found that failure does not take place at the interface between bitumen and aggregate particles, but in the bitumen/fines film (cohesion failure). Figure 1.2 shows the typical advance of the crack front around an aggregate particle [9]. Molenaar [10] reported that there is general agreement that aggregate properties played the central role in overcoming permanent deformation, while fatigue cracking and low temperature cracking were less affected by aggregate characteristics.

1.2 Need for the Research

It is necessary to determine the fracture properties for pure bitumen as well as for bituminous mixtures. For many years, fatigue of bituminous mixtures has been the subject of investigation. Although it is recognised that fatigue damage is mainly caused by cracking or damage within the bitumen binder, very few studies have used binder testing to evaluate fatigue of the binder for example using the dynamic shear rheometer (DSR) [11,12].

A review of literature revealed that there was a significant lack of information about the role of the binder composition or rheological properties of binder in fatigue damage. Some progress has been made towards understanding the role of binder in fatigue cracking [7]. However, there have been few published reports and papers directly addressing the problem of fatigue properties of binder. The reason for this is the lack of a suitable fatigue test for the bitumen binder. Therefore, it is desirable to investigate the fatigue properties for pure bitumen and the bitumen-filler mortar.

1.3 Objectives of the Research

Taking into consideration the problems mentioned above, the main objectives of this research were to investigate the tensile and fatigue characteristics of pure bitumen and bitumen-filler mortar, and to advance the insight into their contribution to the fatigue of bituminous mixture.

1.4 Scope of the Research

In order to meet the above objectives the following research methodology was adopted. A general literature review covered bituminous materials, fatigue characteristics of bitumen, bituminous mixtures and the research done on fatigue for other related materials. A thorough study was carried out on filler characteristics and their effect on the behaviour of bitumen and pavement performance.

A testing programme was initially planned to cover pure bitumen and bitumen/filler mortars using different types of bitumen and different fillers in different combinations and temperatures. Changes were introduced to the original programme when Nynas Bitumen joined the project to sponsor the research, reflecting a wish to include asphalt mixture fatigue using the four-point bend test. Accordingly changes were made, and a large experimental programme has been undertaken. The experimental program was divided into three main phases:

- Phase one: using the direct tension test (DTT) for testing pure bitumen and bitumen/filler mortar.
- Phase two: developing a new simple fatigue test apparatus for testing bitumen and bitumen-filler mortar.

- Phase three: Carrying out fatigue tests for bituminous mixtures using the same bitumen/filler combinations used in phase 2.

The Direct Tension Test (DTT) INSTRON machine was chosen to study the tensile and fracture characteristics of bitumen and bitumen-filler mortar. The DTT machine is capable of testing over a wide range of temperature, from -40°C to +25°C, and a wide range of strain rates, between 0.001 to 300mm/min. The strain rates were chosen to cover a wide range: 2mm/min, 5mm/min, 10mm/min, 20mm/min and 50mm/min (5.9, 14.8, 29.6, 59.2 and 159.9 %/min respectively). A few specimens were tested at 200 mm/min. Because of the behaviour of the bitumen, since it tends to flow at higher temperature, and the difficulties of preparation of the specimens, the chosen temperatures used to perform the tests were: -10°C, -5°C, 0°C, +5°C and +10°C, although the DTT test was actually developed for low temperature fracture (sub-zero temperature).

50 pen bitumen supplied by Nynas was used in the investigation. Four types of fillers: limestone, cement, gritstone, and sewage sludge ash, were used in the research. The DTT was carried out on pure bitumen and on bitumen/filler mortar. The mortar was bitumen mixed with filler using different filler concentrations. The percentages of limestone added to the bitumen by mass were: 5%, 15%, 35%, 50% and 65%; for cement and gritstone filler, 50% by mass was used, and, for sewage sludge ash, 15%. Currently, there is no standard procedure for manufacturing bitumen-filler mortar; therefore, a mortar preparation procedure was devised in order to reduce variability in test results.

To study the fatigue characteristics of the bitumen and bitumen-filler mortars, a special test rig for fatigue determination has been designed, and manufactured in the workshop of the School of Civil Engineering. The new test device had to be fitted onto a uniaxial testing machine, which had a load cell capacity of 1000N and was capable of applying repeated loading to simulate the traffic loading on a pavement. To simulate two particles of aggregate bonded together with a binder, two tests were developed. The first consisted of two cylindrically shaped end blocks (flat ends) to hold a thin film of bitumen in between. The second had two hemi-spherically shaped end blocks (rounded ends). The test equipment was located in a room with a controlled temperature range from -5°C to +70°C.

The fatigue tests were performed using bitumen and bitumen/filler mortars having different filler concentrations and tested at a temperature of +10°C and different frequencies, however most of the tests were carried out at 10Hz. Percentages of 5, 15, 35 and 65% by mass of limestone filler were added to the bitumen to form the mortars. The gritstone and cement fillers were added at a concentration of 35% by mass only, whereas sewage sludge ash was added at 15% by mass.

The mixture fatigue tests were carried out using two types of tests, the 4-point bend test and the Indirect Tensile Fatigue Test (ITFT). Different mixtures were manufactured in a slab of dimensions 305×305×50mm and sawn into beams (305×50×50mm) for the 4-point bend test and cored (100mm diameter × 40mm thickness) for the ITFT. The tests were carried out at a temperature of +10°C and a frequency of 10Hz. Mixtures containing 15%, 35% and 65% by mass of limestone filler were tested in the 4-point bend test, as well as mixtures containing 35% by mass girtstone and cement fillers. The ITFT tests were carried out on the mixtures containing only 35% by mass limestone, cement and gritstone fillers.

To establish a clear picture of the factors affecting pavement fatigue at every stage from bitumen to mixture, comparison of fatigue properties of bitumen and bitumen-filler mortar was made and then those of the mortar and the bituminous mixture. Time-temperature superposition was used to produce master curves to be used for correlation between the DTT and the fatigue test, discussion and evaluation of the results were presented.

1.5 Thesis Structure

This thesis includes a theoretical review, methodology, results and analysis from the laboratory investigation. The thesis is organised into eight chapters followed by references and an appendix. Chapter one is a general introduction and gives the background of the research, the need for the research and outlines the scope of the research work, with brief descriptions of the chapters. Chapter two reviews the literature to date concerning bitumen, filler materials and bituminous mixtures. It also provides a brief description of the tests methods used to characterise the bitumen and filler properties and also gives details of asphalt mixtures and their mechanical properties and testing.

Chapter three reviews the concept and phenomenon of fatigue as related to bitumen and asphalt mixtures, and describes the current methods used in characterising the fatigue of bitumen as well as bitumen-filler mortar and explains mixture fatigue testing. Chapter four describes the testing carried out using the direct tension test (DTT) for bitumen and bitumen-filler mortar. It gives details of: the equipment, materials used, test conditions and discusses some of the results. Chapter five describes the development of the new fatigue test apparatus for bitumen and bitumen filler mortar. It provides details of the testing apparatus, testing programme, materials, test results and discussion.

Chapter six addresses asphalt mixture fatigue. It describes the mixture design, the test equipment, and presents the results with discussion. Chapter seven describes the laboratory results in general with detailed analysis, evaluation and discussion. Conclusions and recommendations for future work are presented in chapter eight.

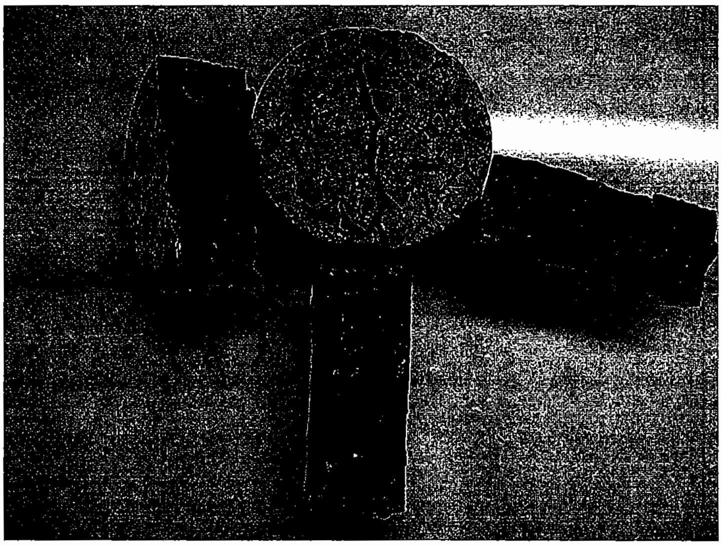


Figure 1.1 Fatigue cracks run mainly through the bitumen/fines mortar

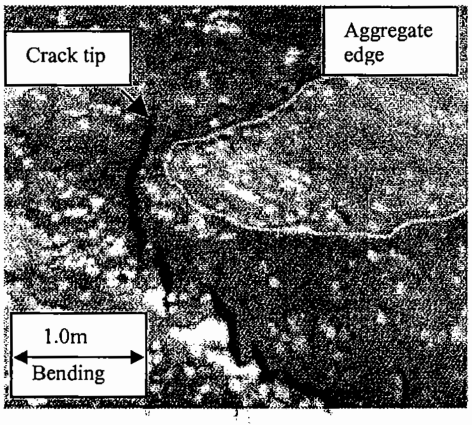


Figure 1.2 Typical advance of crack front around an aggregate particle [9]

2

Review of Material Properties

2.1 General

This chapter is a review of research published to date on bituminous mixtures and the constituent materials which are used in its manufacture, with concentration on bituminous binder and filler.

Bituminous materials have been in known use for thousands of years, when bitumen mastic was used in Mesopotamia as water proofing for reservoirs [1]. Many years before crude oil exploration and its industrial processing began, man had recognised the numerous advantages of bitumen application and had started the production of bituminous material found in natural deposits, using available methods [13].

As reported in the Shell Bitumen Handbook [1]:

"It is widely believed that the term bitumen originated in Sanskrit, where the word "jatu" meaning pitch and "jatu-kirt" meaning pitch creating referred to the pitch produced by some resinous trees. The Latin equivalent is claimed by some to be originally "gwitu-men" (pertaining to pitch) and by others, pixtu-men (bubbling pitch), which was subsequently shortened to bitumen then passing via French to English".

In this research the word bitumen refers to the petroleum product and the word asphalt to a mixture of aggregate and bitumen. There are a very large number of different bituminous materials which may be used in various circumstances. All bituminous mixtures consist of three components aggregate (with or without filler), binder and air.

2.2 Review of Bituminous Binder

Bitumen is manufactured from crude oil. It is generally agreed that crude oil originated from the remains of marine organisms and vegetable matter deposited with mud and fragments of rock on the ocean bed. Over millions of years, organic materials and mud accumulated into layers hundreds of meters thick, the immense weight of the upper layers compressing the lower layers into sedimentary rock. Conversion of organisms and vegetable matter into the hydrocarbons of crude oil is thought to be the result of the application of heat form within the earth's crust, pressure applied by the upper layers of sediments, possibly aided by the effect of bacterial action and radio-active bombardment [1,14].

Petroleum bitumens are supplied in a number of different forms for use for road purposes:

1- Penetration bitumens.
2- Cutback bitumens.
3- Bitumen emulsions.
4- Modified bitumens.

BS 3690-1:1989 [15] defines bitumen as "a viscous liquid, or solid, consisting essentially of hydrocarbons and their derivatives, which is soluble in trichloroethylene and is substantially non-volatile and softens gradually when heated. It is black or brown in colour and possesses water proofing and adhesive properties.

It is obtained by refinery processes from petroleum, and is also found as a natural deposit or as a component of naturally occurring asphalt, in which it is associated with mineral matter".

2.2.1 The Nature of Bitumen

Bitumen is perhaps best described as a complex mixture of components with various chemical structures. The majority of these structures are composed of carbon and hydrogen only and are termed hydrocarbons. In addition to hydrocarbons there are a number of other structures containing heteroatoms, i.e. atoms other than hydrogen and carbon, such as oxygen, sulphur, and nitrogen. It is the complex arrangement of the hydrocarbon molecules and those molecules containing heteroatoms which gives bitumen its unique balance of properties [16]. The hydrocarbon and hetroatoms structure subdivides into distinct chemical groups. The individual compounds are then classified as either saturates, aromatics, resins or asphaltenes [17].

According to the Shell Bitumen Handbook [1], the elementary analysis of bitumens manufactured from a variety of crude oils shows that most bitumens contain:

Carbon	82 – 88%
Hydrogen	8 – 11%
Sulphur	0 – 6%
Oxygen	0 – 1.5%
Nitrogen	0 – 1%

But the precise composition varies according to the source of the crude oil from which the bitumen originates, and ageing in service.

2.2.1.1 Saturates

These are straight and branched chain molecules consisting of carbon and hydrogen only [16]. They are termed saturates because they contain almost exclusively single carbon-carbon or carbon-hydrogen bonds although there may be some aromatic and naphthenic ring structure present.

Since they are composed of only carbon and hydrogen, the saturates are non-polar and show no great affinity for each other. When extracted from bitumen, the saturates appear as a viscous, white to straw coloured liquid with a molecular weight in the range 300-2000. The saturates constitute between 5% and 20% of the total bitumen structure [1,16].

2.2.1.2 Aromatics

These are low molecular weight compounds comprising ring and chain structures and form the major part of the dispersing medium for the asphaltenes. Unsaturated ring structures predominate in the overall aromatic structure which can contribute up to 65% of the total bitumen structure. When purified, the aromatics appear as viscous brown liquids with a molecular weight in the range 300-2000 [1,16].

2.2.1.3 Resins

The resins are highly polar, predominantly hydrocarbon molecules with a significantly higher concentration of heteroatoms than the other species which are present in bitumen. They have been found to contain both acidic and basic groups which means that possibilities exist for hydrogen bonding and strong inter-molecular and intra-molecular attraction. When purified, the resins appear as black or dark brown solids or semi-solids with molecular weight in the range 500-50000, a particle size of 1 nm to 5 nm and hydrogen/carbon atomic ratio of 1.3 to 1.4 [1]. The polar nature of the resins gives the bitumen its adhesive properties [1,16].

2.2.1.4 Asphaltenes

Asphaltenes are arguably the species with the highest molecular weights within the bitumen structure. They are generally described as being very polar molecules containing a high concentration of aromatic ring structures. When purified, the asphaltenes appear as solid black or brown particles with a gritty or brittle feel [1,16]. Depending on the method of purification and analysis, the asphaltenes have been found to have molecular weights ranging between 600-300,000. However, the majority of data suggests that the predominant species in the asphaltenes structure have molecular weight in the range 1000-100,000. The asphaltenes, which may range between 5% and 25%, have an enormous effect on the overall properties of the bitumen. Bitumens with a high asphaltenes content will have higher softening points, higher viscosities and lower penetrations than those with low asphaltene contents [1]. The presence of heteroatoms and polar groups within the asphaltenes structure means that there is a very high possibility for hydrogen bonding. Consequently, it is not only the asphaltenes content that influences the structure of the bitumen but also the degree of interaction and bonding between the asphaltene molecules themselves [16,18].

The asphaltenes form tightly packed structures or clusters within a matrix of aromatics and saturates. The resins act as stabilizers to prevent the asphaltenes from separating. The extent of dispersion of the asphaltenes within the matrix will determine the overall properties of the bitumen.

2.2.2 Production of Bitumen

Although there are a number of different crude oils currently available, only a small percentage of these are directly suitable for manufacture of bitumen. It has been estimated that the total number of crude oils available at present exceeds 1500, with less than 100 of these being suitable for bitumen production [1,16]. The reason for this is the widely varying chemical composition and bitumen content of these crudes.

The four main oil producing areas in the world are the U.S.A., the Middle East, the countries around the Caribbean and Russia. Crude oils differ both as to their physical and chemical properties [1].

2.2.3 Mechanical Testing and Properties of Bitumen

2.2.3.1 Specification Tests for Bitumen

As an almost infinite variety of bitumens could be manufactured, it is necessary to have tests which can characterise different grades. Bitumen grades are specified primarily in terms of their penetration (referred to as 'pen'). Soft bitumens have high pen numbers and hard bitumens have low pen numbers.

British Standard Specifications

The British Standard specification for bitumens used in road construction is given in BS 3690:part 1 [15]. Table 2.1 shows the specifications for bitumen as defined in BS 3690-1:1989 for 50 pen bitumen. The grades of bitumen most commonly used in the UK until recently for road construction were 50, 100, and 200 pen but the stiffer grades 15, 25, and 35 pen are starting to be used more [1,19]. Recent research into the performance of pavements has concluded that the stiffness of the roadbase has a significant effect on the performance of the pavement. As a result, there has been a tendency to move away from the use of 50 or 100 pen bitumen to 35 pen bitumen in roadbases.

Table 2.1 Properties for 50 pen bitumen according to BS 3690-1:1989

Property	Test Method	Technically identical with	Grade 50 pen
Penetration at 25°C (dmm)	BS 2000-49	ASTM D5 – 86 IP 49	50±10
Softening point (°C)	BS 2000-58	IP 58	45 - 58
Loss on heating 5h at 163°C a) Loss by mass (%) (max) b) Drop in penetration (%) (max)	BS 2000-45	IP 45	 0.2 20
Solubility in trichloroethylene (%) by mass	BS 2000-47	IP 47	99.5

European Specifications

The specification of bitumen has been reviewed as part of the European harmonisation process. This is the responsibility of the Comite Europeen de Normalisation (CEN). The standardisation and characterisation of bituminous binders has been prepared by the Technical Committee (TC) for bituminous binders dealing with oil products (TC19). Table 2.2 shows the specifications for bitumen as defined in BS EN 12591:2000 [20] for 40/60 pen bitumen.

There are several differences between the CEN specifications for bitumen and those described in BS 3690: part 1. The most notable is the change in the definition of grades. Whereas BS 3690: part 1 denotes each grade by the mid-point of the penetration specification (e.g. 50 pen), the CEN grades are specified by their limits (e.g. 40/60pen). The grades themselves are also different. The UK grade of 15pen is removed from the proposed specification completely with the hardest grade being 20/30. In addition, 100pen is replaced by two grades: 70/100 and 100/150.

Table 2.2 Specifications for grade 40/60 pen bitumen according to BS EN 2591:2000

Property	**Unit**	**Test method**	**Grade 40/60**
Penetration at 25°C	×0.1 mm	EN 1426	40-60
Softening point	°C	EN 1427	48-56
Resistance to hardening, at 163°C		EN 12607 –1 or	
a) Change of mass, (max), ±	%	EN 12607 – 3	0.5
b) Retained penetration, (min)	%		50
c) Softening point after hardening, (min)	°C	EN 1427	49
Flash point, (min.)	°C	EN 22592	230
Solubility, (min.)	°C	EN 12592	99.0

The SHRP/SUPERPAVE Bitumen Specifications

The Strategic Highway Research Program (SHRP) [14], initiated in the USA in 1987, was a coordinated effort to produce rational specifications for bitumens and asphalt based on performance parameters. The motivation was to produce pavements which performed well in service. These pavements were subsequently called 'SUPERPAVE' (SUperior PERforming PAVments).

One of the results of this work was the 'Superpave asphalt binder specification' which categorizes grade of bitumen according to performance characteristics in different environmental conditions. The specification was intended to limit the potential of a bitumen to contribute to permanent deformation, fatigue failure and low-temperature cracking of asphalt pavements.

2.2.3.2 Penetration Test [BS 2000-49, ASTM D5 – 86, IP 49]

The consistency of bitumen is commonly measured by the penetration test [21,22]. In this test a needle of specified dimensions is allowed to penetrate into a sample of bitumen, under known load (100g), at fixed temperature (25°C), for a known time (5 seconds). The penetration test can be considered as an indirect measurement of the viscosity of the bitumen at a temperature of 25°C.

2.2.3.3 Ring and Ball Softening Point Test [BS 2000-58, IP58]

The softening point test [23,24] is an empirical test used to determine the consistency of a bitumen by measuring the equiviscous temperature at which the consistency of the bitumen is between solid and liquid behaviour. In this test a steel ball (3.5g) is placed on a sample of bitumen contained in a brass ring; this is suspended in a water or glycerol bath. The bath temperature is raised at 5°C per minute, the bitumen softens and eventually deforms slowly with the ball falling through the ring. At the moment the bitumen and steel ball touch a base plate 25mm below the ring, the temperature of the water is recorded.

2.2.3.4 Ductility Test [ASTM D 113]

The ductility of a paving bitumen is measured by the distance to which it will elongate before breaking when two ends of a briquette specimen are pulled apart at a specified speed and temperature. ASTM D113 [25] gives the test procedure to measure ductility at 25°C (77°F) and lower temperatures. The ductility test on bitumen is carried out at a constant rate of elongation which is usually 50mm per minute. Under these conditions the tensile stress on the thread decreases with decreasing cross-section and increasing elongation.

2.2.3.5 Viscosity

Viscosity is a fundamental characteristic of bitumen as it determines how the material will behave at a given temperature and over a temperature range. The basic unit of viscosity is Pascal second (Pa.s). The absolute viscosity of a bitumen is the shear stress applied to a sample of bitumen in Pascals divided by the shear rate per second, and can be measured using a sliding plate viscometer. Viscosity can also be measured in units of m^2/s, or more commonly mm^2/s. These units relate to 'kinematic viscosity'(absolute viscosity divided by density), which can be measured using a capillary tube viscometer. For many purposes it is usual to measure the viscosity of bitumen by measuring the time required for a fixed quantity of material to flow through a standard orifice.

Viscosity at any given temperature and shear rate is essentially the ratio of shear stress to shear strain rate. At high temperatures such as 135°C, bitumen behaves as a simple Newtonian liquid; that is, the ratio of shear stress to shear strain rate is constant. At low temperature, the ratio of shear stress to shear strain rate is not constant, and the bitumen behaves like a non- Newtonian liquid [19].

The Sliding Plate Viscometer

A fundamental method of measuring viscosity is the sliding plate viscometer. This apparatus applies the definition of absolute viscosity, i.e. it takes the shear applied to a film of bitumen (5 to 50 microns thick) sandwiched between two flat plates and measures the resulting rate of strain. The apparatus comprises a loading system which applies a uniform shear stress during a measurement, and a device to produce a record of the flow as a function of time.

Capillary Viscometers

Capillary viscometers are essentially narrow glass tubes through which the bitumen flows. The tube has narrower and wider parts and is provided with two or more marks to indicate a certain volume or flow. The measurement of the kinematic viscosity is made by timing the flow of bitumen through a glass capillary viscometer at a given temperature. Standard ASTM test methods which use capillary viscometers are available to determine bitumen viscosity at 60°C [ASTM D2171] and 135°C [ASTM D2170] on a routine basis.

2.2.3.6 Fraass Breaking Point Test

The brittleness of bitumen plays an important part in many practical applications. The most usual test is the Fraass test [1,26], which can be used to describe the behaviour of bitumen at low temperature. In the Fraass test a steel plaque 41mm×20mm coated with 0.5mm of bitumen is slowly flexed and released. The temperature of the plaque is reduced at 1°C per minute until the bitumen reaches a critical stiffness and cracks. The temperature at which the sample cracks is termed the breaking point and represents an equi-stiffness temperature. It has been shown that at fracture the bitumen has a stiffness of 2.1×10^9Pa which is approaching the maximum stiffness of 2.7×10^9Pa [1].

2.2.4 Determination of High Temperature Properties

Measurement of the high temperature properties of bitumen gives an indication of its ease of handling at a coating plant. SHRP uses a rotational viscometer to measure the viscosity of bitumen at elevated temperature.

ASTM method D4402 [27], describes the use of a Brookfield viscometer and thermosel to measure the viscosity of bitumen at elevated temperature. The SHRP procedure does, however, set various criteria under which the determination should be made. Under SHRP, the determination of viscosity is carried out using a Brookfield viscometer and thermosel at 135°C. In practice, it is desirable to measure the viscosity over a range of temperatures and shear rates so that an indication of binder behaviour during mixing and compaction can be obtained [19].

2.2.5 Determination of Low Temperature Properties

The Fraass Breaking Point Test [26] is used to characterise the low temperature behaviour of bitumen. However, the difficulties in obtaining reliable Fraass data led to the development of the:

a) Bending Beam Rheometer (BBR) [14], in this test a thin beam of bitumen is prepared and subjected to a three-point bending test at low temperature.
b) Direct Tension Test (DTT) [14,28] in this test a dog bone of bitumen is prepared and stretched at a slow constant rate until it fractures. The strain to failure at a particular temperature is then reported.

The BBR and DTT are carried out on materials which have been subjected to ageing in the Pressure Ageing Vessel (PAV) [14].

2.2.5.1 Bending Beam Rheometer Testing (BBR) [AASHTO: TP1]

The bending beam rheometer is used to measure the low temperature creep response of bitumen binder. The main parts of the rheometer (see Figure 2.1) are:

- Testing frame unit,
- Temperature controlled bath, and
- Circulator and data acquisition system using a personal computer.

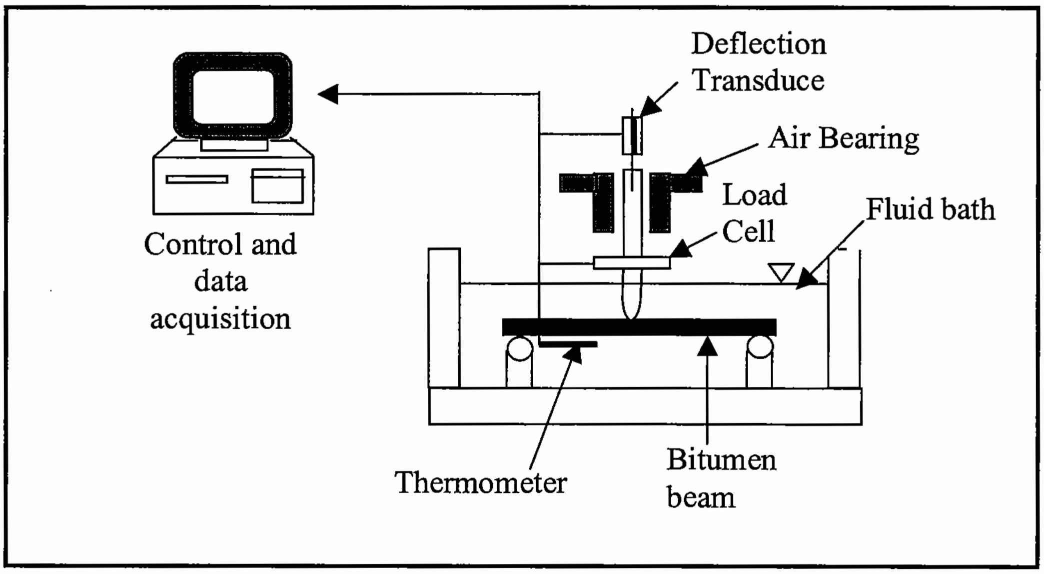


Figure 2.1 Principle of operation of the bending beam rheometer (BBR) [14]

The test apparatus is designed for testing within the temperature range of –40°C to +25°C. The rheometer is operated by applying a constant load at midspan of a small bitumen beam that is simply supported. During the test, the deflection of the centre point of the beam is measured continuously. The beam is 125mm long, 12.5mm wide and 6.25 mm thick (see Figure 2.2). The specimen, the support, and the lower part of the test frame are submereged in a constant temperature fluid bath, which controls the test temperature.

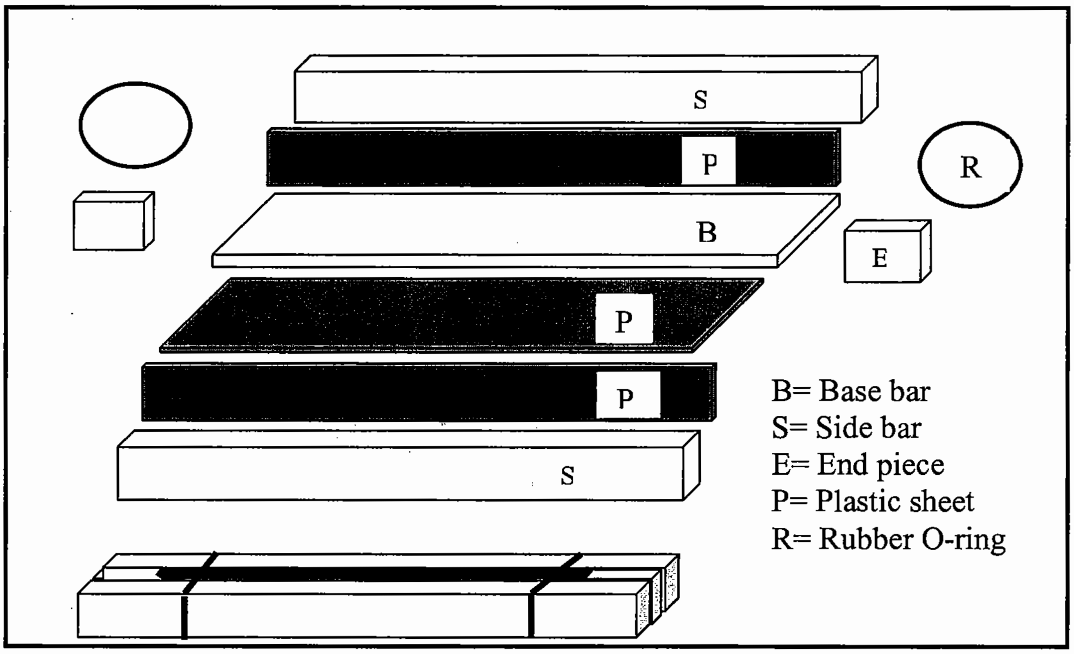


Figure 2.2 BBR specimen mould [14]

The loading unit in the device consists of an air bearing and pneumatic piston that controls the movement of the loading shaft. The loading shaft serves a dual purpose; its upper end is attached to the linear variable differential transformer (LVDT) that precisely measures the shaft movement, and its own weight is used to apply the required load on the test specimen.

The data, which include deflection of the bitumen beam, load on the beam, and time of loading, are collected by a computer through a digital data acquisition system. The collected data are used to calculate the creep stiffness, and the absolute value of the slope of the log stiffness versus log time curve (m-value). The Superpave specification states that the creep stiffness should not exceed 300MPa and the m-value should be ≤ 0.3 [19].

2.2.5.2 Direct Tension Test (DTT) [AASHTO: TP3]

The DTT is similar to the classical determination of ductility but is carried out at much lower temperatures and with smaller samples. A dog bone of material is prepared and stretched at a slow, constant rate until it fractures. The strain to failure at a particular temperature is then reported. The DTT is carried out on materials which have been subjected to ageing in the PAV [14].

The Direct Tension Test (DTT) device was introduced to test bitumens and determine their failure properties such as the stress and strain at failure (see Figure 2.3). The test procedure is used in the SHRP specifications to ensure that the strain at failure at the minimum pavement design temperature is greater than 1.0 %. At a temperature and loading rate where the strain to failure is less than approximately 1.0 % bitumen acts as a brittle material [14].

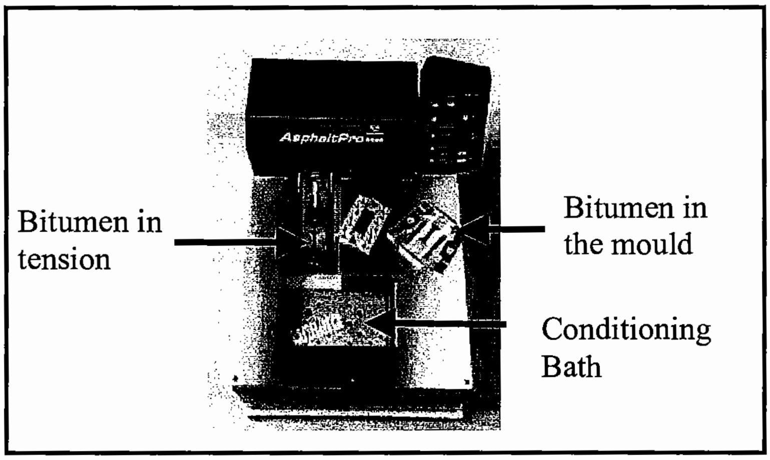


Figure 2.3 Direct Tension Test equipment

A research done by Mckelvey et al [29] investigated fracture of bituminous composites using the tensile test (See Figure 2.4). They concluded that under special circumstances of high strain rate and large incipient flaws, bitumen at +10°C can be made to fail in a brittle matter. But usually the failure of both the bitumen and that of bitumen-sand composites at +10°C and 0.1 sec^{-1} is ductile. The failure is then caused by nucleation of voids at sand particles which subsequently grow to give a final fracture.

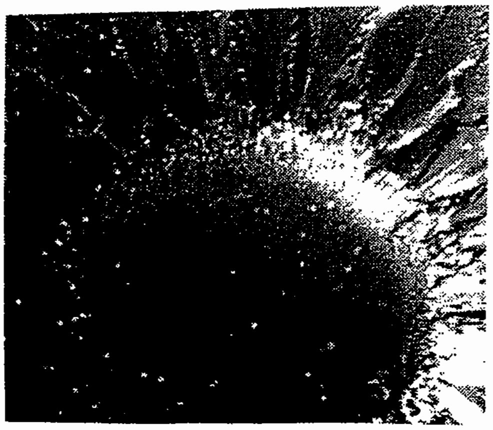


Figure 2.4 Fracture surface of pure bitumen at +10°C [29]

2.2.6 Intermediate Temperature Properties

The properties of the binder between the two extremes of mixing temperature and brittle point are determined using the Dynamic Shear Rheometer (DSR). The Rheometer measures the elastic and viscous nature of the bitumen across a range of temperatures. SHRP established links between the rheological behaviour of the bitumen and the end performance of a pavement [19]. In particular, rheological parameters were established for fatigue and rutting tendencies.

2.2.6.1 Dynamic Shear Rheometer (DSR)

The dynamic shear rheometer (DSR) is used to measure the rheological characteristics of bitumen at different temperatures, frequencies, strains and stress levels using oscillatory-type testing. It is generally conducted within the region of linear viscoelastic response. The principles involved in dynamic shear rheometry testing are illustrated in Figure 2.5, where the bitumen is sandwiched between a spindle and a base plate. The spindle, which can be either a disc-shaped plate or cone, is allowed to rotate while the base plate remains fixed during testing. The temperature of the bitumen in the DSR can be accurately controlled by means of a fluid bath enclosing the whole environment around the bitumen.

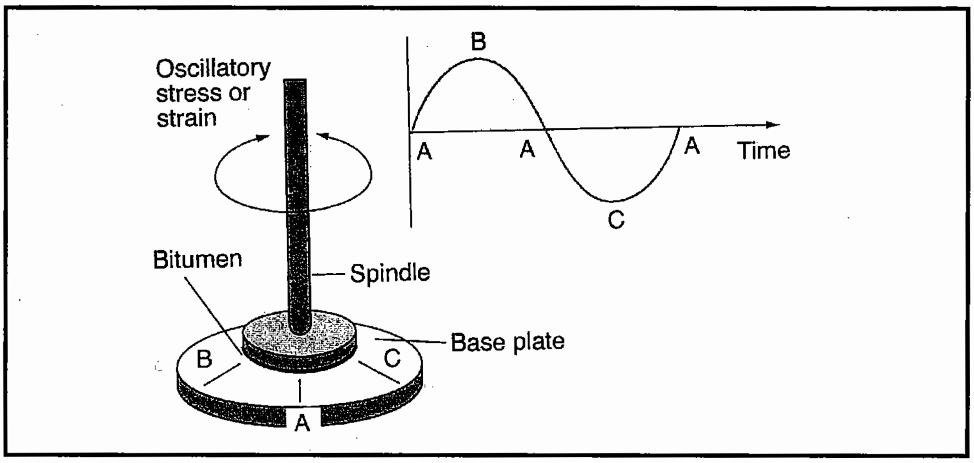


Figure 2.5 Principles involved in Dynamic Shear Rheometer test [30]

The test is performed by oscillating the spindle about its own axis such that a radial line through point A moves to point B, then reverses direction and moves past point A to point C, followed by a further reversal and movement back to point A. This oscillation comprises one smooth, continuous cycle which can be continuously repeated during the test. Normally DSR tests are carried out over a range of frequencies (number of cycles per second) and temperatures.

DSR tests can be carried out in either controlled stress or controlled strain testing modes. In either mode of testing the complex shear modulus, G*, is calculated as the ratio of shear stress to shear strain as shown in Figure 2.6. The complex shear modulus, which provides a measure of the total resistance to deformation when the bitumen is subjected to shear loading, is comprised of elastic and viscous components and is defined as a complex number relating the amplitude and phase difference of stress and strain in harmonic oscillation such that:

$$G^* = G' + iG'' \qquad (2.1)$$

These componenets are known as the storage modulus, G′, and loss modulus, G″, respectively, and are related to the complex modulus and to each other by means of the phase angle, δ, which is the phase lag between the shear stress and shear strain responses during the test.

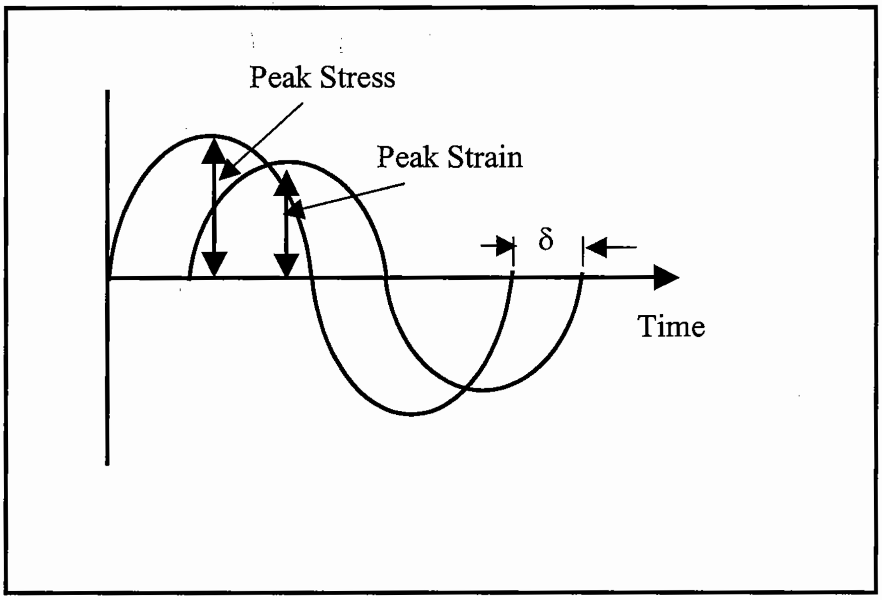


Figure 2.6 Stress and strain for one cycle

2.3 Rheological Properties of Bitumen

Rheology is defined as the study of the deformation and flow of materials. In general terms, rheology explains the behaviour of materials when subjected to a stress. There are two extremes of behaviour for any material: elastic and viscous. If a stress is applied to a perfectly elastic material then the resultant strain appears immediately. A load is applied to the materials and the material responds immediately by deforming; when the load is removed, the material recovers its original form instantaneously. Viscous materials behave differently as the resultant strain does not accumulating until the stress is removed. In this case, when the load is applied the material will show an initial resistance to the load but will then flow. When the load is removed, there is no recovery of the initial form.

If the stress is continually applied and removed in a cyclic fashion then, for elastic materials, the stress and strain are said to be 'in phase' whereas for viscous materials the stress and strain are completely 'out of phase'. Another way of expressing this is to state that the strain lags behind the stress by an angle of 90° for viscous material whereas for elastic materials the angle is 0°.

Bitumen exhibits behaviour which is somewhere between these two extremes, which is described as 'viscoelastic' and the stress and strain are out of phase by an angle delta (δ) (phase angle) which will have a value between 0° and 90°.

Most rheology measurements on bitumen are carried out using parallel plate geometry as shown in Figure 2.5 which applies shear to the sample.

2.3.1 Bitumen Ageing

Freddy, et al [19] stated that the first significant hardening of the bitumen takes place in the pugmill or drum mixer where heated aggregate is mixed with hot bitumen. During the short mixing time, the bitumen, which is in very thin films, is exposed to air at temperatures which range from 135 to 163°C. Substantial rheological changes such as a decrease in penetration and an increase in viscosity of the bitumen take place during this short mixing period. Age hardening of the bitumen continues, although at much slower rate, while the bituminous mixture is processed through a storage silo, transported to the paving site, laid, and compacted. After the mixture has cooled and the pavement has been opened to traffic, the age hardening process continues at a significantly slower rate for the first 2-3 years until the pavement approaches its limiting density under traffic. Thereafter, the rate of age hardening is further reduced and longer time periods are needed to observe the changes in the rheological properties of bitumen.

Age hardening in service takes place at an accelerated rate if the bituminous mixture in the pavement has a higher air void content than originally designed, which provides for easy entry of air, water and light. Thicker bitumen films around the aggregate particles harden at a slower rate compared to thin films.

Many long term pavement performance studies involve periodic core sampling and testing to determine the changing bitumen properties such as penetration at 25°C and viscosity at 60°C. Changes in such properties have been known to affect pavement performance with time [31,32].

2.3.1.1 Short-Term Ageing

There are two methods for simulating short term ageing of bitumen: The thin film oven test (TFOT) [ASTM D1754], and the rolling thin film oven test (RTFOT) [ASTM D 2872]. The TFOT generally simulates the bitumen hardening which occurs in the pugmill of a bituminous mixture batch facility. In this test the bitumen is stored at 163°C for five hours in a layer 3.2mm thick.

The rolling thin film oven test (RTFOT) [ASTM D 2872] simulates what happens to a bitumen during mixing. In this test eight cylindrical glass containers, each containing 35g of bitumen, are fixed in a vertical rotating shelf. During the test the bitumen flows continuously around the inner surface of each container in a relatively thin film with pre-heated air periodically blown into each glass jar. The test temperature is normally 163°C for a period of 75 minutes. Clearly the conditions in the test are not identical to those found in practice but experience has shown that the amount of hardening in the RTFOT correlates reasonably well with that observed in a conventional batch mixer [1].

2.3.1.2 Long-Term Ageing

Long term ageing, which occurs after construction of a pavement, continues as a result of oxidation at moderate temperature. The Pressure Ageing Vessel (PAV) [14] was developed by SHRP to simulate long term ageing of bitumens. In this method, 50 grams of aged bitumen, by the TFOT or RTFOT method, is placed on one of 10 stainless steel plates, which are placed into the pressure vessel. The bitumen is then aged under a pressure of 2.7MPa at temperatures between 90 and 110°C for 20 hours. After this time the pressure is slowly released over a time period of 8 to 10 minutes. Finally, to remove any contained air, the samples are placed in an oven at 150°C for 30 minutes.

2.3.2 Temperature Susceptibility

Bitumen is a thermoplastic material. Its consistency changes with temperature. Temperature susceptibility is the rate at which the consistency of bitumen changes with a change in temperature and is a very important property of bitumen, and is usually quantified through parameters calculated from consistency measurements made at two different temperatures. Different approaches for determining temperature susceptibility of bitumen are currently used.

2.3.2.1 Penetration Index (PI)

Pfeiffer and van Doormaal (see ref.[1], pp75) expressed the temperature susceptibility quantitatively by a term designated as "penetration index" (PI). If the logarithm of penetration, P, is plotted against temperature, T, a straight line is obtained such that:

$$\log P = AT + K \tag{2.2}$$

Where

A = is the tempertaure susceptibility of the logarithm of penetration

k = is a constant

The value of 'A' varies from about 0.015 to 0.06 showing that there may be a considerable difference in temperature susceptibility. Pfeiffer and van Doormaal developed an expression for the temperature susceptibility which would assume a value of about zero for road bitumen. For this reason they defined penetration index (PI) as:

$$PI = \frac{20(1-25A)}{1+50A} \tag{2.3}$$

The value of PI ranges from about –3 for highly temperature susceptible bitumen to about +7 for highly blown low temperature susceptible bitumen [1]. Most paving bitumens have a penetration index between +1 and –1 [19]. The values of A and PI can be derived from penetration measurements at two temperatures.

2.3.3 Bitumen Stiffness

Bitumens are visco-elastic materials and their deformation under stress is a function of both temperature and loading time. Stiffness (or stiffness modulus) is the relationship between stress and strain as a function of time of loading and temperature, this relationship between stress, strain and time is also referred to as the rheological behaviour of bitumen or mixture. Ideally, for a pavement surface course, increased bitumen stiffness is desirable at high service temperature to avoid rutting, and decreased bitumen stiffness is desirable to resist low temperature shrinkage cracking. High bitumen stiffness is primarily responsible for cracking at low service temperature [19,33].

In order to define the visco-elastic properties, Van der poel [34], in 1954, introduced the concept of stiffness modulus as a fundamental parameter to describe the mechanical properties of bitumens, by analogy with elastic modulus of solids. He suggested a single parameter termed stiffness (S) as follows:

$$S(t,T) = \frac{\sigma}{\epsilon} \tag{2.4}$$

where,

S = stiffness, in Pa

σ = axial stress;

ε = axial strain;

t = loading time; and

T = temperature.

2.3.4 Glass Transition Temperature

The observation of the glass transition phenomenon and the measurement of the glass transition temperature T_g for pure bitumens were started in the early 1960s [35]. The glass transition temperature, T_g, is defined as the temperature range at which amorphous polymers change from a glassy state to a fluid. Bitumens have similar characteristics to amorphous polymers and therefore the glass transition temperature of a bitumen can also be determined [36].

2.4 Viscoelastic Properties of Bitumen

Bitumens are viscoelastic materials, viscous at high temperatures and elastic at low temperatures and exhibiting viscoelastic behaviour under intermedaite conditions.

Materials with elastic behaviour return to their initial state after removal of the applied loads, whereas permanent deformations remain under applied loads under viscous behaviour. Temperature is the most critical factor affecting the behaviour of viscoelastic materials; another factor affecting this behaviour is the loading time or rate of loading. Bitumen behaves like an elastic solid at high rate of loading, whereas it behaves as a viscous liquid at long times of loading.

Cheung [37] studied the mechanical behaviour of bitumen and developed mathematical models. He presented the range of deformation behaviour of bitumen using mechanism maps (see Figure 2.7). Six regimes were identified, all of which were described by physical models, based on the understanding of the molecular structure of pure bitumen. He concluded that at temperatures above the glass transition the deformation behaviour of the bitumens is linear viscous at low stress levels and power-law creep at high stress levels. The transition stress from linear to power-law behaviour is a measure of the failure strength of the structural linkage in bitumen. At temperatures in the vicinity of glass transition, the temperature dependence of deformation is described by the diffusion model. At temperatures well above the glass transition, it described by the free volume model.

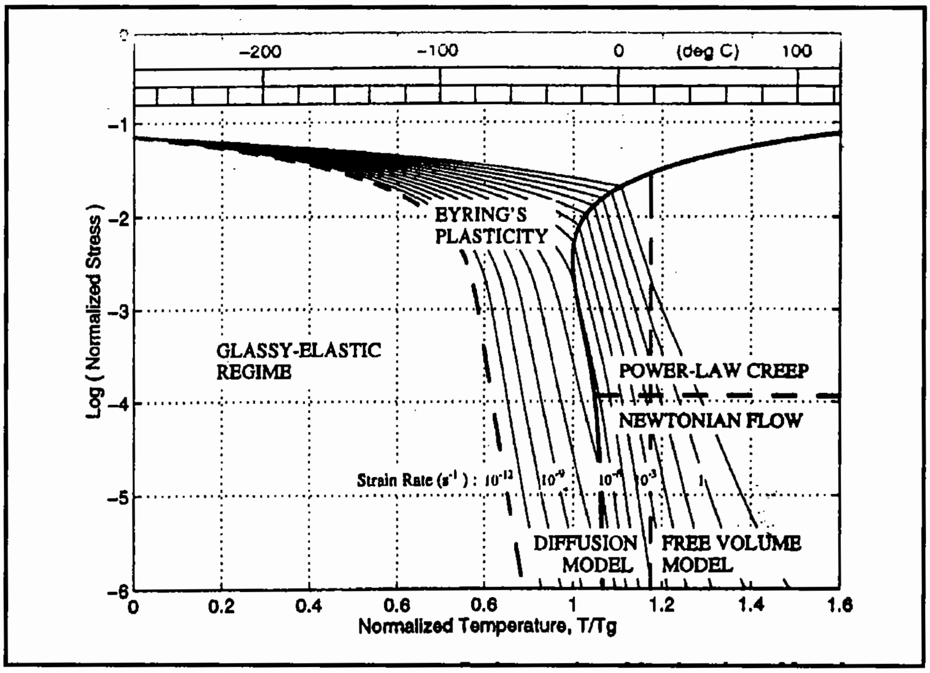


Figure 2.7 Stress/temperature deformation-mechanism map in tension for 50pen grade bitumen [37]

2.5 Bitumen Healing

Bitumen is a very complex material. Another aspect that makes it even more complex is the aspect of healing. Healing can be defined [10] as the self restoring capacity of the bituminous mix. This means that a mix can sustain less load repetitions if the test specimens are subjected to repeated cyclic loading than it can sustain if, between the load cycles, rest periods are introduced. The rest period is the time between consecutive applications of wheel loads, and they are important as they allow time for cracks to heal and stresses and strains to relax due to viscous flow of the bitumen. Rest periods are different for different roads and for different times of the day as they depend on the volume of traffic [4].

The healing phenomenon is not well understood. The significance of rest periods in extending the fatigue life of asphalt mixture has been well established by researchers [38,39,40,41]. The general conclusion is that the inclusion of a rest period gives an increase in fatigue life of between 5 and 25 times dependent upon the ratio of the loading duration to the rest period. But there are few publications which relate to the healing of pure bitumen. Research has shown clear evidence of a healing mechanism occurring in asphalt mixture during rest periods [42,43].

Research was done by Hammoun, de la Roche, and Piau [7] to estimate the healing capacities of bitumen using a specific test called Repeated Local Fracture of Bitumen test. Further reference to the test is made in chapter 3 section 3.6.4 (fracture of bitumen between hemi-spherically shaped end platens). They stated that the healing capacity of bitumen depends on the rest period duration. At a rest period of 4 hours they found that the slope of the curve before the sudden drop (see Figure 2.8) is the same and this lead to the conclusion that near total healing had taken place of the crack initiated during the first loading. However, the slope of the curve after the sudden drop is a little lower, indicating that the second loading led to a bigger crack opening than the first one. Also they examined a rest period of 2 minutes, and observed that some healing of the materials took place. The comparison of the initial slopes for the first and second loading showed that healing was only partial. They also concluded that healing capacity is linked to temperature: a more elevated temperature, like a longer rest period, increases the healing capacity of the bitumen.

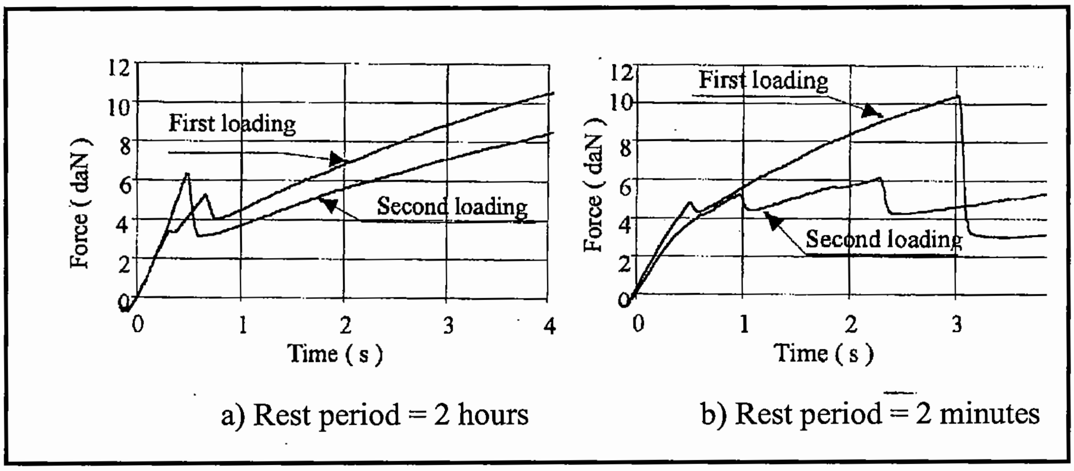


Figure 2.8 Force Vs time at Local Fracture test for 50/70 pen bitumen at 0°C after a rest period [7]

2.6 Types of Bitumen Modifiers

A variety of additives are currently used as bitumen modifiers in paving applications. They can be classified, on the basis of their composition and effects as: polymers (elastomeric and plastomeric), fillers, fibres, hydrocarbons, antistripping agents, oxidants, antioxidants, crumb rubber, and extender [44]. Bitumen modifiers are classified on the basis of the mechanism by which the modifier alters the bitumen properties, the composition, and physical nature of the modifier, or the target bitumen property that must be modified.

There are a large number of bitumen modifiers used in paving application; Bahia et al [44] listed a total 48 commercial brands of bitumen modifiers, classified in six categories: filler and extender, thermoplastic polymers, thermoset polymers, liquid polymers, ageing inhabitors, and adhesion promoters.

2.7 Review of Mineral Filler

2.7.1 General

Asphalt paving mixtures have been designed to include mineral filler since about 1890 [45]. The term mineral filler has been generally applied to a fraction of the mineral aggregate, most or all of which passes the No. 200 sieve (75μm).

Normally, the mineral aggregate represents from 65 to 85% of the total volume of an asphalt mixture [46]. Paving mixtures are classified in general mix types according to the gradation of the mineral aggregate. It can be stated that the contact points within the aggregate skeleton in the pavement, and the properties of the binder within this structure, are two important factors determining the performance of the pavement. Both factors can be influenced by the mineral filler in the paving mixture. This finest portion can participate in producing contact points and also, because of its fineness, can be suspended in the bitumen, changing the properties of the binder films [46].

When incorporated in the mixture, filler greatly increases the surface area which must be coated with bitumen. If these surfaces are compatible and easily coated with bitumen considerable benefits can be anticipated from the use of filler.

2.7.2 Definitions of Fillers

Mineral fillers are generally considered to be fine mineral materials a high proportion (at least 65% by ASTM and AASHO specifications) of which will pass the No. 200 (75μm) sieve. This description is improved by adding a statement to the effect that filler is important because of the surface area involved, and that properties of a pavement which may be improved by the use of filler include strength, plasticity, amount of void, resistance to water action, and resistance to weathering.

Tunnicliff [47] suggested that filler is mineral material which is in suspension in the bitumen and which results in a mortar with a stiffer consistency. Thus, bitumen is filled with mineral matter because it is desirable to increase its viscosity. In 1980 Ervin and David [48] defined the filler in their study as the materials passing a No. 200 (75μm) sieve.

The SHRP developed a mix design procedure for hot mix asphalt referred to as Superpave. The only requirement with Superpave for mineral aggregate finer than 0.075 mm (filler) was a range of filler-to-bitumen ratio (Superpave use the term dust-to-asphalt ratio) of between 0.6 and 1.2 based on mass [49].

2.7.3 Mineral Filler Properties

Usually bituminous mixtures have been designed to include mineral filler. The fact that many satisfactory mixtures are designed with different quantities and various types of filler, thus it is important to classify and characterise the filler [45]. Therefore, filler characteristics can be classified as follows:

1- Primary characteristics of fundamental importance: particle size, size distribution, shape.
2- Primary mineralogical characteristics: texture, hardness, strength, specific gravity.
3- Secondary characteristics dependent on one or more primary characteristics: void content, surface area.

The followings are the summary of the general properties of the mineral filler:

1- Particle size and shape: A number of engineering properties, e.g. Compressibility, is related directly to the particle size. The grading curve is a graphical representation of the particle size, and is therefore useful in itself as a mean of describing the material. Filler as a fine grained materials, the shape of the particle has the greater influence on its properties. Particle shapes, which have been called irregular, have been identified as the source of unusual relationships involving viscosity [45]
2- Specific gravity: Determination of the specific gravity of the filler are required for the calculation of the void ratio of the specimen and in the particle size analysis. The high value of the specific gravity indicates the presence of unusual minerals.
3- Bulk density: The quantities referred to as density provide a measure of the quantity of material related to the amount of space it occupies. It is density is the total mass divided by the total volume of material.
4- pH filler-water: pH value, in some natural filler, especially those containing sulphides a high acid content may be found or high alkali content. The measure of acidity/alkalinity is given by the pH value of the material.

5- Specific surface area: It is normally applied as a measure of fineness of cement (cm^3/g). As it is an indicator of fineness, there are no limits set for interpreting the values obtained

6- Plasticity characteristics: Plasticity is a major characteristic of the so-called "cohesive" materials, that is materials containing an appreciable proportion of clay particles. It is the property that enables a material to suffer deformation without noticeable elastic recovery and without cracking or crumbling.

2.7.4 Methods Used for Filler Evaluation

A large number of the test methods which have been used for filler evaluation have been adopted from soil technology. These have the advantage of utilising available equipment and techniques.

Grain Size Analysis [BS 812-103.2:1989]

Hydrometer analysis for grain size distribution is a valuable means of evaluating those characteristics of filler which are related to particle size [50].

Liquid Limit [BS 1377-2:1990]

A range of 25% to 50% has been proposed for the liquid limit of the filler materials passing No. 200 (75μm) [51].

Plasticity Index [BS 1377-2:1990]

Some specifications require that blended filler passing a No.40 sieve shall have a plasticity index of not more than 6 and L.L. of not more than 25 [51].

From the available reviewed research in practice there seems to be agreement that material which forms into hard lumps after wetting and drying is not suitable as filler.

Specific Gravity [BS 1377-2:1990]

A small pyknometer method is a suitable method for measuring the specific gravity of the filler particles [51].

2.7.5 Filler Behaviour

The filler, as one of the ingredients in a bituminous mixture, plays a major role in determining the properties and behaviour of the mixture. Generally, the role of the filler in the mixture has been shown to be very complex. On one hand, the filler serves as an inert material for filling the voids between coarse aggregate particles in the mixture. On the other hand, because of its fineness and its surface characteristics, the filler serves as an active material, the activity being clearly in the properties at the interface between the filler and the bitumen. These properties have a great influence on improving the properties of the binder, and on such basic and important properties of the mixture as mechanical behaviour, optimum bitumen content, durability, workability, ..etc [52].

Traxler [53] listed size, size distribution, and shape as the fundamental physical properties of fillers. These properties in turn were said to determine void content and average void size which were of primary importance.

Warden, Hudson and Howell in 1959 [54] conducted a study of several fillers. Evaluation was based on the effect of these fillers on consistency, ductility, stability, ability to fill voids, resistance to water, and temperature susceptibility. It was believed that the softening point could be raised or the penetration lowered significantly. Research done by Tunnicliff in 1967 [45] concluded that shearing resistance due to cohesion in paving mixtures is a function of concentration of filler in the binder. As filler concentration increases, shearing resistance due to cohesion increases.

In 1978 Craus, Ishai and Sides [55] stated that the loss of adhesion in a mixture induces instability and promotes failure conditions in a bituminous pavement and defined adhesion of the bitumen to the surface of the aggregate as the property of binder to adhere to the aggregate surface, and to maintain this condition in the presence of water. They also suggested that in addition to the inert mechanism of the filler, some types of filler show physio-chemical activity at the aggregate-mortar interface. This activity has a great influence on the adhesion and stripping of the mixture. The activity also depends on the physio-chemical nature of the aggregate.

Huschek and Angst in 1980 [52] reported that the geometric irregularity (shape, angularity, and surface texture) plays a major part in the role of filler in the mixture. Specifically, geometric irregularity affects the optimum bitumen content of the mixture, the interfacial properties in a filler-bitumen system (mastic) and the rheological behaviour of the mastic. They said that this has a direct effect on the structural and mechanical behaviour of bituminous concrete mixtures.

Six types of filler were studied by Ishai, Craus, and Sides [56]. They concluded that: it was evident that the properties and the proportion of the mastic, as related to the properties of their filler, are the major filler factors that govern the strength behaviour of the mixture.

Bassam and Hamad [57] conducted a research using baghouse fines as mineral filler in the asphalt mixture. They concluded that filler affects the mix properties to a large extent; also increasing the percentage of baghouse fines will increase the viscosity, shear strength, and softening point of the mortar. Another research done by Anderson and Terris in 1983 [58] used baghouse fines as a filler. The researchers reported that baghouse fines can stiffen the bitumen and consequently affect compaction. They concluded that baghouse fines can be quite coarse, exceeding the limits generally accepted for mineral filler and get still be effective.

Sewage sludge ash waste is also used as mineral filler in bituminous mixtures [59]. A research done by Mohammed, et al [2] concluded that municipal sludge ash waste can be utilised as a replacement for filler in asphaltic concrete mixtures, and they suggested the suitability of the waste ash as a filler in hot environments.

A variety of fillers are currently used in asphalt mixtures [44,60,61,62,63], such as: coke dust, cement bypass dust or cement kiln dust, calcium carbonate, carbon black, .. etc.

2.7.6 The Filler–Bitumen System

Two mechanisms of attraction, absorption and adsorption, have been used in the literature. Adsorption is a surface phenomenon involving molecular forces on or between surfaces. Absorption is also a surface phenomenon, but the action is through, not on the surface. Absorption is a very complex subject. If absorption occurs there must be space or volume for the absorbed material to occupy after it goes through the surface. Filler presents a large surface and therefore a large capacity for adsorption, but its volume, and therefore its capacity for absorption is small. Absorption may occur, but probably its effect is negligible, and its presence does not eliminate adsorption.

It thought that bitumen absorption, with its consequent effects upon the remaining effective bitumen film, may contribute to premature pavement failure through various mechanism such as accelerated ageing and cracking. The amount and extent of bitumen absorption will depend on such properties as viscosity, composition, surface tension and time. The bitumen directly in contact with the particle is important because bitumen must adhere and remain adherent under different stresses. However, the bitumen that lie between the particles serves a major cohesive role for the bitumen-filler mortar by binding the particles together and maintaining the integrity of the mortar [97].

Tunnicliff [47] assumed that bitumen is most rigid character immediately adjacent to the filler particle, and becomes increasingly less rigid as the distance from the particle increases. The forces of surface attraction gradually dissipate until they become negligible and the bitumen retains its original consistency, or until they encounter the influence of similar forces from an adjacent particle-bitumen interface. The most desirable consistency occurs when all of the bitumen is under the influence of surface attraction.

Particle size distribution in a filler-bitumen system has the same significance as in any system of particles. The largest particles, when packed as closely as possible, do not occupy all available space, and if the unoccupied space is to be filled smaller particles must be provided. The largest particles may be considered to be somewhat larger than the largest filler particle because they include the filler particle and the adsorbed layer of bitumen [47]. Void space between these particles is occupied by bitumen which is not influenced by adsorption. Hence, it is necessary to provide smaller and smaller particles until no free bitumen is present. Probably there is always a small amount of bitumen not adsorbed because of imperfect packing.

The adsorbed film of bitumen has been described in terms of stiffness which exceeds the stiffness of the original bitumen. The stiffening is caused by the surface energy of attraction between the filler surface and the bitumen. This adsorbed layer would then exhibit greater resistance to external forces attempting to puncture or penetrate through it.

Some researchers [47] noted that the asphaltenes are adsorbed more readily, that they are the most complex components of bitumen, and that they are also the most viscous materials. The remaining bitumen, which is less susceptible to adsorption, is more fluid and of an oily nature.

Research has been done by Cooley, et al [49] to evaluate the stiffening potential of fillers on bitumen binder using the Superpave binder tests. They found that there is an increase in the Superpave binder test properties. These increases ranged between three and four times over that of the neat bitumen binder. They also observed the relationship between the change in softening point temperature and the stiffening effect of fines on asphalt binder.

Another research done by Soenen and Teugels [64] examined possible bitumen filler interaction by means of dynamic shear rheology. They found that the relative stiffening effects are only dependent on the volume percentage of filler added. Also the stiffening was independent of the type of filler (quartz, rhyolite, and tuff rhyolite) as well as the origin of the bitumen (4 bitumens) as shown in Figure 2.9.

2.8 Mineral Aggregate

Mineral aggregate may be divided into three main types:

- Natural: gravel and sand
- Processed: natural crushed
- Synthetic: slags, fired clay, etc

The particle sizes of natural aggregate are often suitable for the production of bituminous mixture without the need for mechanical size reduction. It is generally considered that crushed aggregate is more desirable as the angular particle shape contributes to mechanical strength.

The particle size of aggregate used for bituminous mixture can range from 37.5mm down to fine dust. The maximum aggregate size depends upon the type of the mixture, the application for which it is used and the thickness of the layer.

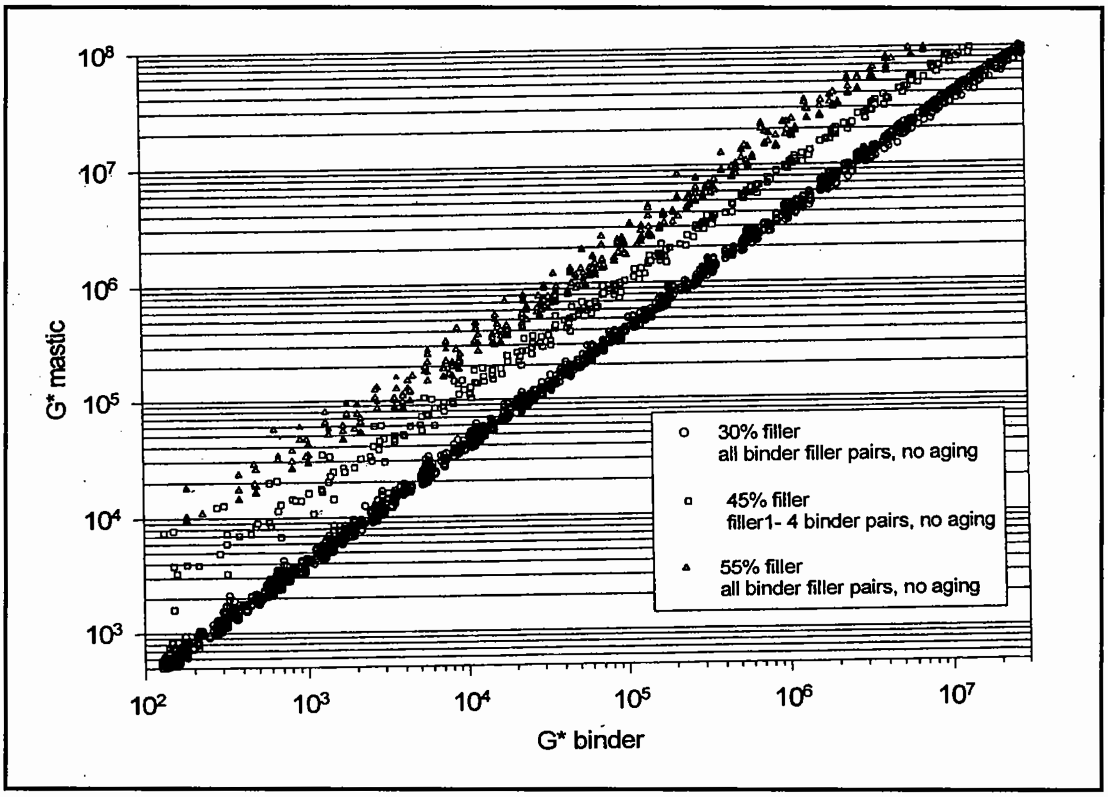


Figure 2.9 The increase in modulus of the binder after the addition of different types and concentrations of fillers [64]

In the U.K. the test methods which are typically specified for the assessment of aggregate are found in BS 812:1989 [50]. The testing of aggregate has been reviewed as part of the European harmonisation process. The proposed test methods to assess a range of aggregate properties include skid resistance (BS EN 1097-8,1999), abrasion (BS EN 1097-1,1999), fragmentation/impact (BS EN 1097-2,1998), soundness (BS EN 1367-1, 1998) and thermal shock (draft Pre EN 1367-5-1996).

The implementation of the new European Aggregate Standard started in the UK in January 2004. To aid the understanding of the European standard, the UK adopted the use of guidance documents, known as PD 6682 series (document published by BSI). PD6682-2: 2003 recommends the properties of aggregates and filler obtained by processing natural, manufactured or recycled materials for use in bituminous mixtures and surface treatments [1].

2.9 Types of Asphalt Mixture

If all the possible combinations of particle sizes and bitumen contents are considered, it is obvious that an infinite number of mixture compositions are possible. The following is a summary of the different types of asphalt mixtures:

Dense Bitumen Macadam (DBM) (BS 4987)

This is a continuously graded mixture. It contains all the aggregate sizes from the maximum down to filler. This results in an aggregate skeleton where there is stone on stone contact throughout the mixture. In other countries continuously graded mixtures are normally known as asphaltic concrete. Macadams generally provide better resistance to permanent deformation than other mixes and can be stiffer, depending on the grade and type of bitumen used in the mixture.

Hot Rolled Asphalt (HRA) (BS 594:1988)

This is a gap-graded mixture consisting of a fines/bitumen mortar which, in the case of the wearing course mixture, also contains added filler. The fines are generally natural sand in contrast to the crushed material normally used in Dense Macadams. The mortar is mixed with a single sized coarse aggregate to provide a grading in which the intermediate sizes are missing.

Due to the greater quantity of fine materials present in HRA the amount of binder necessary is higher, since the surface area of the aggregate is greater, which increases the cost of production. However, the increased binder content improves the fatigue resistance of asphalts and makes them more impermeable to air and water, increasing their durability.

Coated Stone

Coated stone comprises virtually single sized coarse aggregates mixed with a soft grade bitumen and small quantity of fines and filler. It is similar to porous asphalt as will be discussed later.

Mastic Asphalt (BS 1447:1988)

Mastic asphalt is a fine mixture which is made with a high percentage of filler and hard grades of bitumen as specified in BS 1447:1988. The durability or resistance to the effect of the environment increases as void content decreases. However, the increase in bitumen and fine material means that the resistance to permanent deformation could be low and, to counter this, harder grades of bitumen are used.

Greater quantities of bitumen mean that the material is more expensive. Increased costs are also incurred with harder bitumens as the material must be mixed at higher temperature.

Heavy Duty Macadam

Continuously graded dense macadams are widely used in the U.K. as basecoarse and roadbase materials on heavily trafficked roads [1]. To cope with increase in traffic loading, a dense macadam with 50 pen bitumen and increased filler content has been developed. This material, which is known as Heavy Duty Macadam (HDM) has been found to have increased resistance to permanent deformation and a higher level of stiffness modulus when compared with conventional dense macadams. It is important that bituminous mixtures such as DBM and HDM are well compacted in order to ensure maximum aggregate to aggregate contact.

Porous Asphalt

Porous asphalts are used as surface courses so that any water deposited on the surface can percolate rapidly into the interconnected voids and drain laterally through the mixture [65]. This mixture is also known as pervious macadam or friction course. Porous asphalt which like coated stone consists of a single sized coarse aggregate with a little fines, filler and bitumen, is usually laid over an impervious surface. The void content of freshly laid porous asphalt can be over 20% and water will pass through it and drain off laterally. It has the additional advantage that it can significantly reduce traffic noise. The disadvantage of it is that the bitumen can oxidise relatively quickly and water can cause the bitumen to strip from the aggregate.

Stone Mastic Asphalt (SMA)

This is a wearing course developed in Germany [1]. It can be classified as a gap-graded mixture, consisting of coarse aggregate particles filled with fines/filler/bitumen mortar. It is a durable material with high stability due to stone-stone interlock of its coarse aggregate skeleton. SMA is very high stone content asphalt and uses the addition of fibres to keep the binder content at a comparable level to traditional asphalts [10].

If the bituminous mixture is not well chosen, early degradation of the pavement is possible. The type of the mixture to be used in a road should ideally be chosen according to the type of loads that will be imposed on the pavement, i.e. mixture more resistant to fatigue should be used in roads with intense traffic.

2.10 Mechanical Properties of Bituminous Mixes

2.10.1 Stiffness Modulus

Stiffness modulus is the ratio of stress to strain under uniaxial loading conditions and is analogous to Young's modulus of elasticity. Since asphalts are sensitive to temperature and loading time, this alternative terminology is used since stiffness modulus is also a function of these parameters and is, therefore, not a constant for a particular material [16]. The stiffness modulus is a function of load, deformation, specimen dimensions and Poisson's ratio.

Various methods have been employed to measure the stiffness of asphalt mixture. Conventional techniques have principally included uniaxial and triaxial and/or tension tests and flexural beam tests [1]. Cooper and Brown [66] introduced the indirect tensile test method to the UK with the development of the Nottingham

Asphalt Tester (NAT) which has subsequently gained widespread use in much of the world.

The indirect stiffness modulus test (ITSM), defined in DD213:1993 [67], is the most commonly used test method in the NAT and is used for the determination of the stiffness modulus of a specimen. The ITSM test is a non-destructive test using cylindrical specimens that may be prepared in the laboratory or sampled from site. It allows cylindrical specimens to be tested (100mm or 150mm in diameter).

The test is simple and can be completed quickly. The operator selects the target horizontal deformation and a target load pulse rise time. The load is measured with a precision strain gauged load cell and the deformations are measured with linear variable differential transformers (LVDTs). The load applied to the specimen is then automatically calculated by the computer and a number of conditioning pulses are applied to the specimen. These conditioning pulses are used to make any minor adjustments to the magnitude of the force needed to generate the specified horizontal deformation and to seat the loading strip correctly on the specimen [1]. Once the conditioning pulses have been completed, the system applies five load pulses. This generates an indirect movement on the horizontal diameter and, since the diameter of the specimen is known beforehand, the strain can be calculated. As the cross-sectional area of the specimen is also known and the force applied is measured, the applied stress can be calculated. Thus, since the stress and strain are now known, the stiffness modulus of the material can be calculated. The recommended test temperature in the UK is 20°C [67].

2.10.2 Permanent Deformation

Permanent deformation which takes place within a bituminous mixture develops primarily through shear displacement [1,65], though there may also a degree of densification under traffic, particularly in pavements which are not adequately compacted during construction. Permanent deformation behaviour of bituminous mixtures can be affected by several factors such as: aggregate type, aggregate gradation, binder type and content, degree of compaction, method of compaction, temperature and magnitude and frequency of loading

Probably the most widely used tests are the uniaxial creep, repeated load axial and repeated load triaxial methods. In the uniaxial creep test a constant load (stress) is applied to a cylindrical specimen and the accumulation of displacement (strain) with time is monitored. After some specified period the load may then be removed to assess the elastic recovery of the material [1]. One of the problems associated with this type of test is the lack of specimen confinement.

The repeated load axial test is very similar to the creep test except that the load is pulsed rather than maintained at a constant level. The period of duration of the load pulse is typically chosen to represent average material loading times due to passing traffic. This type of test can also be conducted in the Nottingham Asphalt Tester (NAT) to assess the permanent deformation resistance of bituminous mixture. The maximum deformation in each load cycle is recorded and plotted against the number of cumulative cycles. The repeated load triaxial test most closely reproduces the stress conditions in the pavement. It has the advantage that both vertical and horizontal stresses can be applied at the levels predicted in the pavement [1].

Another permanent deformation test developed by the Strategic Highway Research Program (SHRP) in the US [65] is the Simple Shear Test (SST). It is capable of directly applying shear stresses (as well as normal stresses) to a specimen.

The most popular type of laboratory simulative test is the wheel tracking test. This type of test is relatively simulative in nature. The size of the specimens tested ranges from relatively large slabs of material where the loading is applied through a full size pneumatic tyre to 200mm diameter cores cut from a pavement loaded through a solid rubber tyre of 200mm diameter. The test is usually terminated when either 45 minutes have elapsed or the central deformation exceeds 15mm.

2.10.3 Mixture Fatigue

One of the primary structural distress modes for pavements is fatigue cracking. Due to increasing concern with pavement cracking, the importance of testing and formulating fatigue performance of bituminous mixtures has been recognised for decades. Over this period, extensive fatigue testing has been carried and a number of different test configurations. This topic is the subject of the next chapter.

2.11 Summary

Binder, filler and aggregate properties have been reviewed. Bitumen can be described as a complex mixture of components with various chemical structures. The penetration test, softening point test, ductility and viscosity tests are the general tests for the binder. At low temperature, the bending beam rheometer and direct tension tests are recommended, in addition to the dynamic shear rheometer for intermediate temperature.

These are the key points:

- Bitumen is a complex mixture of components with various chemical structures. The majority of these structures are composed of carbon and hydrogen.
- Measurement of high temperature properties of bitumen give an indication of its ease of handling at a coating plant and usually measured by the viscosity of the bitumen at elevated temperature. At low temperature bitumen is characterised by using Fraass breaking point, the bending beam rheometer or direct tension test, whereas at intermediate temperature its properties are measured using the dynamic shear rheometer.
- High bitumen stiffness is primarily responsible for cracking at low service temperature.
- At temperature above the glass transition the deformation behaviour of the bitumen is linear viscous at low stress level and power law creep at high stress level.
- Mineral filer plays an important role on the asphalt mixture.
- Sudden fracture occurred in bitumen after first loading in tension using hemi-spherical end platens followed by a bigger crack opening at the second loading
- Healing capacity of bitumen depends on the rest period duration, and temperature.

- Filler size, particle distribution, shape and surface texture are the fundamental physical properties effect the optimum bitumen contents of the mixture.
- Filler changes the softening point temperature and binder stiffness.
- The relative stiffening effect of filler are only dependent on the volume of filler concentration.
- The stiffening effect of filler was independent of type of filler as well as the origin of the bitumen.

3

Review of Fatigue Characteristics

3.1 General

Many flexible pavement designers and researchers have expressed concern over the premature failure of pavements under various loading conditions [12]. Generally, based on laboratory studies it is considered that fatigue cracking caused by repeated loading from traffic is a major form of structural deterioration of asphalt pavement [68].

The word fatigue originated from the Latin expression fatīgāre which means "to tire" [69]. The word fatigue has become a widely accepted term in engineering vocabulary for the damage and failure of materials under cyclic loads. Research into the deformation and fracture of materials by fatigue dates back to the nineteenth century. This branch of study has long been concerned with engineering approaches to design against fatigue crack initiation and failure.

Fatigue is a term which applies to a change in properties which can occur in metallic or non metallic materials due to the repeated application of stresses or strains, although this term applies especially to those changes which lead to cracking or failure [69].

The fatigue response of asphalt pavements has been the subject of a number of investigations throughout the world for many years. The importance of the fatigue behaviour of asphalt mixtures is seen in the incorporation of a fatigue factor into the design of asphalt pavement [70]. Most pavement cracking is known as fatigue cracking because it grows slowly with repeated load applications, i.e. the material fatigues. In pavement with thin bituminous layers, load associated cracking starts at the undersides of the bituminous layer(s) due the tensile stresses induced by the traffic. Once the crack is formed, it will propagate towards the surface with repeated load applications. In thicker pavements the cracks may also start at the surface and propagate towards the bottom of the boundlayers.

3.2 Background of Fatigue Phenomenon

The expression 'fatigue' has been in use for a very long time. The first study of fatigue was done on metal. Richard [71] stated that "Wohler (1860) conducted systematic investigations of fatigue failure in railroad axles for the German railway industry. His work also led to the characterisation of fatigue behaviour in terms of stress amplitude-life (S-N) curves".

Fatigue considerations in the design of bituminous pavements date back to the early 1940's when increase in the traffic volume and the magnitude of wheel loads became significant. In 1942 Porter noted that flexible pavements failed under deflections as small as 0.02 to 0.03 inch (0.5-0.8 mm) [72]. In 1953, Nijboer and van der Poel [73] showed that cracks which often appeared in the later stages of the life of an asphaltic concrete could be related to the bending stress produced by the moving traffic, exceeding the flexural strength of the material. Examinations of the results of the AASHO Road Test [74] also revealed that cracking and initial failure of the pavement were primarily caused by repeated bending of the bituminous layers.

Fatigue failures occur in many different forms. Fluctuations in externally applied stresses or strains result in mechanical fatigue. Cyclic loads acting in association with high temperature cause creep - fatigue; when the temperature of the cyclically loaded component also fluctuates, thermomechanical fatigue (i.e. a combination of thermal and mechanical fatigue) is induced [69].

Non-load associated cracking is also considered an important performance aspect of an asphalt pavement. In moderate climates, thermal stresses generally are not high enough to cause instant cracking such as they can in cold climates. Temperature and stress variations below the cracking limit induce fatigue phenomena in the asphalt pavement and make it more susceptible to cracking due to thermal and/or traffic induced stresses [75].

3.3 Different Approaches to Fatigue

There are several different stages of fatigue damage in an engineering component where defects may nucleate in an initially undamaged section and propagate in a stable manner until catastrophic fracture ensues. The total fatigue life is defined as the sum of the number of cycles to initiate a fatigue crack and the number of cycles to propagate it to some final crack size.

Castell, Ingraffea and Irwin [76] identified two different crack growth patterns in pavements. The first is associated with tensile stresses at the bottom of the base. The second one is caused by tensile stresses at the top of the surface course, see Figure 3.1, when the load is ahead of or behind the crack. They said that the growth rate for surface cracks (growing downward) is much smaller than the internal crack growth rate (growing upward).

3.4 Binder Fracture

The cracking of asphalt mixtures during their service life is of importance to highway engineers. Many investigations have thus been conducted on the nature of cracking and the factors affecting it. It is generally accepted that as the mixture ages, it becomes more brittle and thus more susceptible to cracking [77,78]. A study of flexural cracking in asphalt beam specimens by Genin and Cebon [79] revealed that cracks in a typical asphalt composite often run through the binder rather than through the aggregate particles or aggregate/binder interface. Generally there are different approaches commonly used to analyse fatigue of asphalt pavements: stress-strain versus fatigue life, constitutive damage mechanics, dissipated energy and rate of change of dissipated energy.

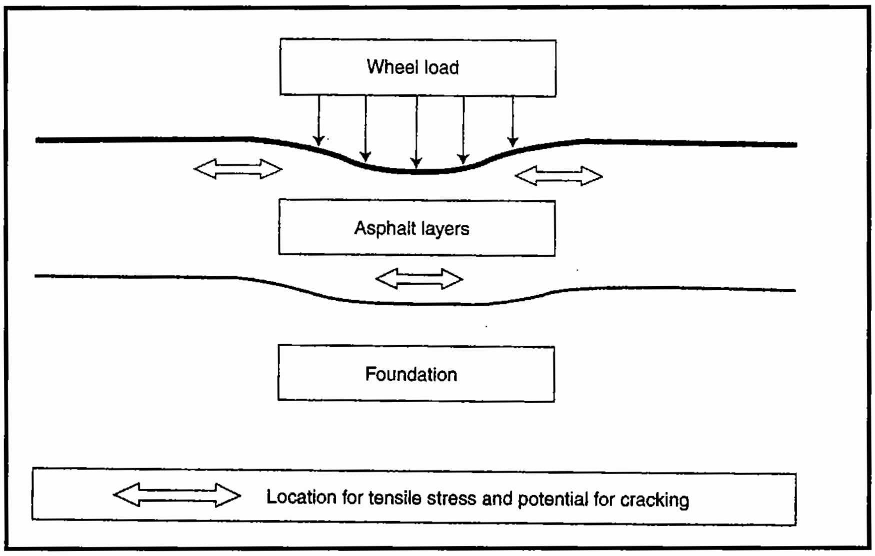


Figure 3.1 Locations for crack initiation

3.4.1 Fracture Mechanism Concepts

Moavenzadeh [78] stated that “the fracture of the material is simply due to overcoming the bond energy required to create new surfaces and since the bonding of crystalline solid materials is generally much stronger than that of amorphous materials such as bitumen, and it is very much greater than the bonding which develops between the bitumen and the aggregate, it is therefore not conceivable for a crack to propagate through the solid phase of the mixture”. Although there are instances in which the examination of fractured surfaces of the mixture have indicated the passage of a crack through the solid phase, this can primarily be attributed to the pre-existence of cracks and flaws in such pieces [79]. It can therefore be assumed that the fracture behaviour of the bitumen is the primary controlling factor in the cracking of the mixture.

3.4.2 Theory of Brittle Fracture

The basic concept behind the theory of brittle failure was that since the material failed when the average strain energy was far below the theoretical value required for failure, the strain energy was non-uniformly distributed. This non-uniform distribution of strain energy was attributed to the presence of small inherent cracks which acted as stress concentrators and thereby produced the corresponding concentrations of strain energy [71,78,80]. Griffith's theory states that the most critical crack will grow only when elastic energy released during crack growth exceeds the energy needed to create the new surface area [71].

3.4.3 Application of Brittle Fracture Theory to Bitumen

Bitumen is a viscoelastic material whose properties are dependent on the loading rate and temperature, as well as the degree to which it has aged. It is known that, at extreme low temperatures, the bitumen shows almost complete brittle behaviour. It would be expected, however, that in normal service temperatures, it would show a certain degree of plastic flow, which depends upon the temperature, the rate of loading, and the degree of ageing.

Moavenzadeh [78] in his research on bitumen fracture concluded that at sufficiently low temperature, the bitumen behaves as a brittle, amorphous material and the Griffith theory of brittle fracture can be applied to study the fracture behaviour of bitumen.

3.4.4 Stress Analysis of Cracks

The fracture of flawed components also may be analysed by a stress analysis based on the concepts of elastic theory. A crack in a solid can be expressed in three different modes [71,81], as illustrated in Figure 3.2. The displacements of the crack surfaces are perpendicular to the plane of the crack.

Mode I

Opening or tensile mode, where the crack surfaces move directly apart.

Mode II

Sliding or in-plane shear mode, where the crack surfaces slide over one another in a direction perpendicular to the leading edge of the crack.

Mode III

Tearing or anti-plane shear mode, where the crack surfaces move relative to one another and parallel to the leading edge of the crack. This may be regarded as a pure shear problem.

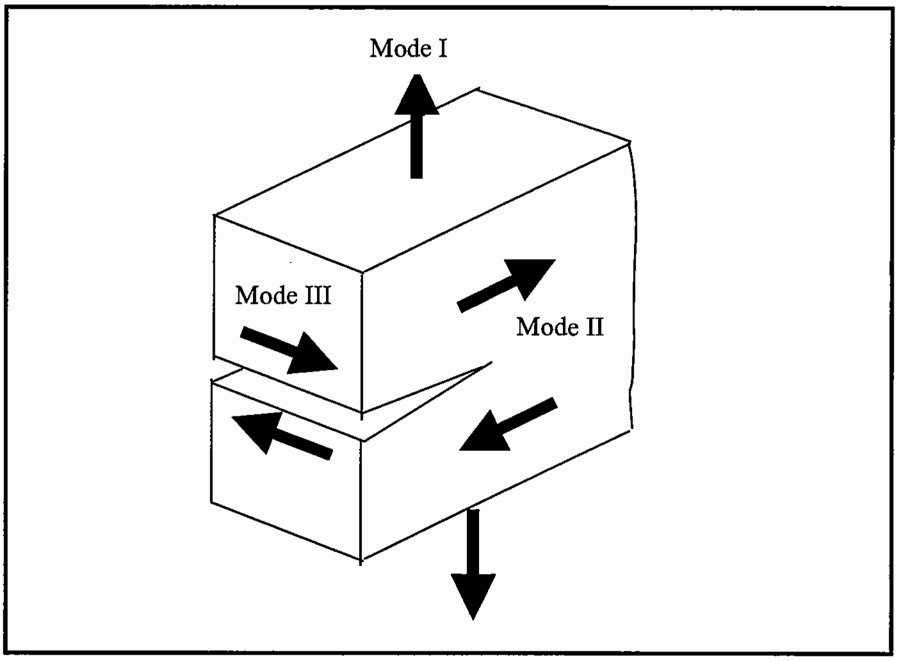


Figure 3.2 Basic modes of loading involving different crack surface displacement

The elastic stress field in Mode I loading can be calculated in various ways [81]. Irwin and Williams (1957) (see Ref. [12], pp206) examined the stress distribution around the tip of a crack in a linear elastic solid and defined what they called "stress intensity factor," relating the near-field stresses to the far-field stresses (stresses caused by applied loads). The stress intensity factor, denoted by K, defines the near-filed stresses for any of the three possible deformation modes at a crack tip shown in Figure 3.2.

In 1983 Schapery (see ref.[12], p206) advanced the application of the J integral to viscoelastic materials and gave a comprehensive treatment of this subject in his paper. Little and Mahboub [82] studied the fracture mechanics properties of first generation plasticized sulphur binders and recommended using J_{Ic} as a fracture mechanics characterisation parameter for sulphur binders. Abdulshafi and Majidzadeh [83] successfully applied J_{Ic} criteria to hot mix Marshall asphalt specimens. In 1989 Dongre et al. [84] studied the effect of bitumen source on mixture behaviour, using the J_{Ic} parameter. Although these studies on hot mix asphalt were successful, the tests required to develop the fracture parameters are difficult to perform and are therefore not recommended as routine design or specification tests.

To date there has been little research on fracture mechanics conducted on neat bitumen or binders. However the complexity of fracture mechanics testing and characterisation dictates the use of surrogate methods of characterisation for specification purpose.

Research was done by Harvey and Cebon [85] to investigate the bitumen film in tensile tests. They used butt joints and a double cantilever beam specimen and the fracture mechanics approach. They found that the failure strain of bitumen is generally independent of equivalent strain rate, in both the brittle and ductile mode and concluded that the brittle toughness G_{IC} of bitumen is rate independent and notch sensitive see Figure 3.3. It was estimated to be between 5 Jm^{-2} and 10 Jm^{-2} for the 100pen grade bitumen.

3.5 Fatigue Concepts of Binder

The failure mechanism for both fracture and fatigue can be postulated in two ways. One mechanism is a single excursion of stress, with the pavement flaw size remaining constant, that rises to critical value and results in failure. The other mechanism is the propagation of a starter or initial flaw into a critical flaw size, which, stress remaining constant, results in failure. The critical stress beyond which a flawed material fails is related to fracture elastic materials [12].

Engineering materials have been observed to fail as a result of the repeated application of stresses that are considerably less than their single cycle or static load fracture stress. The repeated stresses that are insufficient in magnitude to produce failure in one cycle nonetheless induce damage in the material with every cycle. This damage accumulates and ultimately leads to failure. Such failure in a material is known as fatigue failure.

Traditionally, engineers have designed against fatigue by using S-N diagrams in which the magnitude of alternating stresses is plotted versus the logarithm of number of cycles to failure. An engineering structure may be designed to resist fatigue by limiting the magnitude of the stress applied to the structure to a value less than those described by the S-N curve. In case of some materials, the S-N diagram identifies a stress amplitude threshold below which fatigue failure would not occur even after infinite number of load application. Thus, using this information, design stresses can be limited to a value below the failure threshold.

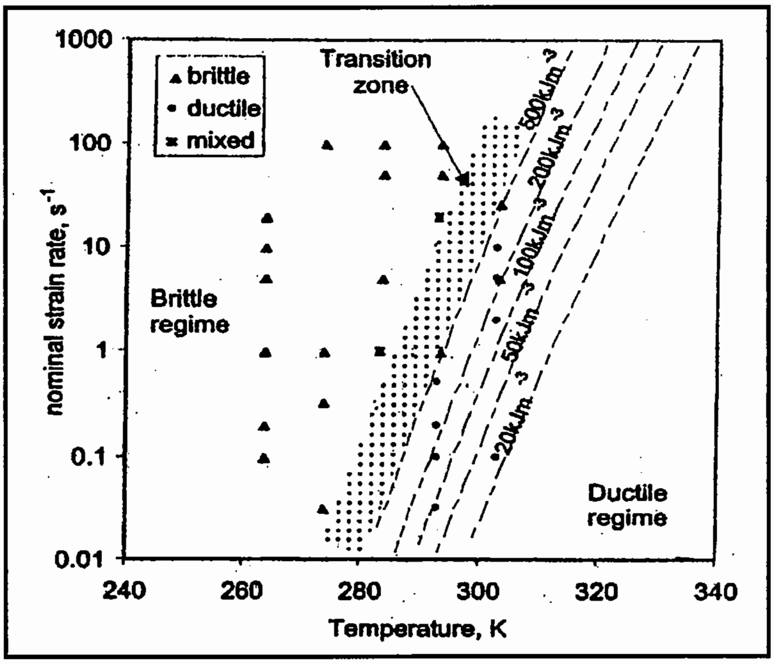


Figure 3.3 Mechanism map for bitumen films in tension [85]

Although a simple concept, the S-N diagram approach has two drawbacks:

1- Large numbers of time-consuming tests are required to prepare the S-N diagrams, and
2- The concept does not address the mechanics of fatigue process.

Another method of tackling the fatigue problem is to apply fracture mechanics. Paris in 1962 (see ref. [12], p207) used concepts of fracture mechanics to demonstrate that the rate of change of crack length with the number of applied stress cycles was proportional to the difference in stress intensity factor, ΔK_I, computed at applied stress levels. Paris and Erdogan [87] later proposed the following equation:

$$\frac{da}{dN} = A(\Delta K)^m \tag{3.1}$$

Where:

da/dN = rate of change of crack growth with number of applied cycles
A,m = material constant
ΔK = change in stress intensity factor

Equation 3.1 suggests that the fatigue process is controlled through ΔK by the near-field stresses at the crack tip. As was the case in fracture, the area around the crack tip should be small enough so that it has a negligible effect on the total elastic deformations. However, because of the viscoelastic nature of bitumen binders, the approach described above needs to be modified. All viscoelastic materials undergo some from of time-dependent flow or, as in the case of polymers, even yield at some stress levels. These deformations are partly viscous and therefore dissipate energy. The energy dissipation approach has been used by various researchers to analyse fatigue in viscoelastic materials [53,88].

3.5.1 Bitumen Fatigue Life Relation

SHRP [12] obtained an empirical law for the fatigue properties of bitumen, similar to the power law for bituminous mixture fatigue. The fatigue properties of bitumen and asphalt concrete can be calculated from log-log plots of cycles to failure against the maximum applied strain by obtaining the slope K_2 and the intercept K_1 of the resulting straight line. The fatigue behaviour of bitumen can therefore be represented by the following empirical power law relationship [12]:

$$N_f = K_1(\frac{1}{\epsilon})^{K_2} \tag{3.2}$$

Where

ε = strain amplitude

N_f = number of cycles to failure

K_1, and K_2 = fatigue parameters

In equation 3.2, K_1 represents the number of cycles to failure corresponding to unit strain, which is found to be a function of loading rate and temperature. The parameter K_2 represents the rate of change of fatigue life with strain amplitude.

The analysis of asphalt concrete fatigue data has also indicate that fatigue can be represented by the following relationship that effects of both temperature and strain amplitude [12]:

$$N_f = k(\frac{1}{T})^{n_0}\exp(-\frac{m}{T})(\frac{1}{\varepsilon})^{k_2} \tag{3.3}$$

Where

K, n_0, m, and K_2 = constants

T = temperature

ε = strain amplitude

Comparing equation 3.3 with equation 3.2,

$$K_1 = K(\frac{1}{T})^{n_0}\exp(-\frac{m}{T}) \tag{3.4}$$

Equation 3.4 indicates that K_1 is a function of temperature and influenced by the rheological properties of bitumen, where as K_2 is independent of temperature.

3.5.2 Energy Dissipation Approach

Under the repeated loading that is used in fatigue testing, energy is dissipated with each loading cycle. In materials such as bitumen and bituminous mixtures, the energy dissipation results from viscoelastic and plastic flow mechanisms. In the energy dissipation criterion developed for fatigue, it is hypothesised that failure under cyclic loading occurs when the energy absorbed in each cycle in excess of a certain non-damaging amount accumulates to a critical value. The total dissipated energy obtained by summing the areas of hysteresis loops under cyclic loading is assumed to be a measure of fatigue damage [12].

3.6 Review of Bitumen Fatigue Tests

Since bitumens under service conditions experience cyclic variation of temperature and loading, there is a need to obtain fatigue properties of these materials under different loading, mixtures, materials and temperature combinations. A knowledge of fatigue properties is also important for specification purposes.

Current performance prediction tests such as ductility, force ductility and penetration reflect the viscoelastic behaviour of bitumen but do not describe its brittle behaviour [89]. The Fraass test characterises bitumen in its brittle state at temperatures as low as –30°C from which a brittle point temperature is obtained, however, it does not provide fundamental material properties. A literature review revealed limited fundamental work pertaining to fracture testing of pure bitumen.

3.6.1 Notched Beam Test

In 1967 Moavenzadeh [78] studied the fracture of bitumen. Three types of bitumen of different rheological properties were used in the study. The variables included three temperatures (-7°F, 0°F, +7°F) , three rates of loading (0.05, 0.10, 0.2in/min), and three degrees of ageing of bitumen (0, 3, 6 hours at 375°F). The basic specimen shape used was a notched, five inches long beam with a cross section measuring one-half inch by one-half inch. Figure 3.4 shows the notch geometry and the three point bending method of loading apparatus.

He concluded that at low temperatures the bitumen behaves as a brittle, amorphous material and the Griffith theory of brittle fracture can be applied to study the fracture behaviour of bitumen. Also the strain-energy release rate can be calculated for bitumen and its value is independent of the geometry of the specimen.

The work done by the applied load, which causes the specimen to deform, is stored in the specimen as strain energy. For an infinite plate of unit thickness containing a very flat, elliptical hole with a major axis of length 'c' perpendicular to a stress filed, σ, applied at the edge of the plate, the excess strain energy, u, of the plate over that of a plate without a crack would be [78]:

$$u = \frac{\pi \times c^2 \times \sigma^2}{E} \qquad (3.5)$$

per unit thickness of the plate. E is the elastic modulus of the material.

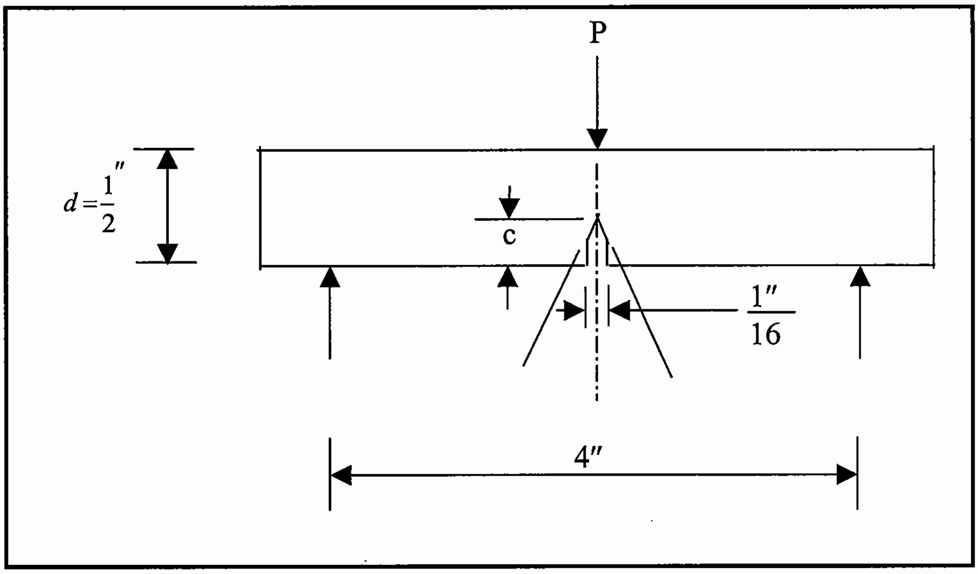


Figure 3.4. Notch geometry and method of loading the beam [78]

Another research done by Todd and Simon [90] studied the low temperature fracture of bitumen binder using specimens of three different sizes in three-point bend configuration, both with and without a notch. They concluded that it is possible to measure a stiffness modulus in three-point bending from a notched fracture toughness specimen by assuming that the specimen in unnotched. And there appear to be large differences in fracture toughness and fracture energy for unmodified and modified bituminous binder in the brittle regime.

3.6.2 Composite Beam Test

SHRP [12] evaluated the fatigue properties of bitumen by using specially fabricated apparatus, as shown in Figure 3.5. A composite beam specimen - an aluminium strip with a thin film of bitumen bonded to the under-surface (see Figure 3.6), is subjected to a four-point pulse loading. The deflection is controlled by the resistance of the aluminium beam, thus the strain in the bitumen is relatively constant throughout the test. Therefore, the test procedure may be considered a controlled strain test. The test is conducted in an environmental chamber that can controlled to ±0.1°C. The pulse loading is imposed on the sample pneumatically by a Bellofram air cylinder (see Figure 3.5). The compressed air supply line is connected to the inlet port of the accumulator.

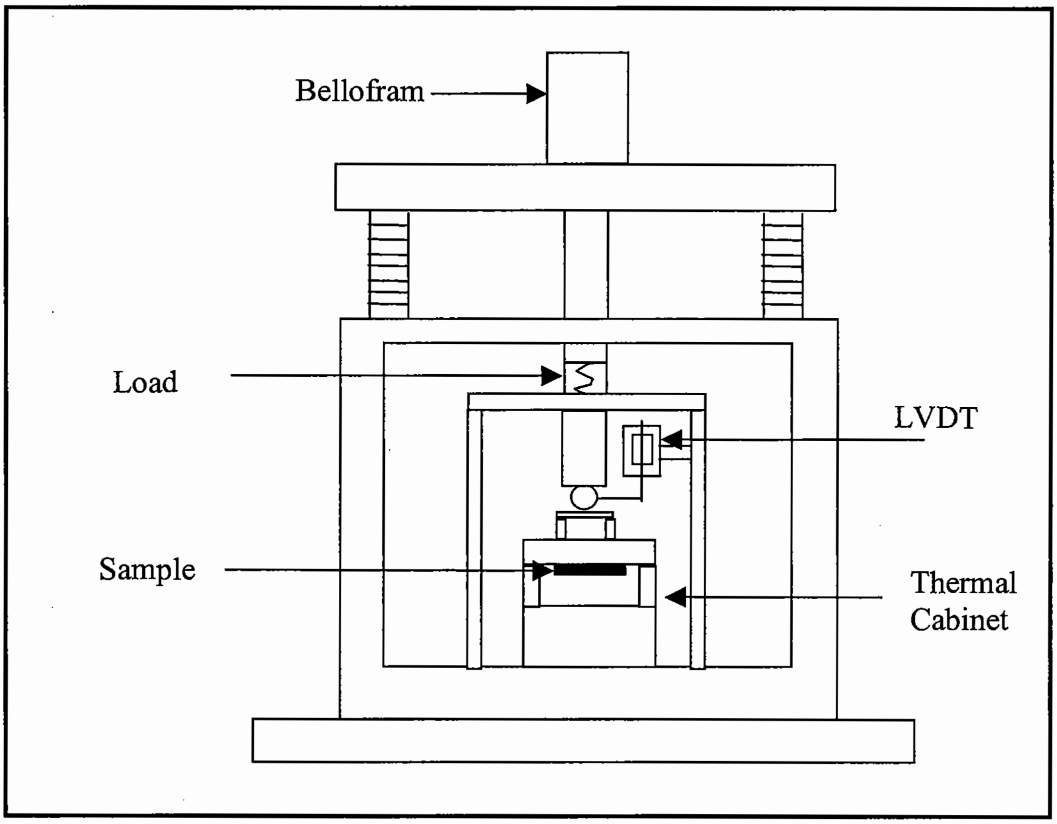


Figure 3.5 Fatigue testing configuration [12]

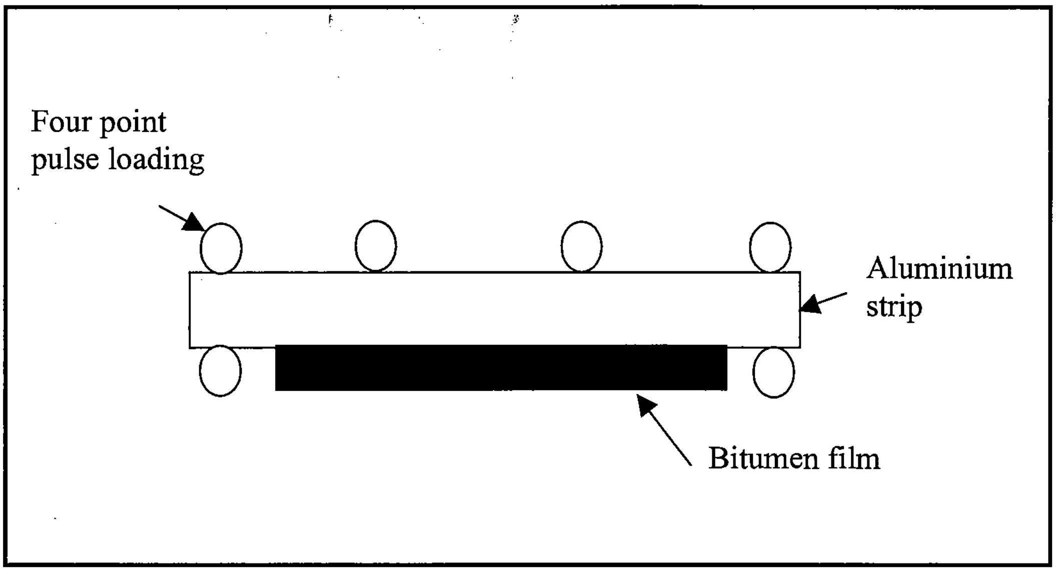


Figure 3.6 Composite beam specimen [12]

The duration of the load pulse is controlled by a pulser. The magnitude of the load pulse is measured by a load cell that is connected to the stem of the bellofram cylinder. A counter indicates the accumulated cycles of loading on the specimen. The entire assembly is enclosed in a thermal chamber, the temperature of which can be varied within a temperature range –40°C to 60°C using a digital temperature controller. The vertical midpoint deflection of the beam specimen is measured by a Linear Variable Differential Transformer (LVDT). The test was terminated when a hairline crack appeared on the surface of each of the specimens tested and the cycles of loading corresponding to this point were noted.

3.6.3 Dynamic Shear Rheometer (DSR)

A time sweep using the dynamic shear rheometer (DSR) has been proposed as a test method developing load-associated fatigue information for bituminous binder [88]. In the Superpave binder specification, a single point measurement, $G^*\sin\delta$ at a fixed frequency and temperature, is used as the criterion for fatigue. The dissipated energy or work per cycle for controlled strain cyclic loading can be determined from this equation [91]:

$$W = \pi.\tau_0.\epsilon_0.\sin(\delta) \tag{3.6}$$

where

W = energy dissipated

τ_0 = maximum stress amplitude

ε_0 = maximum strain amplitude

δ = phase angle

Stress, strain and complex modulus are related together by the following equation:

$$\tau_0 = \epsilon_0.G^* \tag{3.7}$$

from equation 3.6 and 3.7 the dissipated energy W is given by:

$$W = \pi.\epsilon_0^{\ 2}.[G^* \times \sin(\delta)] \tag{3.8}$$

Based on field data, dissipated energy, which is related to G*sinδ, was selected by the SHRP team as a specification criterion for fatigue of the bitumen binder (see Figure 3.7) [92].

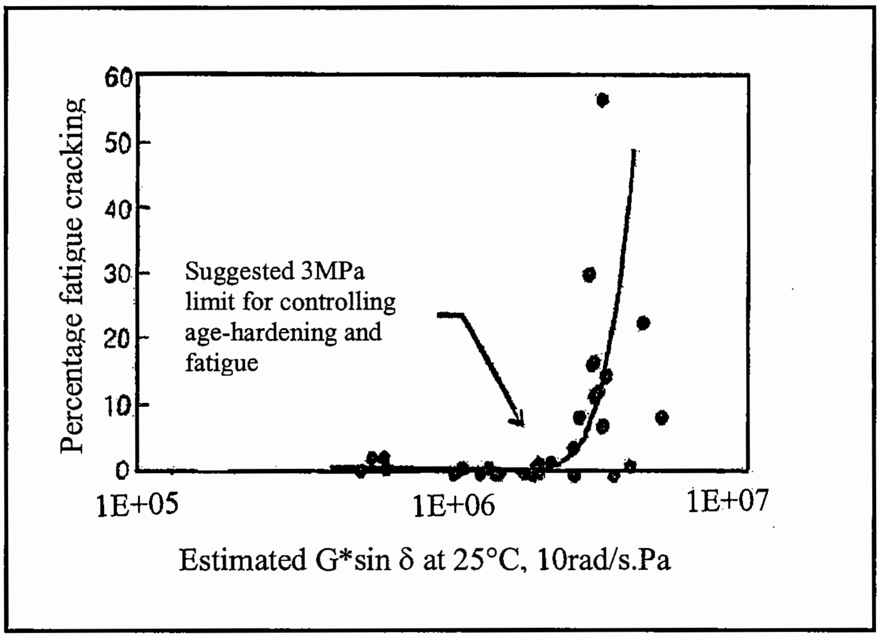


Figure 3.7 Analysis of a site performance results using G*sinδ [92]

A research was done by Cooley, et al [49] to try to explain the relationship between dissipated energy and volume of compacted filler using the DSR on a bitumen-filler mortar. As the percentage bulk volume increases, the stiffness of the mortars also increases. This indicates that as the percentage volume of the filler increases, the ability of the hot mix asphalt pavement to withstand fatigue cracking decreases.

Another research done by Soenen and Eckmann [93] has evaluated the DSR as a test method for characterising the fatigue of bituminous binder. Their tests were conducted at 50Hz in controlled stress and at temperatures in which the initial modulus was above 20MPa. They reported that, during repeated shearing, hairline cracks developed and propagated toward the centre of the sample as long as the modulus was sufficiently large. As the modulus decreased, they also observed edge flow. They concluded that no correlation with the SHRP fatigue parameter $G^*\sin(\delta)$ could be established.

Anderson, et al in 2001 [94] concluded that the DSR is not a suitable method for characterising the true fatigue behaviour of bitumen binder because of the edge fracture phenomenon or the instability flow dominates.

A joint research was done by Nynas bitumen and Laboratoire Central des Ponts et Chaussees (LCPC) to investigate the fatigue of binders and to compare the results with laboratory fatigue properties of the corresponding mixtures [3]. They used the DSR under constant strain and at a frequency of 25Hz for testing the binder and the two point bending test on trapezoidal samples for the mix.

They found a relation between the strain levels at which fatigue occurs after one million cycles in the binder and in corresponding mix fatigue tests (for non polymer modified binders) as shown in Figures 3.8 and 3.9. The researchers concluded that binder fatigue tests are well related to laboratory mix fatigue behaviour but neither of these tests seem to be relevant for in-situ behaviour since neither test correctly simulates in-situ stresses.

Another research [95] to support this finding found that there is poor correlation between the fatigue indicator G*sin(δ) and the laboratory measured fatigue performance of mixture produced with modified binder.

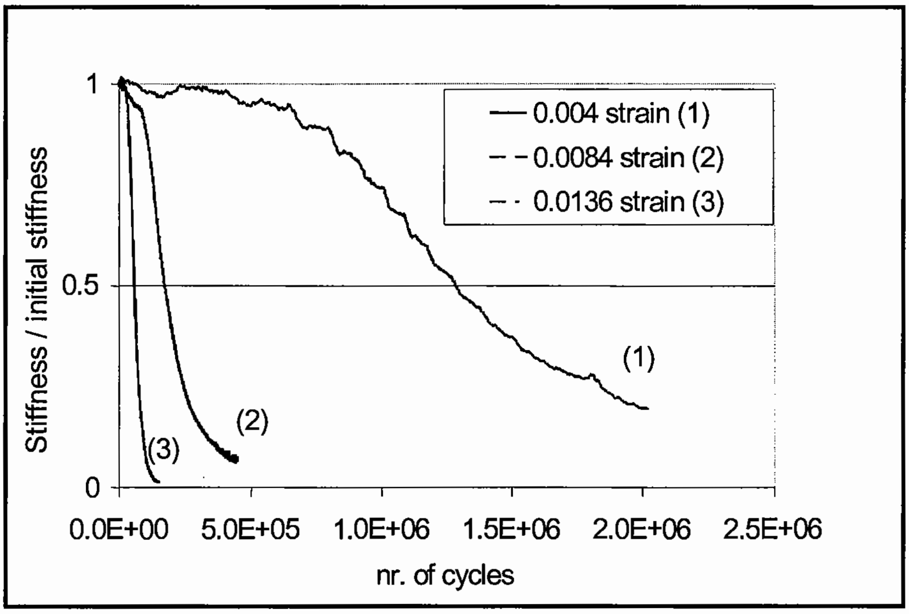


Figure 3.8 Binder fatigue test of 50/70 bitumen at three strain levels [3]

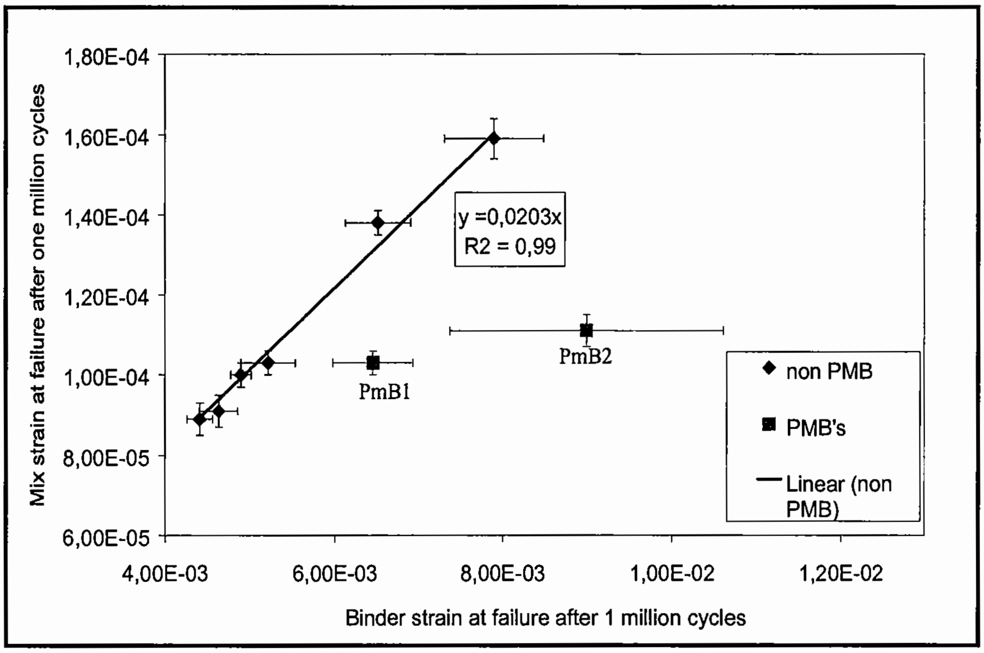


Figure 3.9 Comparison of binder and mix strain level at which the fatigue life is equal to one million cycles [3].

To resist fatigue cracking, it is desirable to have bitumen that is low stiffness and elastic. Therefore low values of (G*) and (δ) are considered beneficial. As G*sin(δ) increases, the potential for fatigue cracking in an asphalt concrete pavement will also increase. The Superpave specification is for G*sin(δ) not to exceed 5000kPa [92].

3.6.4 Repeated Local Fracture of Bitumen

A specific test named Repeated Local Fracture of Bitumen has been designed in France, to study the bitumen's part in the fatigue of bituminous mixture due to loss of cohesion [5,7,96]. A film of bitumen, between two steel convex protuberences (see Figures 3.10 and 3.11), simulating two aggregate particles, is submitted to successive tensile stresses at a constant strain rate, and at a constant temperature of 0°C. The sample holder has been designed such that, when no loading is applied to the sample, the bitumen thickness in the centre of the specimen can be set from about 0.1 to 0.3 mm. The test is performed in a climatic chamber to reach low temperature. The loading of the sample is performed at a constant strain rate and the resulting force is measured.

The resulting load curves showed sudden drops. These drops were more numerous when the maximum applied strain increased and they were explained by a sudden fracture within the binder. This hypothesis was confirmed by the fracture faces obtained [96]. Figure 3.12 shows the result of the local fracture test carried out on 50/70 grade bitumen at temperature 0°C, over a total loading time of 4 s and initial thickness 220μm. According to the setting of the test, the curves can present one or more discontinuities. The researchers said that the number of discontinuities seem to correspond to the number of propagated cracks during the test, and this can be observed by inspection of the fracture faces after total separation of the specimen at the end of the test.

The test seems to study the mechanisms of fatigue and healing, but no relationship between the results of the test and other properties of the binder, or fatigue behaviour of the mixes has been proposed.

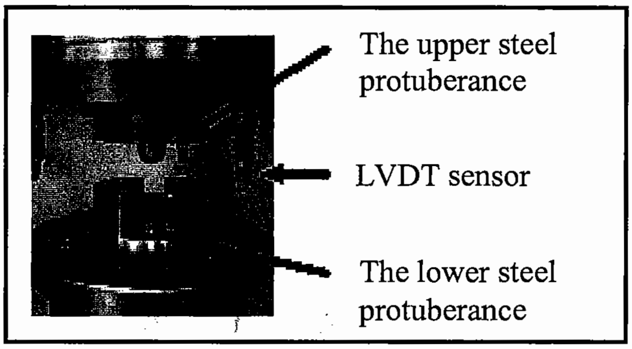


Figure 3.10 Local fracture fatigue test apparatus [5]

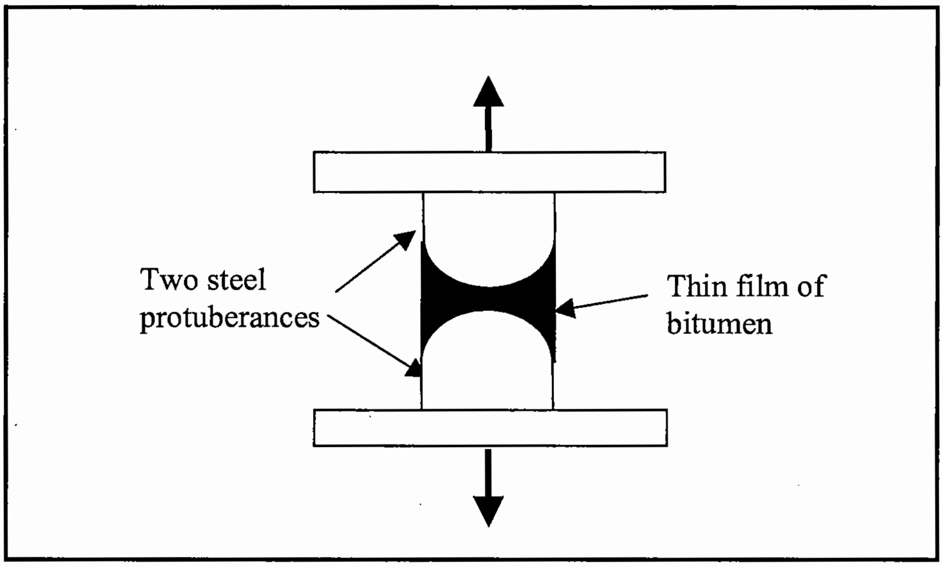


Figure 3.11 Schematic presentation of Local Fracture test apparatus [5,7,96]

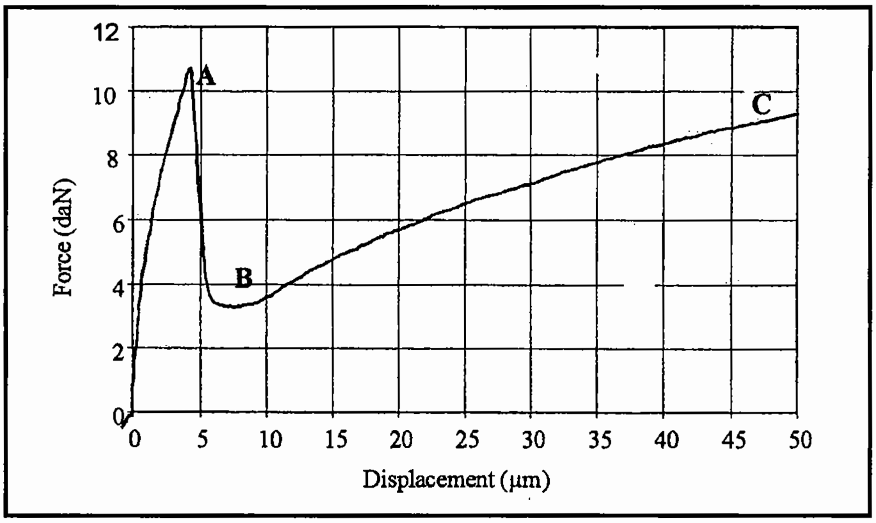


Figure 3.12 Local fracture test at temperature 0°C for 50/70 bitumen [7]

3.7 Review of Mixture Fatigue

There are two aspects of asphalt material properties relevant to pavement design: firstly, the load deformation or stress-strain characteristics required for the analysis of the structure, secondly, the performance characteristics which determine the mode of failure.

The fatigue resistance of bituminous mixtures is generally defined as the ability of the mix to respond to repeated traffic loads without significant cracking [97]. Under traffic loading the layers of flexible pavement structure are subject to continuous flexing. The magnitude of the strains is dependent on the overall stiffness and nature of the pavement construction but analysis, confirmed by measurements, has indicated that tensile strains in order of 20-300 microstrain occur under a standard wheel load. Under these conditions, the possibility of fatigue cracking exists [1].

Top-Down Cracking (TDC) is one of the recognised modes of deterioration of pavements. The diversity of climates and traffic conditions for which this phenomenon is observed has led to the suggestion of numerous possible causes for this type of cracking (complex tyre/pavement surface interaction, ageing of binder, thermal cycles, structure and construction, etc). Top-down cracking in flexible pavements is normally observed in thick bituminous layers [98,99].

The traffic induced stresses at the surface of the pavements may have horizontal tensile component due to tyre shape and pressure. The induced tensile surface stresses under the new wide-base tyres (super single) are predicted, under certain circumstances, to be much larger than at the underside of the base [98].

Freitas and Periera [100] reported that initiation and propagation of TDC follows three stages. The first stage consists of a single short longitudinal crack appearing just outside the wheel path. Over time, other cracks develop 0.3 to 1m away and parallel to the original crack as shown in Figure 3.13. Finally, cracking evolves into a third stage, where the parallel cracks are connected via short transversal cracks.

Thermal cracking is normally associated with a significant temperature variation. The literature identifies two forms of thermal cracking [101]. The first failure mode identified is commonly called low temperature cracking, and the second phenomenon is thermal fatigue. This second form of thermal cracking is similar to traffic load-induced fatigue cracking, with daily temperature cycles equivalent to traffic-induced loading of the pavement. During periods of cooling, bituminous mixtures try to contract; however, due to the constraint of the layer within the road structure, the materials cannot change their length. As a result, tensile stresses develop in the layer, which may result in a transverse crack if the tensile strength of the asphalt is exceeded. This type of cracking normally starts at the surface and grows towards the bottom of the layer because the temperature gradient gives greater contraction at the surface.

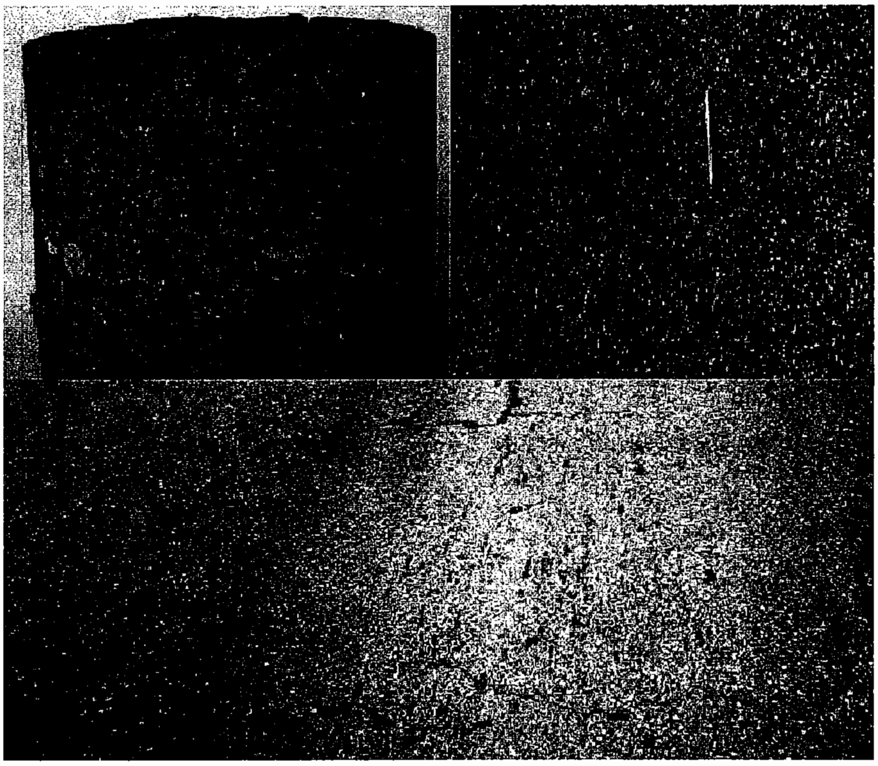


Figure 3.13 Different views of top-down cracking [100]

It is usually found that a reasonably linear relationship exists between applied stress and cycles to failure when plotted on a logarithmic basis [1,4,102]. This relationship can be expressed as:

$$N = k_1 (\frac{1}{\sigma})^{k_2} \tag{3.9}$$

where

N = geometric mean value of the number of cycles to failure obtained at a particular stress level

σ = maximum amplitude of the applied stress

k_1 = a constant which determines the position of the line

k_2 = slope factor of the fatigue line

The coefficients k_1 and k_2 are determined by linear regression analysis.

It is generally found that over the range investigated, there is a relationship between fatigue life and applied strain which can be considered linear and be expressed as:

$$N = k_3 (\frac{1}{\epsilon})^{k_4} \tag{3.10}$$

where

N = geometric mean value of the number of cycles to failure obtained at a particular strain level

ε = maximum amplitude of the applied strain

k_3 = a constant which determines the position of the line

k_4 = slope factor of the fatigue line

Again the coefficients k_3 and k_4 are determined by linear regression analysis, both are dependent on the composition and properties of the mix.

Further, in relation to field performance, the laboratory and the field results often do not agree, leading to shift factors being introduced [103]. The shift factors proposed by various researchers vary a great deal and are dependent on the bituminous mix characteristics, testing methods, test and field conditions, including mode of loading and rest periods between load applications. As an example [103], to allow for the above mentioned differences, the life at any strain level may be increased by a factor of up to 100. This is explained as a factor 5 for rest periods and 20 for crack propagation.

The selection of the appropriate test method and test conditions is to some extent related to the analytical approach used to interpret the output from the test. There are many ways of analysing the fatigue results:

- Conventional stress or strain Vs number of cycles to failure
- Dissipated energy approach
- Fracture mechanics approach
- Damage mechanics approach

Conventional strain-based fatigue approaches do not consider the effects of flaws (micro-cracks or macro-cracks) in asphalt mixtures (i.e. stress redistributions induced by geometry changes are not considered). Furthermore, realistic cyclic loading conditions and healing are not considered.

Majidzadeh in 1969 [72] reported that fracture mechanics may provide a rational approach to describe fatigue behaviour in an asphalt mixture. Conventional fracture mechanics begins with the assumption that there are inherent flaws or micro-cracks in a material. It also provides a physical interpretation of damage (i.e. a crack length has a physical meaning). It assumes that crack growth is continuous, and crack growth rate is generally determined by fracture mechanics.

Viscoelasticity and continuum damage mechanics [104] offer a more fundamental explanation of damage than traditional fatigue approaches. Micro-cracks in asphalt mixtures are analysed under realistic loading conditions and the influence of healing is considered, but this approach is incapable of specifically addressing the mechanics of crack propagation (i.e. once a crack develops, the system is no longer a continuum). In addition, damage mechanics does not provide a realistic physical interpretation of damage: failure is generally assumed to coincide with 50% reduction in stiffness.

Most fatigue tests are carried out under simple loading conditions that apply continuous cycles of loading of a particular magnitude, at a particular temperature and speed. In practice, the material is subjected to a succession of load pulses of varying size and with varying time interval between pulses, depending on the details of traffic, and the temperature and speed conditions are changing continually [97].

A reduction of the stiffness on the asphalt mixture under fatigue tests has been reported by Di Benedetto and Soltani [105] due to heat generated in the specimen (an increase of 1.3°C). They conclude that a reduction in stiffness more than 30 % is due to heating during phase I of testing.

Di Benedetto, Soltani and Chaverot [106] carried out tension-compression triaxial fatigue tests on cylindrical samples at 10°C and 10Hz. They found that they could clearly distinguish three phases of the test. The first phase corresponds to a phase where heating and fatigue occur, but heating has the predominant effect. The temperature is stabilized during the phase II. Generally, an important and rapid degradation is observed in the third phase. They concluded that the most important problem is the heating of the sample due to dissipated energy accumulating after each cycle as shown in Figure 3.14.

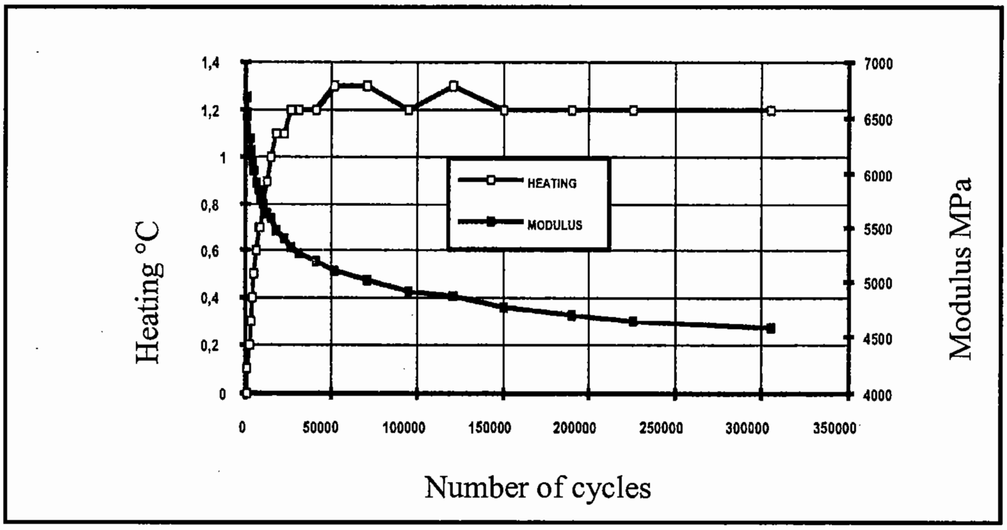


Figure 3.14: Evolution of the modulus and the temperature (probe inside the sample) during a fatigue test at 10°C and 10 Hz [106]

Similar results were found by Didier et. al. [86]. They reported that bituminous materials exhibit a viscoelastic behaviour which leads to energy dissipation during each loading cycle of fatigue tests. This dissipated energy is transformed into heat leading to a temperature increase inside the specimen, and a 1°C rise may cause a 5% loss of stiffness for usual conditions of temperature and frequency of fatigue tests.

Baaj, Di Benedetto and Chaverot [107] also report the same phenomenon and state that the reduction of stiffness in the first phase is not only due to the generated heat but that a third phenomenon (thixotropy or a so-called local phenomenon) has an important role. The same phenomeneon has been reported by Xiaohu, Soenen and Redelius [108] for pure bitumen tested in the DSR test.

The different researches agreed that stiffness reduction of the asphalt mixture is partially due to the generated heat, and a 1°C rise in temperature may reduce up to 10% of the stiffness.

3.7.1 Testing Modes

The characterisation of fatigue behaviour of a bituminous mixture generally involves the application of either a controlled stress or a controlled strain. It has been suggested by many researchers [4, 70, 97] that controlled stress mode of loading is applicable to thick bituminous pavements (>150mm), whilst controlled strain tests are applicable to thinner bituminous pavements (<50 mm).

In controlled stress testing, a constant repeated load amplitude is applied and the displacement of the specimen is monitored in order that the strain resulting from the applied load can be calculated. The formation of the initial crack is quickly followed by rapid crack propagation and complete specimen failure [4]. The fatigue life is usually defined as the number of load repetitions when the elastic stiffness of the mixture is reduced by half its original value i.e. when the strain amplitude is doubled [97]. In this mode of testing, the crack propagation phase is very rapid because the strain amplitude increases as the number of cycles of constant load increases and the specimen fractures not long after fatigue crack initiation. In controlled stress testing the measured lives do not usually contain a large amount of crack propagation time and the end point of a test is very definite, i.e. complete fracture of the specimen.

In a constant strain test the strain amplitude remains constant. As damage accumulates the stress required to maintain the initial strain gradually decreases after crack initiation as the stiffness of the mixture effectively decreases. In this mode of testing, when the crack initiates, to maintain the same strain level, a reduction in stress occurs around the crack. Because of the reduction in stress which occurs as the crack length increases, crack propagation is relatively slow compared with the controlled stress mode of failure, therefore controlled strain gives a greater fatigue life.

The design criterion for fatigue is generally the maximum tensile strain. Laboratory fatigue tests are based on applying a constant load or displacement amplitude for a specific number of repetitions. Ideally, the test may be continued until the specimen actually fails in fatigue for a range of loads/displacements, temperatures and stiffnesses. However many researchers have defined fatigue failure by monitoring changes in the stiffness modulus, with the failure indicated when the stiffness of the sample drops to 50% of the initial stiffness (typically measured after 100-200 load repetitions [70,109]). Initial stiffness has also been measured at 500 cycles [97]. Khalid [110] used a reduction of specimen stiffness to 10% of its original value to define the number of cycles to failure in controlled stress fatigue tests whereas a value of 50% was used for controlled strain tests.

In controlled stress tests higher stiffness results in longer life, whereas in controlled strain mode stiffness has the opposite effect [102]. If the tests were conducted at various initial stress amplitudes and temperatures, plots of stress or strain Vs number of cycles will vary and, for a given fatigue life, at higher temperatures the initial stress is lower but the initial strain is higher both in constant strain and constant stress mode.

3.7.2 Asphalt Mixture Fatigue Tests

The application of laboratory determined fatigue lives to predict actual pavement performance in practice is a complex problem but is likely to yield conservative results as simple continuous cycles of loading neglect the beneficial effects of rest periods which occur in practice between axle loads [1]. Longer lives in practice are also likely because of the lateral distribution of wheel loads in the wheel track, and a degree of crack propagation will have to occur before pavement performance is adversely affected. Because of these problems, laboratory fatigue lines have to be calibrated to correlate with actual pavement performance and the calibration factor is likely to depend on environmental and loading conditions [102].

Realistic stress conditions can be reproduced in the laboratory only with great difficulty. In practice, the stresses are applied three dimensionally: horizontal and shear stresses occur on planes that are perpendicular to those shown in Figure 3.15. As the wheel passes over the element, these stresses change with time and this is shown in Figure 3.16 [1]. With the possible exception of the Hollow Cylinder Apparatus, it has not yet been possible to reproduce this complex stress regime accurately in the laboratory.

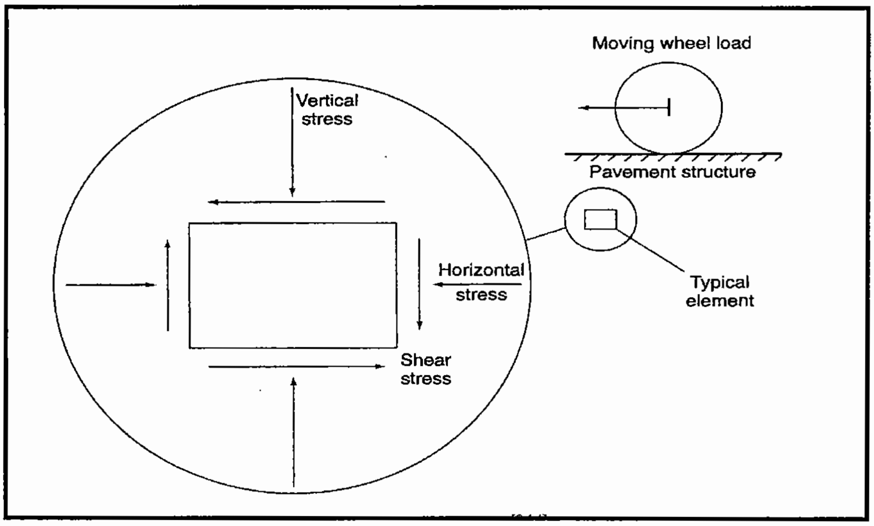


Figure 3.15 Stresses on an element in a pavement [1]

A number of test methods have been developed for the evaluation of fatigue characteristics of asphalt. These include: 4-point bend loading of a flexural beam [70,110,111], two point trapezoidal, direct axial tension/compression, diametral [4,110,112,113], triaxial [106] and wheel tracking [114], see Figure 3.17. Characteristics of these test methods vary in terms of load pulse wave form, loading configuration, stress distribution within the specimen, specimen geometry, etc.

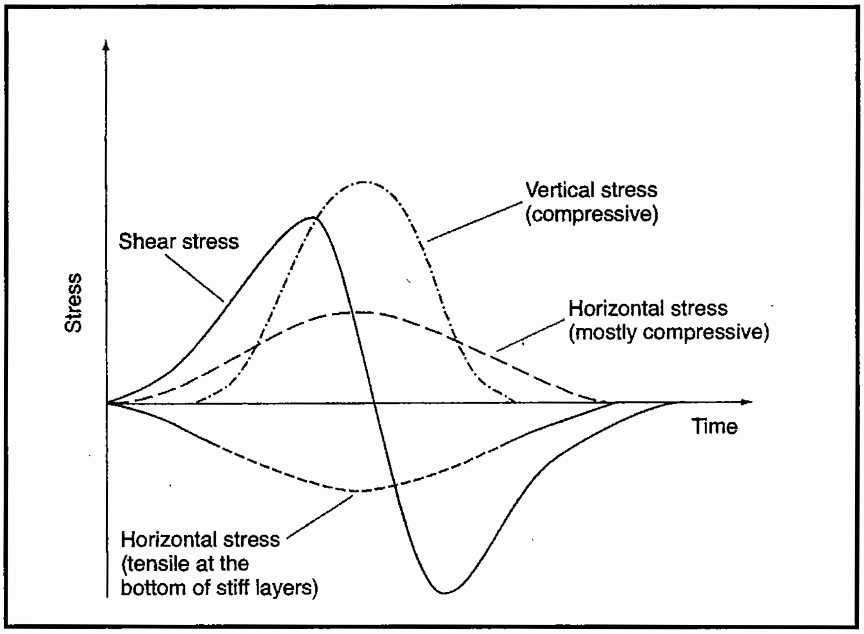


Figure 3.16 Stresses induced by a moving wheel load [1]

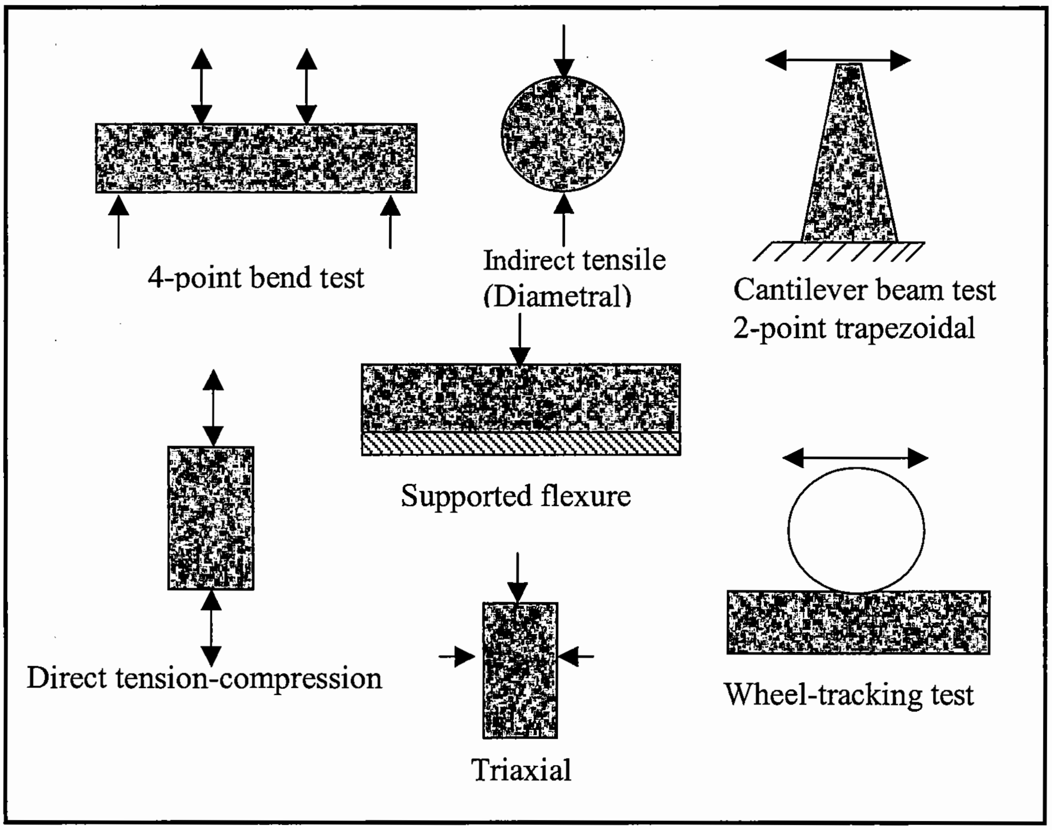


Figure 3.17 Tests for measuring dynamic stiffness and fatigue

The general categories of asphalt mixture fatigue tests are [115]:

1. Triaxial with a direct relationship between fatigue life and stress/strain developed by testing similar to direct axial testing but with confinement.
2. Diametral with a direct relationship between fatigue life and stress/strain developed by applying pulsating loads to cylindrical specimens in the diametral direction.
3. Simple flexure with a direct relationship between fatigue life and stress/strain developed by subjecting beams to pulsating or sinusoidal loads in either a third or center-point configuration; and trapezoidal cantilever beams subjected to sinusoidal loading.
4. Supported flexure with a direct relationship between fatigue life and stress/strain developed by loading beams or slabs that are supported in various ways to directly simulate in-situ modes of loading.
5. Direct axial with a direct relationship between fatigue life and stress/strain developed by applying pulsating or sinusoidal loads.
6. Wheel-tracking tests, including both laboratory and full-scale arrangements, with a direct relationship between the amount of cracking, the number of load applications, and the measured and/or computed stress/strain.

SHRP [115] carried out a survey of the most recognised and widely used methods for measuring fatigue properties of asphalt mixtures. They ranked the methods based on governing parameters including test simplicity, ability to simulate field conditions, and the applicability of the test results for designing a pavement for fatigue adequacy. The survey showed that repeated flexure tests were ranked highest followed by diametral indirect tensile tests in second place.

In flexure tests a sinusoidal loading pattern is usually applied to evaluate fatigue properties, whereas in the indirect tensile test only a repeated pulsating load without stress reversal can be applied.

3.7.2.1 Indirect Tensile Fatigue Test (ITFT)

This is a diametral fatigue test conducted by repeatedly loading a cylindrical specimen with a compressive load which acts along the vertical diametral plane. This loading configuration develops a reasonably uniform tensile stress in the specimen perpendicular to the direction of the applied load and along the vertical diametral plane see Figure 3.18. The indirect tensile fatigue test uses a repeated controlled stress pulse to damage the specimen, using the same machine as for ITSM [1]. The cumulative vertical deformation against number of load pulses is plotted. The duration of the load pulse and the magnitude of the stress are specified by the user. Standard test conditions and requirements for the ITFT according to BS DD 226: 1996 [67] are:

Stress level	between 50 and 600 kPa (determined by the user)
Target rise time	120 ms
Test temperature	20°C

The test is simple to conduct and is considered by some to be an effective method for characterizing materials in terms of "fundamental" properties. A number of investigators have utilized this test for material evaluations and pavement analyses [4,116,117].

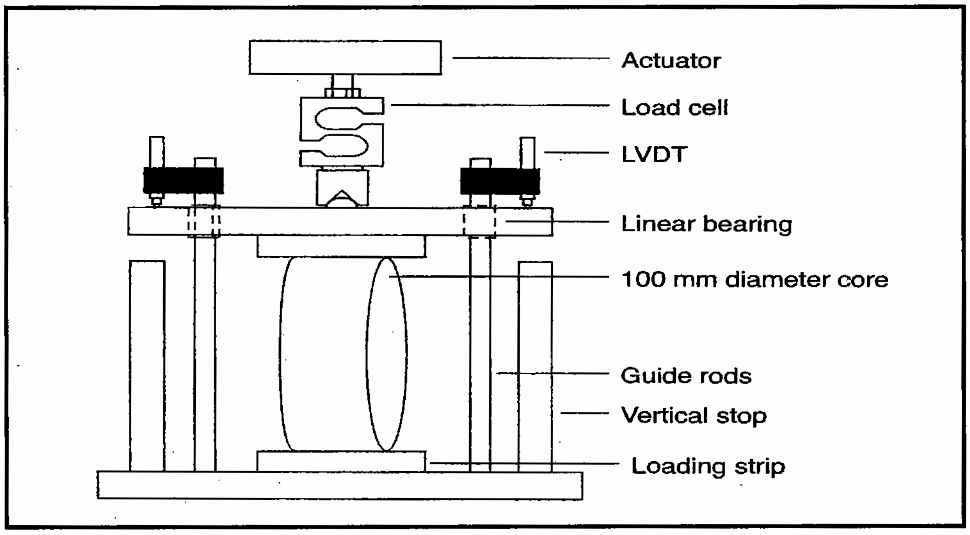


Figure 3.18 Schematic of the indirect tensile fatigue test [1]

Figure 3.19 (a) shows the repeated load Indirect Tensile Stiffness Modulus (ITSM) test in which only a few load applications are applied and the specimen suffers no significant damage. Figure 3.19(b) involves the same test regime but repeated loading is continued until failure by cracking is achieved. This is the Indirect Tensile Fatigue Test (ITFT) [16]

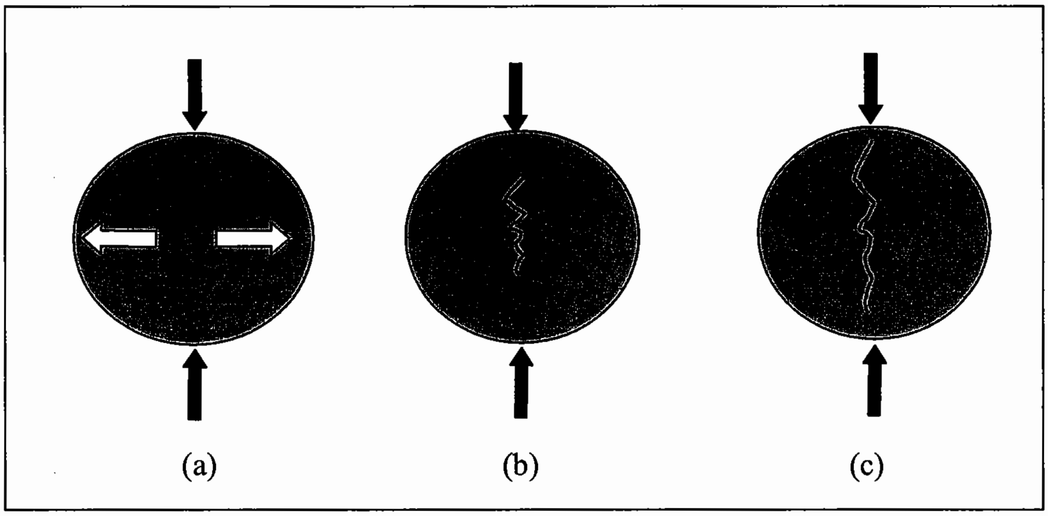


Figure 3.19 Indirect Tensile Test (a) repeated load stiffness, (b) fatigue, (c) monotonic splitting

3.7.2.2 Four-Point Bending Test

This is one of the flexural tests developed to study the fatigue of asphalt mixtures. The advantage of these tests is that they simulate, to some extent, the stress state in the lower part of the bound layer (large horizontal tensile stresses combined with small vertical compressive stresses). The most commonly used beam flexural test is the 4-point loading in which the load is applied at locations spaced at third points along the beam length. The analysis of the results is based on elastic beam theory in which a linear stress-strain relationship is assumed [115].

A simply supported asphalt mixture beam specimen is subjected to a controlled load or deflection under 4-point loading; see Figure 3.20 [115]. Loads are applied at two locations as shown in Figure 3.20 to insure a uniform bending moment through the mid span of the beam. Both controlled-load (stress) and controlled-deflection (strain) tests have been performed.

In 1969 Santucci and Schmidt [70] carried out research to study the effect of bitumen properties on the fatigue resistance of asphalt mixtures. They used a 4-point bend test under strain-controlled mode and tested beams with dimensions 15×1.5×1.5 inches (380×38×38mm). They found that there was a good correlation between bitumen properties and asphalt mixture fatigue resistance.

In another research [29] it was observed that, in 4-point bend tests on asphalt mixture specimens, several cracks nucleated on the tensile surface of specimens. It was not possible to achieve a single dominant crack. The results suggest that distributed damage rather than a single dominant crack may be the true damage mechanism.

In 2004 research done by Remakant and Animesh [111] used 4-point bend tests to study fatigue crack propagation in asphalt mixes according to Indian specifications. They conducted the tests under strain-controlled mode using beam samples of size 380×80×80mm with a pre-fabricated crack of size 2mm×4mm at the bottom. They found that conventional bituminous concrete mix (according to their standard) showed satisfactory fatigue performance in terms of its fatigue life compared to other non-standard mixes.

Other research done by Hartman and Gilchrist [9] used 4-point bend tests to evaluate fatigue of asphalt mixtures with the aid of image analysis. They tested a beam of rectangular section (305×45×50mm dimensions). They showed that a fracture mechanics approach can satisfactorily quantify the relative superiority of different asphalt mixes and offers time and expense advantages over in-situ and full scale tests.

They also found the occurrence of micro-cracking in front of the macro-crack as shown in Figure 3.21, as well as small opening tensile cracks, which aligned in a direction perpendicular to bending, observed at the bottom of the tensile face of the beam between the loading clamps, as shown in Figure 3.22. An important observation from their test was that the propagation of cracking was largely within the bitumen mortar and tended to deviate around aggregate particles.

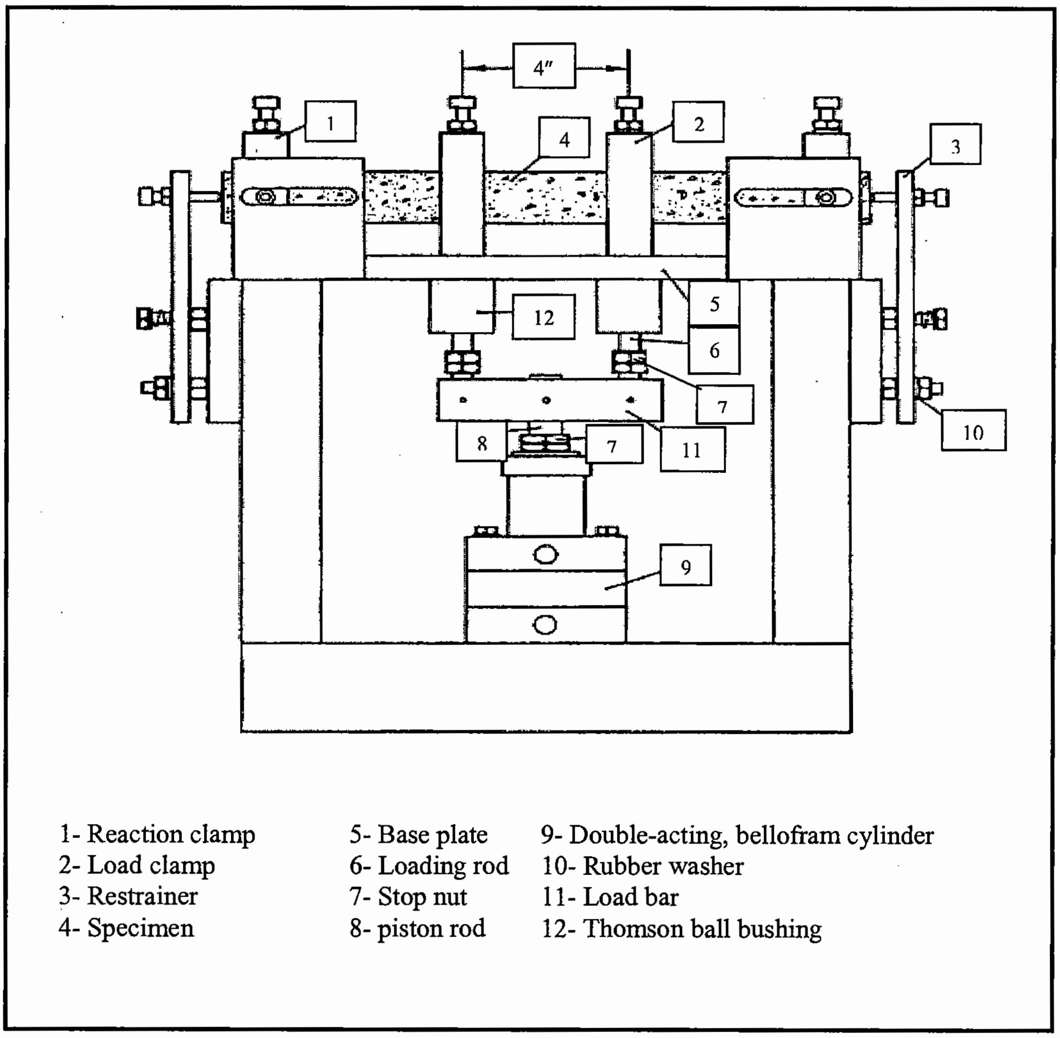


Figure 3.20 Four-point bend test apparatus [115]

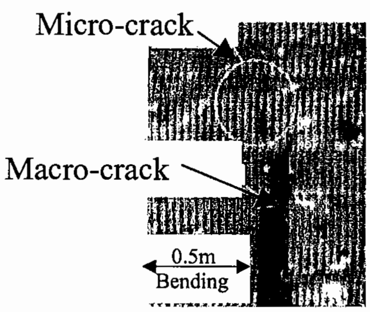


Figure 3.21 Image taken from side face of beam specimen indicating crack tip with micro-cracks in front of macro-crack [9]

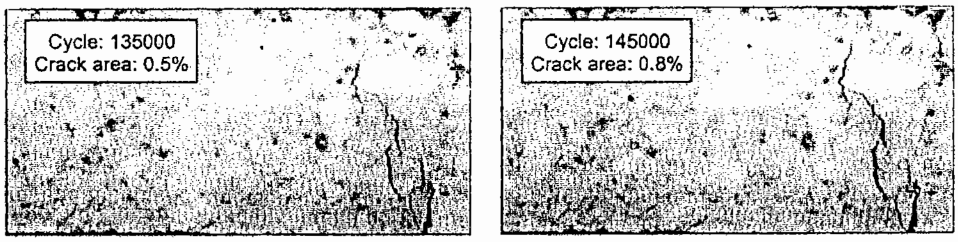


Figure 3.22 Example of tensile cracks on bottom tensile face of beam specimen between central loading clamps [9]

3.8 Summary

Different tests have been used to study fatigue of binder, the most interesting of which are the Dynamic Shear Rheometer (DSR) and the Repeated Local Fracture Test. A number of test methods have been developed for the evaluation of fatigue of asphalt mixtures. The 4-point bend test is thought to be the best at simulating the field condition as a uniform bending moment is applied over the mid span of the beam.

These are the key points:

- Cracks in a typical asphalt composite often run through the binder rather than through the aggregate particles or aggregate/binder interface.
- Different approaches have been used to characterise the fatigue properties. For bitumen fatigue an empirical law was found, similar to the bituminous mixture law.
- At low temperatures the bitumen behaves as a brittle and the theory of brittle fracture can be applied to study the fracture behaviour of bitumen.
- As the percentage volume of the filler increases, the ability of the hot mix asphalt pavement to withstand fatigue cracking decreases.
- Binder fatigue tests (using DSR) are well related to laboratory mix fatigue behaviour but neither of these tests seem to be relevant for in-situ behaviour since neither test correctly simulates in-situ stresses.
- In relation to field performance, the laboratory and the field results often do not agree, leading to shift factors being introduced.
- Three phases of the fatigue tests can be distinguished. The first phase corresponds to a phase where heating and fatigue occur, but heating has the

predominant effect. The temperature is stabilized during the phase II and a rapid degradation is observed in the third phase.

- Stiffness reduction of the asphalt mixture is partially due to the generated heat, and a 1°C rise in temperature may reduce up to 10% of the stiffness.

4

Experimental Work Using The Direct Tension Test

4.1 General

This chapter describes tests performed using the INSTRON Direct Tension Test (DTT) to study the tensile and fracture characteristics of bitumen and bitumen-filler mortar. 50 pen grade bitumen and bitumen-filler mortars (filler: limestone, cement, gritstone and sewage sludge ash) were tested over a range of temperatures and strain rates. The test methodology used with the DTT and bitumen-filler mortar manufacturing procedure will be described. The results of the DTT performed on the bitumen and bitumen-filler mortars with then be presented and discussed.

Brittle fracture of bitumen binder occurs under conditions of high strain rate or low temperature, where the material is glassy. Bituminous mixtures for pavements are made of bitumen binders and aggregates. Although bitumen is predominantly considered, the binder holding the aggregate together, the actual product used to connect large size aggregate particles, is the bitumen-mineral filler mastic.

At the minimum pavement temperature, where thermal cracking occurs, bitumens are very brittle, low toughness materials. The strain to failure at these temperatures is typically 1.0% or less and the failure stress is typically less than 1 MPa [118]. The characterisation of bitumen-filler mortar is essential to improve the understanding of the response and performance of bituminous pavements. Several researchers have used changes in penetration, viscosity, ductility, and softening point temperatures to show the effect of fillers in the mixture mortar [20,64,119]. The Superpave binder tests were developed as part of the SHRP research for characterising bitumen. Though these tests were not specifically developed to evaluate a filled system (bitumen-filler system), a number of studies have tried to use these tests to characterise bitumen-filler mortar [49,93].

4.2 Direct Tension Test (DTT) [AASHTO:TP3-00]

The Direct Tension Test (DTT) device was introduced to test bitumens and determine their failure properties such as the stresses and strains at failure. The test procedure is used in the SHRP specifications to ensure that the strain at failure at the minimum pavement design temperature is greater than 1.0 % as shown in Figure 4.1 [28]. At a temperature and loading rate where the strain to failure is less than approximately 1.0 %, bitumen acts as a brittle material [14].

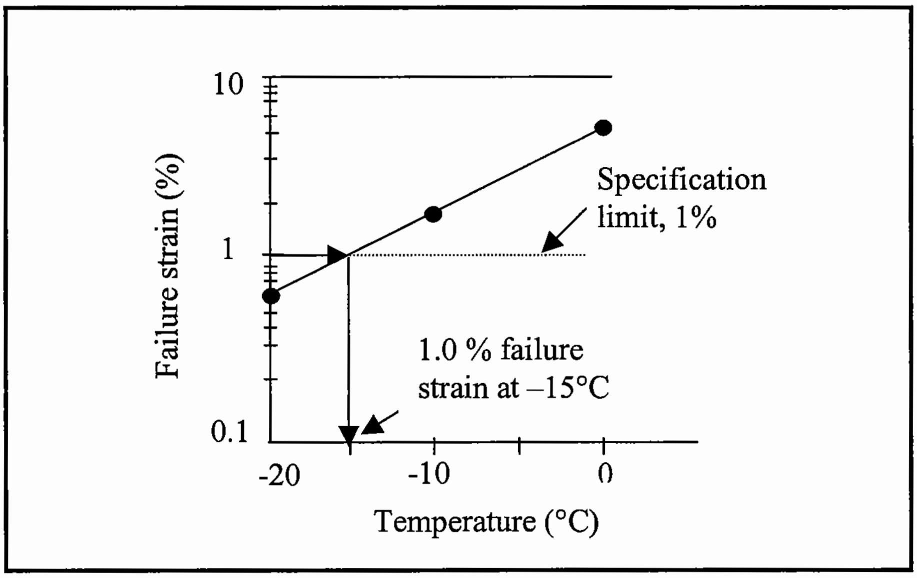


Figure 4.1 Reporting Direct Tension Specification test data [12]

Failure is defined as that point on the stress-strain curve where the load reaches a maximum value as shown in Figure 4.2. When the specimen exhibits brittle-ductile behaviour, the specimen will rupture, and the maximum load will be very obvious. At higher temperatures, where the bitumen does not rupture but extends to larger strains, the maximum load will often be difficult to determine. When the specimen does not rupture, the failure point is defined in this thesis as the point corresponding to the maximum stress value.

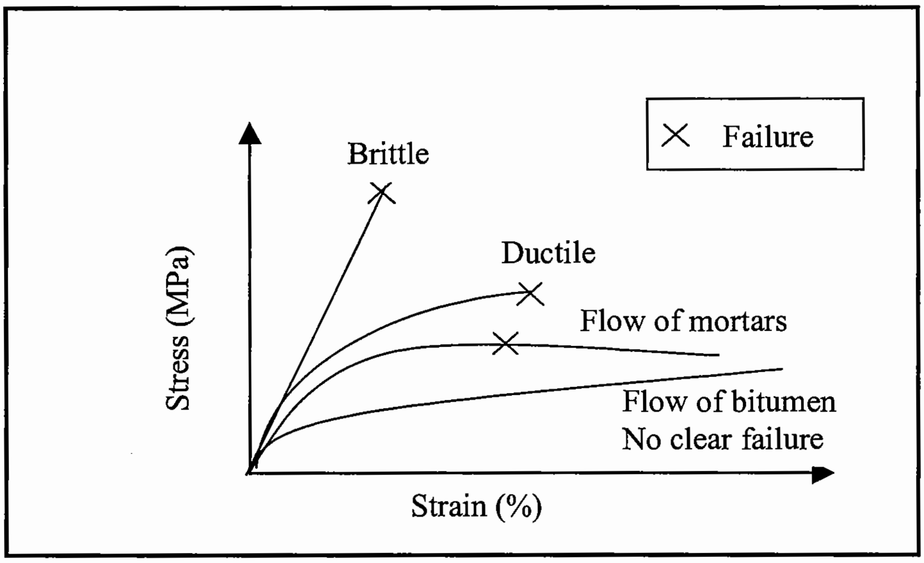


Figure 4.2 Typical stress versus strain curves

4.2.1 Apparatus

4.2.1.1 Direct Tension Test System

The direct tension test system consists of:

1- A closed feedback loop, displacement-controlled tensile loading machine equipped with temperature control. The load is measured with a load cell having a capacity of 500N and sensitivity of 0.1N. The load and elongation are monitored using a data acquisition system. Loading is effected by a grip mechanism pulling directly in tension in the plane of the specimen.

2- A specimen gripping system: The gripping mechanism consists of specially designed pins precisely centred in the top and bottom loading grips. The top and bottom grips are identical. The lower grip remains stationary while the upper grip is pulled at a specified rate. The gripping system is completely submerged in a cooling fluid.

3- Temperature control system: The DTT temperature control consists of two chambers. The remote chamber is where laboratory air is introduced, dried, and cooled. This conditioned air is then circulated in a closed loop to the second chamber, which is mounted on the testing frame. The temperature control range for the cooling chamber ranges from -40°C to +25°C with a temperature stability of ±0.1°C.

4- Load and elongation measuring and recording devices: Load is measured by a load cell having a capacity of 500N. Specimen elongation is measured between the grips by a displacement transducer. The load and elongation is monitored with the data acquisition system.

4.2.1.2 Specimen Moulds

The moulds were manufactured from aluminium. A release agent along with Teflon-coated release papers were used to prevent bitumen from adhering to the aluminium mould sides and back plate when making the specimen. Each specimen requires approximately 2g of bitumen [28]. The specimen is 100mm in overall length including the plastic end inserts. The end inserts are each 30mm long by 20mm wide. The purpose of the end inserts was to minimise the amount of bitumen required for a test specimen and to provide a means for gripping the test specimen. The bitumen portion of the specimen is 40mm in total length and the necked down portion of the specimen is 18 mm long with cross-sectional dimensions of 6mm wide by 6mm thick. The necked-down section widens at each end at a radius of 12mm to provide a gradual transition to the end inserts. The necked-portion of the specimen is sufficient to develop a region of relatively uniform tensile stresses. Figure 4.3 shows the direct tension test geometry and specimen mould.

4.2.2 Sample Preparation

Bitumen has to be heated in an oven at a temperature not exceeding 165°C until it becomes sufficiently fluid to pour. The sample should be stirred from time to time to aid heat transfer and ensure uniformity. The moulds should be cleaned and placed on a flat horizontal surface.

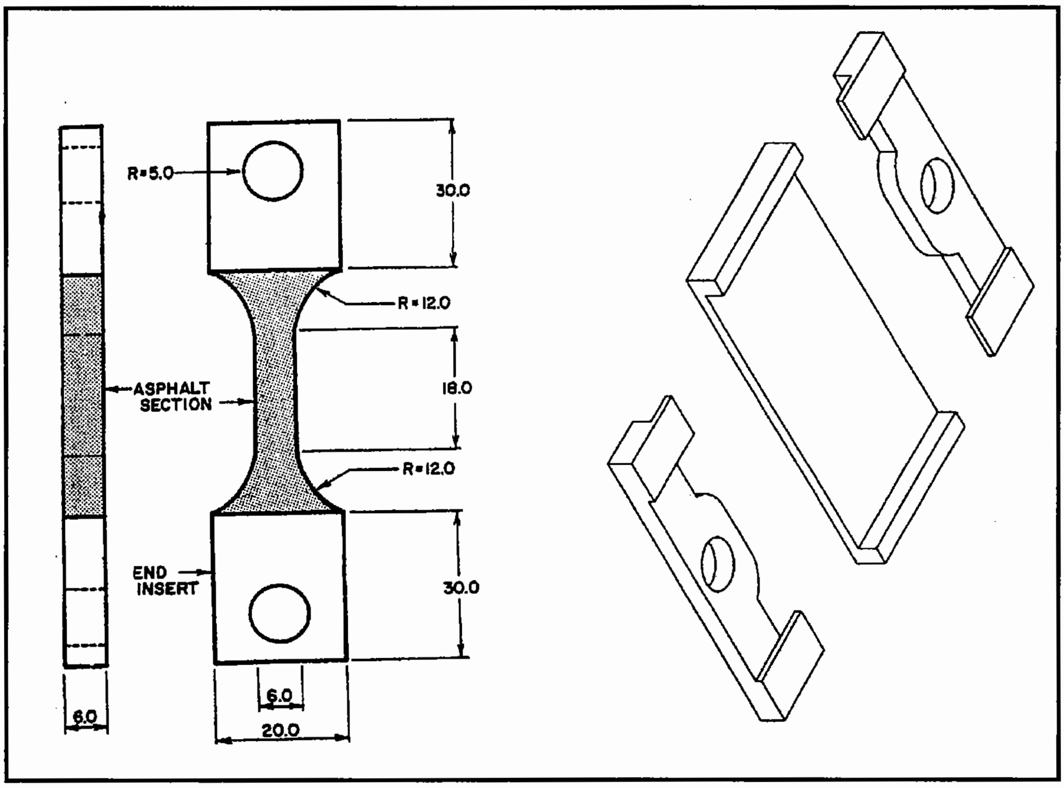


Figure 4.3 Direct tension test geometry and the aluminium mould (all dimensions in mm) [28]

Teflon-coated release papers are placed at the bottom of the mould to prevent bitumen from adhering to the mould bottom. The end inserts are placed at both ends of the mould, and the side moulds coated with the release agent. Pouring of bitumen into the mould is started from one end insert and moves slowly along the mould to the other end in a single pass. Pouring should be stopped when the bitumen is slightly higher than level with the top of the mould. After pouring the bitumen into the moulds, the entire assembly is allowed to cool at room temperature.

The specimen is allowed to cool to room temperature for about one hour. Then the specimen is trimmed off with a heated flat cheese cutter so that the bitumen is flush with the top of the mould. Then the test specimen is cooled at room temperature again to stiffen the binder.

The control chamber should be set to the required test temperature before releasing the specimen from the mould. The specimens are then removed from the mould and placed in the chamber to be ready for testing after 60±10 minutes.

4.2.3 Test Procedure

The specimen is placed in the chamber for 60±10 minutes. After conditioning, the specimen is mounted on the pins. The desired deformation rate should be selected, the specimen is loaded to failure, and the test data is saved. After testing, the bitumen portion of the spent specimen is discarded. The end inserts are cleaned and wiped and a detergent solution is used to remove any oil residue left by the spirit cleaner. The end inserts are reused.

4.2.4 Calculations of Strain

The strain in the gauge length of the bitumen specimen with dog bone geometry is measured by dividing the grip separation as measured by the mechanical extensometer by a length called the effective gauge length (L_{eff}) [28]. The effective gauge length (L_{eff}) is defined as the length of the same cross section as the gauge length that gives the same strain energy as that of the entire dog bone bitumen specimen between the plastic inserts [120]. Raj Dongre, D' Anglelo, and McMahon [120] computed L_{eff} for a bitumen specimen using the following formula:

$$L_{eff} = L_G + W_G \left\{ \frac{2\beta}{\sqrt{\beta^2 - R^2}} \tan^{-1}\left[\frac{(\sqrt{\beta^2 - R^2})\tan\left(\frac{\theta}{2}\right)}{\frac{W_G}{2}} \right] - \theta \right\} \tag{4.1}$$

where

L_G = length of the constant cross section, gauge length (18mm),

W_G = width of the constant cross section (6mm)

β = $(W_G/2) + R$ (15mm)

R = radius of the transition region (12mm)

θ = $\sin^{-1}(X_{max}/R)$, in radians

X_{max} = length of the transition region, that is, $X_{max} = [(L_A - 2W_G)\text{-}L_G]/2$, and

L_A = length of the bond-to-bond bitumen section (39.91mm)

Therefore, L_{eff} is the effective gauge length (33.8mm).

$$\text{Strain, \%} = \frac{\delta_t}{L_{eff}} \times 100 \tag{4.2}$$

where

δt = is the grip-to-grip elongation.

L_{eff} = is the effective length

4.3 Testing Conditions

The direct tension test is normally carried out, according to the specifications, at a strain rate of 1mm/min (3%/min) and at temperatures below 0°C (where the bitumen fails in a brittle-ductile mode). Also the test is not specified for the bitumen-filler mortar. As an objective of this research is to investigate the fatigue characteristics of pure bitumen and bitumen-filler mortar over a wide range of temperature and strain rates, the direct tension test has been carried out on the bitumen and bitumen-filler mortar under the following conditions:

- **Temperatures**: bitumen is a viscoelastic material; at higher temperature it behaves like a fluid and at lower temperatures like a glass. Because of the difficulties of testing the bitumen specimen at very high temperatures using the direct tension test, the tests have been carried out using the temperatures: -10°C, -5°C, 0°C, +5°C, and +10°C.
- **Strain rate**: To study the effects of strain rate on the fracture characteristics of bitumen and bitumen-filler mortar, different strain rates have been used: 2mm/min (5.9%/min), 5mm/min (14.8%/min), 10mm/min (29.6%/min),

20mm/min (59.2%/min), 50mm/min (147.9%/min), and a number of specimens tested at 200mm/min (591.7%/min).

4.4 Materials

4.4.1 Bitumen

50pen bitumen, supplied by Nynas Bitumen, was used for the direct tension testing, which is typical of types used for base course in UK. The testing program for the bitumen included the following tests:

- Conventional tests (Penetration, softening point)
- Direct tension test.

The penetration test and softening point test were carried out for the bitumen under investigation. Table 4.1 presents the tests results.

Table 4.1 Bitumen properties

Bitumen Type	Penetration (dmm)	Softening Point (°C)
50 pen	50	50.2

4.4.2 Fillers

Mineral fillers, as defined in this research, are those materials, 65% of which pass through a No. 200 standard sieve. This definition is not applicable to sewage sludge ash, which contains large particles. Four types of mineral filler were used for the investigations of fracture characteristics of bitumen-filler mortar. They are:

- Limestone filler: typical filler used in bituminous mixture
- Cement: sometimes used in mixture as filler
- Gritstone filler: natural mineral filler from crushed stone
- Sewage sludge ash: alternative filler, different from the above

The gradation of the fillers was determined using a hydrometer and sieve analysis tests [BS 1377-2:1990]. Specific gravity tests were carried out to identify the fillers according to BS 812. The fillers were further defined by measuring their bulk density. Bulk density is defined as weight divided by volume. The bulk densities were obtained by tapping a 100cm^3 glass cylinder filled with dry and loose filler until no further consolidation was observed. The physical properties of the different fillers are tabulated in Table 4.2. The values obtained reflect the bulking properties of the different materials in the dry state. Figure 4.4 shows the particle size distribution for the different fillers under investigation.

Table 4.2 Physical properties of fillers

Filler type	**Specific gravity**	**Bulk density (g/cm^3)**	**Void content in bulk sample (%)**
Limestone	2.74	1.032	62.3
Gritstone	2.65	0.8695	67.2
Cement	3.18	0.932	70.7
Sewage sludge ash	2.64	0.7719	70.8

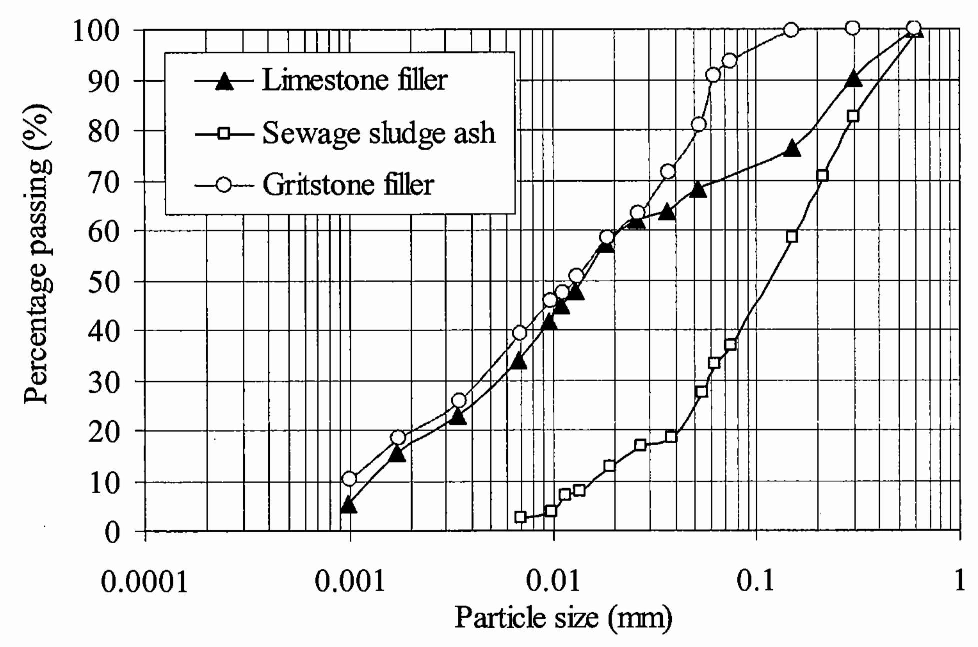


Figure 4.4 Particle size distribution curves for the different filler types

4.5 Bitumen-Filler Mortar

The filler plays an important role in determining the properties and behaviour of the bituminous paving mixture. The amount of filler material is specified as a percentage of the weight of the mix, and becomes part of the mixture design. To characterise bitumen-filler mortar and to study the effect of mineral filler on the behaviour of bitumen under the tensile test, a bitumen-filler mortar with different ratios has been used. The bitumen/filler ratio in a typical real mixture usually is about 50/50% to 65/35% by mass. The bitumen-filler mortars were divided into three groups according to filler concentration:

- **Bitumen/limestone mortar**: Filler concentrations used were: 5%, 15%, 35%, 50% and 65% by mass (percentages by volume: 2%, 6%, 17%, 27% and 41% respectively). In one test series, a coarse limestone (retained on 75μm sieve) was used with 35% concentration.
- **Bitumen/gritstone (or cement) mortar**: Filler concentration was 50% by mass (percentage by volume: 28% for gritstone and 24% for cement)
- **Bitumen/sewage sludge mortar**: Filler concentration was generally 15% by mass (percentage by volume is 6.3%) although some specimens tested contained 35% by mass.

4.5.1 Bitumen-Filler Mortar Preparation

As mentioned before, the DTT is designed for pure bitumen testing. Therefore, to test the bitumen-filler mortar, a special procedure for making the specimen is required. Because there is no standard procedure for making bitumen-filler mortars, the following procedure has been used to prepare all bitumen-filler mortar specimens, in order to reduce variability in the test results:

- Place a filler sample in an oven at 110±5°C for drying to a constant weight.
- Place the bitumen into an oven at 160±5°C, until it reaches a uniform temperature of 160°C. Stirring is needed from time to time.
- After preheating the bitumen and filler samples, remove each from its respective oven.

- Place the correct quantities of the dried filler sample and the heated bitumen into a sample container and place it in an oven at 160°C.
- Place the sample container on an electric hot plate at a temperature of approximately 160°C and hand mix with a spatula.
- The mortar is heated until the air bubbles escape, and stirred to mix the filler particles, which would otherwise settle at the bottom of the container. Care must be taken to prevent loss of fines during mixing. Stirring of the mixture is necessary to produce a homogeneous specimen, but this can introduce air bubbles. Great care must therefore be taken to minimise voids in the specimen.
- When the mortar (blended bitumen and filler) appears visually homogenous, the mortar will be ready for pouring into the testing moulds.

The specimens prepared by this technique were found to be reasonably homogenous. The homogeneity of the specimen was checked by selecting some specimens and sectioning the specimen into two parts along its length and the density of each section measured, no significant difference between the two parts (see Table 1 in the appendix)

4.6 Repeatability of the Test

The achievement of good repeatability is a particular concern in testing bitumen. The Superpave direct tension test produces a stress-strain curve up to the point of failure. The repeatability of the data collected has two aspects: the stress-strain curve as well as the failure values (failure stress and failure strain) must be consistent and repeatable. To check repeatability, two temperatures (–10°C and +10°C) were selected in order to test the bitumen in both brittle and ductile modes, and 2mm/min (5.9%/min) was used.

The repeatability of the test can be seen from the six repeat tests shown in Figures 4.5 and 4.6. The variability of the failure values (failure stress and failure strain) was found to lie in the range 10% to 18% at the low temperature –10°C, and was 5% at +10°C. Table 4.3 shows results of statistical analysis on the 50pen bitumen at these temperatures and at a strain rate of 5.9%/min.

Table 4.3 Typical variability of failure stress and failure strain values (50pen bitumen at 5.9%/min)

Parameter	**Temperature**	
	−10°C	**+10°C**
Failure stress, MPa	3.195	0.074
Standard deviation, MPa	0.32	0.004
Coefficient of variation (C.V.), %	10.2	5.4
Failure strain, %	1.72	- *
Standard deviation, %	0.32	- *
Coefficient of variation (C.V.), %	18.6	- *

* No clearly defined failure point

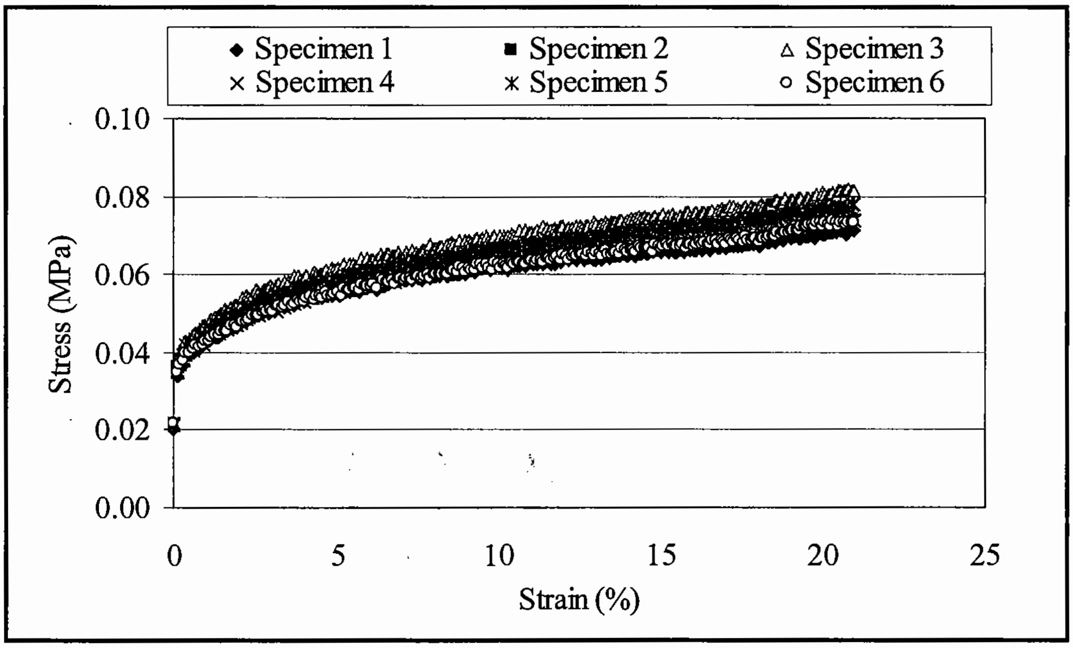


Figure 4.5 Repeatability of DTT for pure bitumen tested at temperature +10°C and 5.9%/min

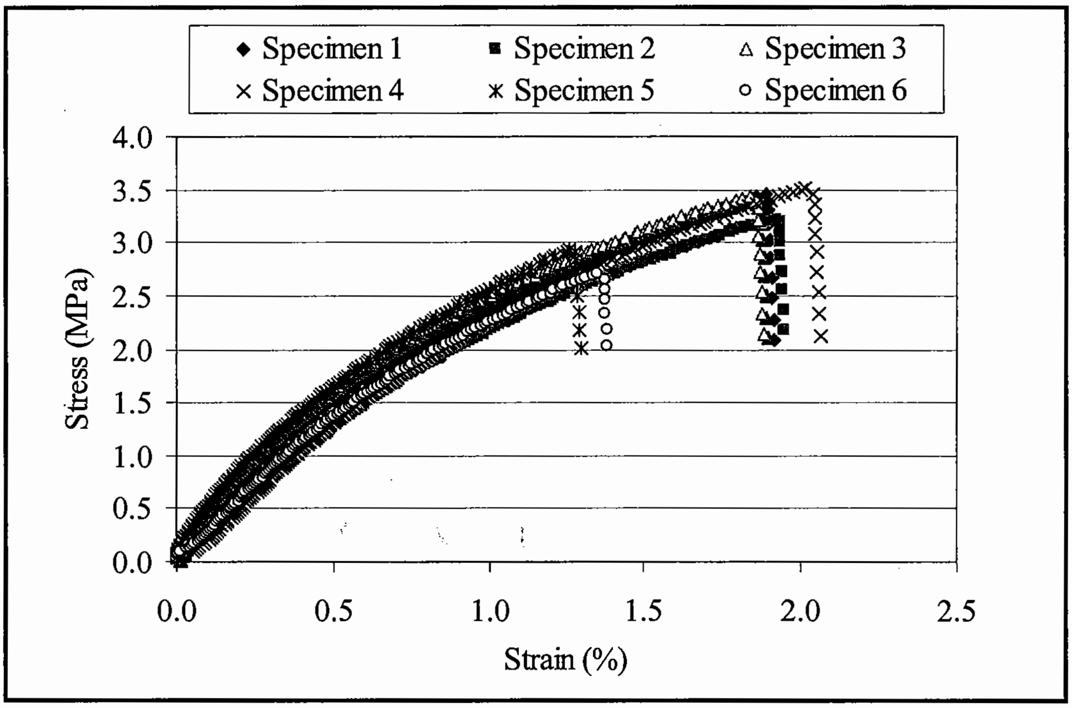


Figure 4.6 Repeatability of DTT for pure bitumen tested at temperature –10°C and 5.9%/min

4.7 Test Results

4.7.1 Results for Pure Bitumen

The direct tension test has been carried out on a 50 pen grade bitumen in the temperature range from –10°C to +10°C. Different strain rates were used: 2, 5, 10, 20, and 50mm/min (5.9, 14.8, 29.6, 59.2, 147.9%/min). A strain rate of 200 mm/min (591.7%/min) was used at temperatures of +5°C and +10°C to check the possibility of bitumen fracture at higher strain rate (see Appendix A).

True stresses (related to the instantaneous cross-sectional area) and true strains (obtained by adding up the increments of strain) have been used to express stress and strain in order to take into account the reduction of the cross-sectional area during testing. When testing at high temperature, the specimen was found to stretch up to the machine's limit without failure. This means that there is great reduction in the cross-sectional area, which makes the use of uncorrected stress and strain unrealistic.

Figures 4.7 to 4.9 show the results of stress and strain for pure bitumen tested at temperatures of +10°C, 0°C and –10°C (for other temperatures results see Appendix)

The failure point can be detected easily at –10°C and –5°C at all strain rate levels, hence defining the maximum stress and the corresponding strain. All the specimens broke in the middle portion of the dog bone (within the 18mm gauge length). But at +5, and +10°C and low strain rate it was not possible to identify the maximum stress and the corresponding strain at which the bitumen failed, due to the flow of the materials at these temperatures. However, the specimen fractured when tested at a temperature of 0°C and a very high strain rate (147.9%/min).

At +10°C, the specimens do not reach fracture at any strain rate levels and there is no peak in the stress. Generally, the stress at failure increased as the strain rate increased except for the specimens tested at a temperature of –10°C, where the stress at failure decreased as the strain rate increased.

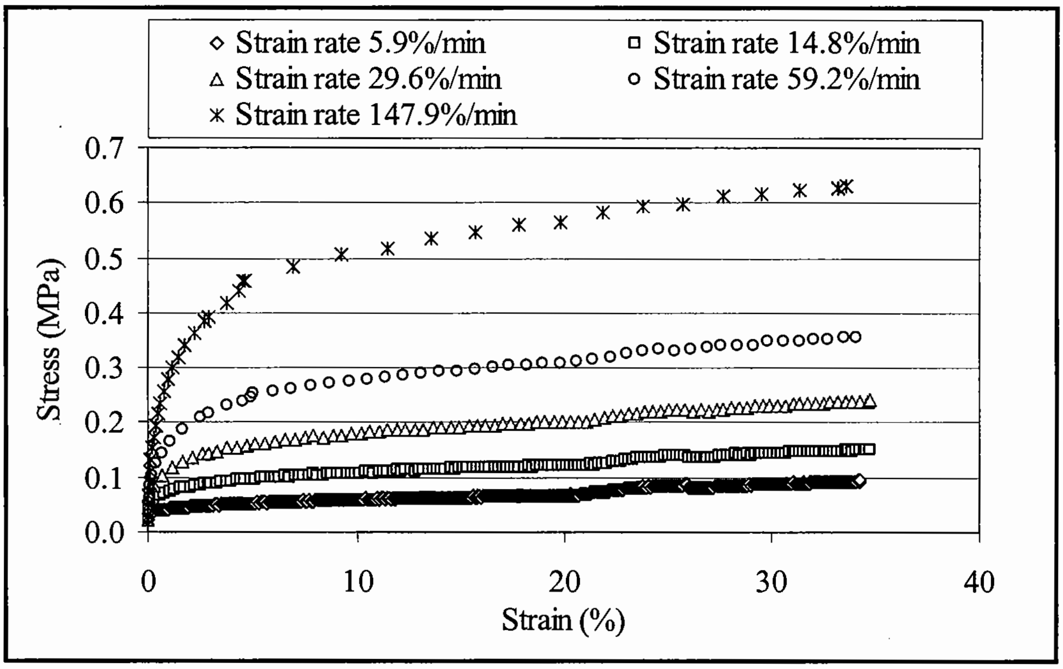


Figure 4.7 Stress Vs strain for pure bitumen tested at temperature +10°C

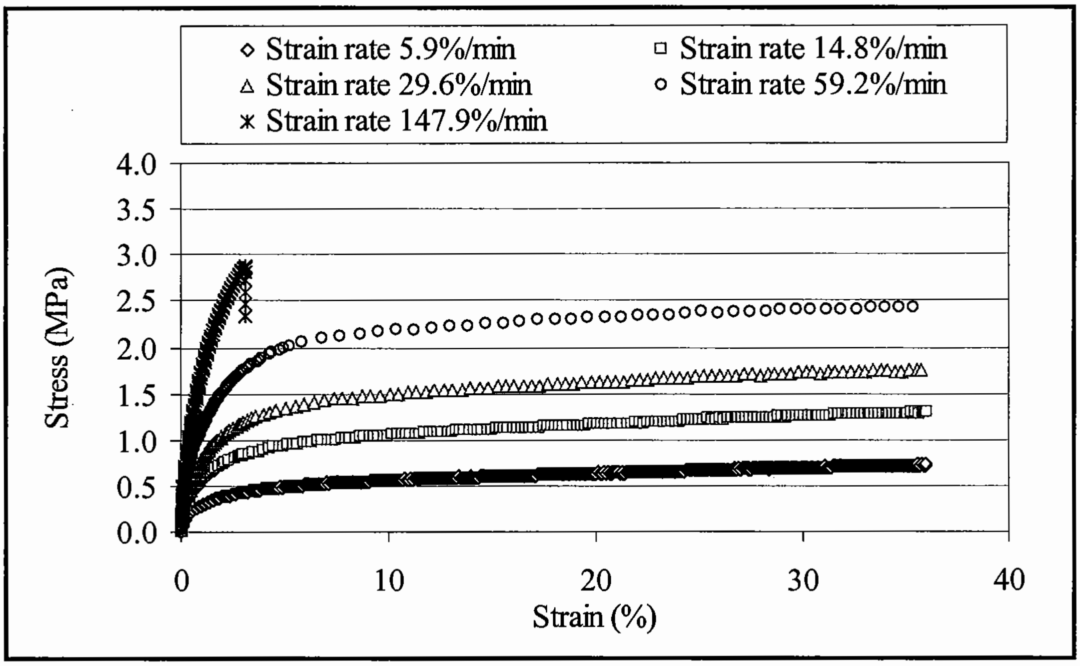


Figure 4.8 Stress Vs strain for pure bitumen tested at temperature 0°C

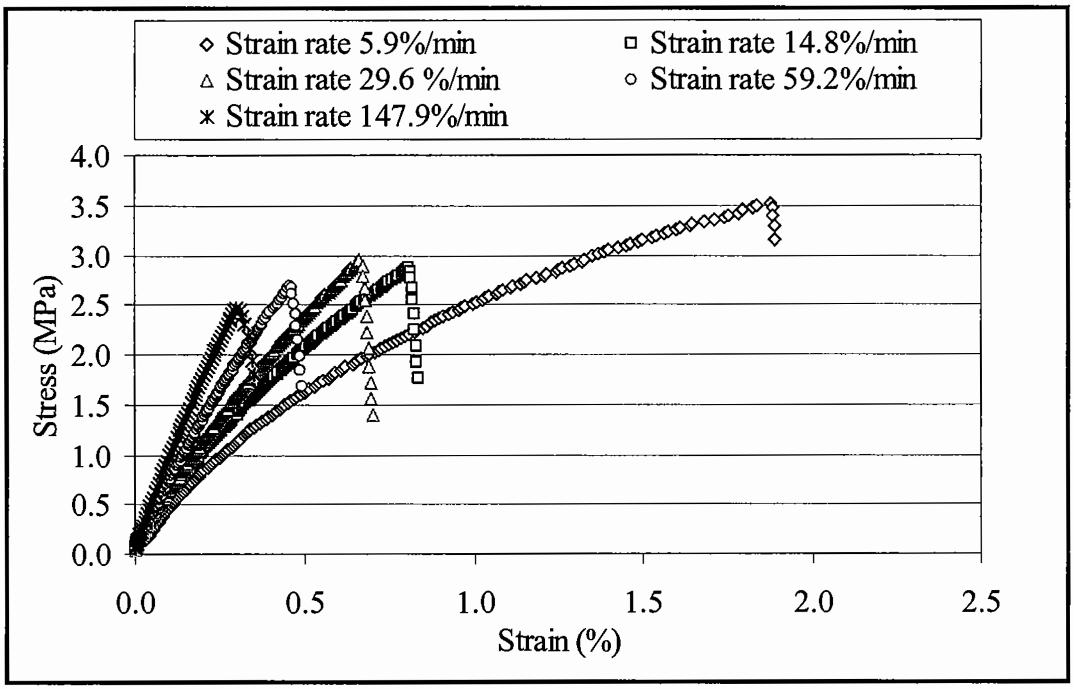


Figure 4.9 Stress Vs strain for pure bitumen tested at temperature –10°C

4.7.2 Results of Bitumen/Filler Mortar

4.7.2.1 Results of Mortar Containing 35% Limestone Filler

The direct tension test results for the bitumen-limestone mortar containing 35% limestone at temperatures +10, +5, 0, –5 and –10°C are shown in Figures 4.10 to 4.14. An increase in maximum stresses at failure was found with increasing strain rate; however most of the specimens failed completely at the lower temperatures –5 and –10°C except that there was no failure detected for the specimen tested at –5°C and a lower strain rate (5.9%/min). It was also observed that at –10°C, the stresses at failure decreased slightly as the strain rates increased.

At temperatures 0°C, +5°C and +10°C, the specimens flowed and it was not possible to define the stresses at failure and the corresponding strains. But it was possible to define the peak stress and the corresponding strain at higher strain rates.

After failure, the samples still adhered well to the end inserts, and no air bubbles were observed in the fractured surfaces of the specimens. This observation confirmed that the DTT procedures including; preparing, trimming, conditioning, and testing can be successfully conducted on bitumen-filler samples in the ratios used in this work by observing the special precautions noted in making the specimens.

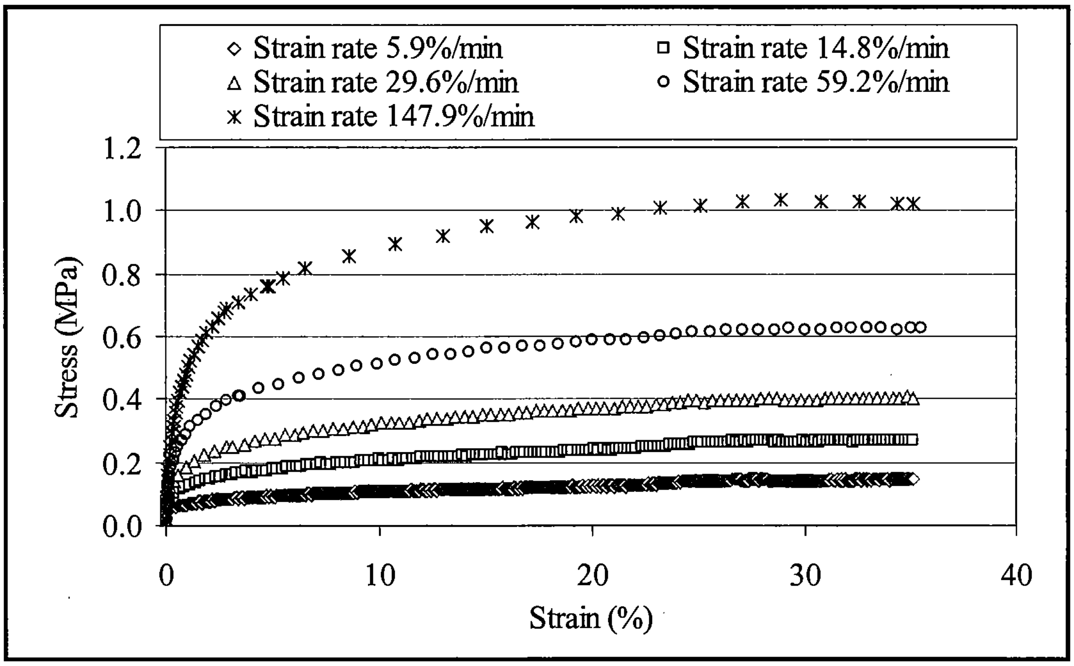


Figure 4.10 Stress Vs strain for mortar containing 35% limestone tested at temperature +10°C

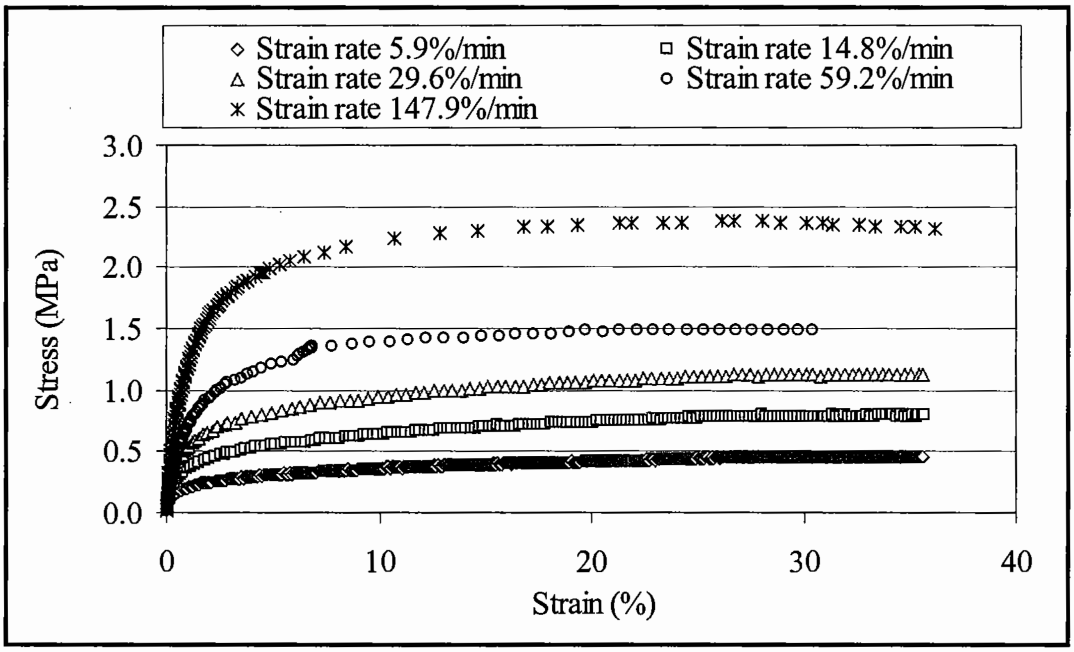


Figure 4.11 Stress Vs strain for mortar containing 35% limestone tested at temperature +5C

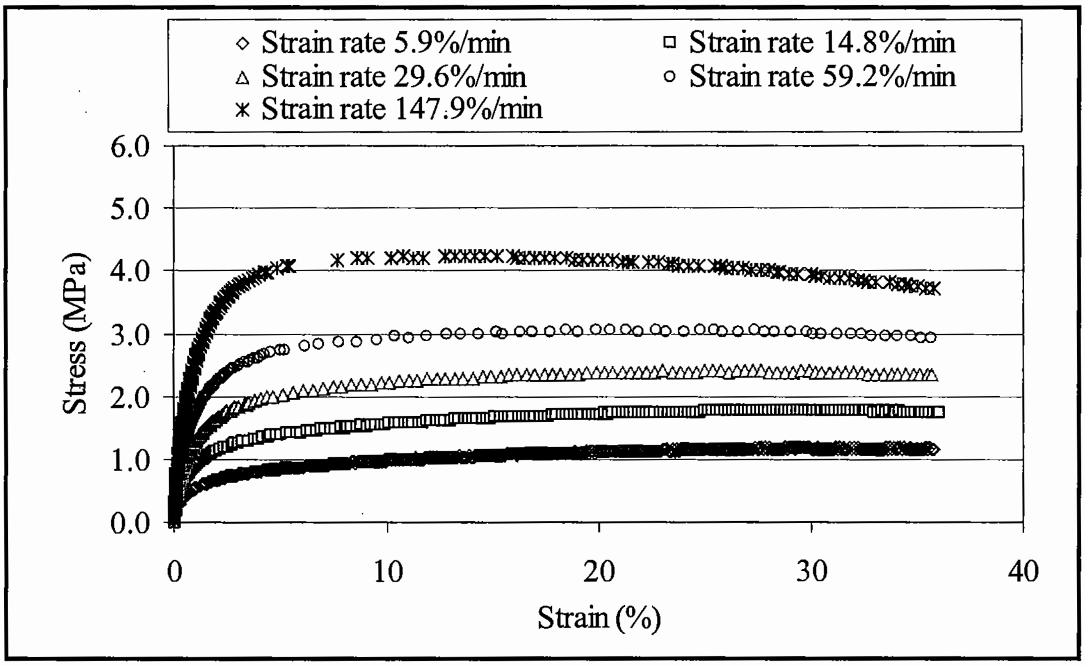


Figure 4.12 Stress Vs strain for mortar containing 35% limestone tested at temperature 0°C

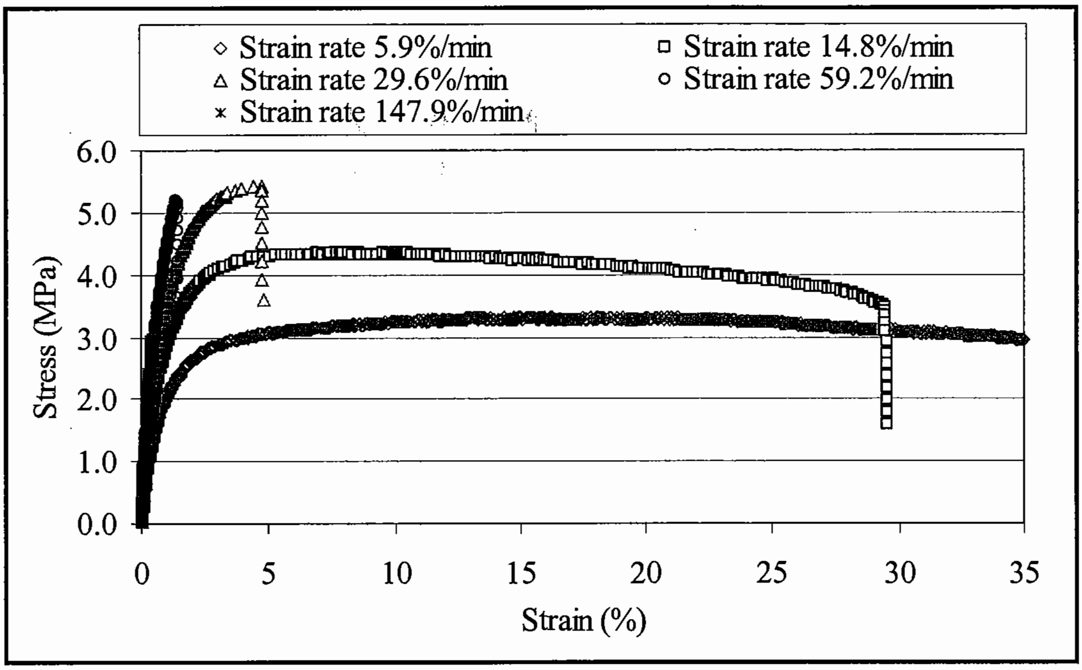


Figure 4.13 Stress Vs strain for mortar containing 35% limestone tested at temperature –5°C

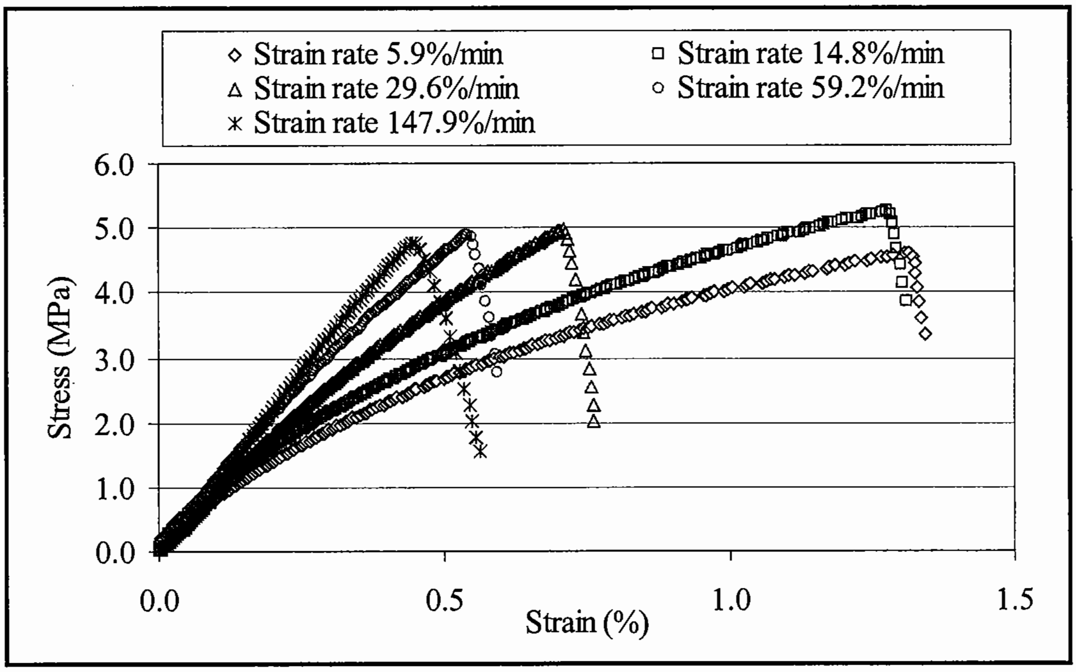


Figure 4.14 Stress Vs strain for mortar containing 35% limestone tested at temperature –10°C

4.7.2.2 Effect of Filler Percentages

Figures 4.15 and 4.16 show the results of testing bitumen and mortar containing different limestone content at different strain rates and at 0°C and –10°C (see also the Appendix for complete data set). The specimens failed completely at –10 and –5°C, and the maximum stresses and corresponding strains were obtained.

At +5 and +10°C, there was no fracture detected at any strain rate level and the specimens flowed up to the machine's strain limit. But for mortar containing 65% limestone filler failure was observed at high strain rates (59.2%/min and 147.9%/min). However, it was possible to detect the peak stresses and corresponding strains for others mortars at higher strain rates. The maximum stresses at the failure points generally increased as the filler content increased.

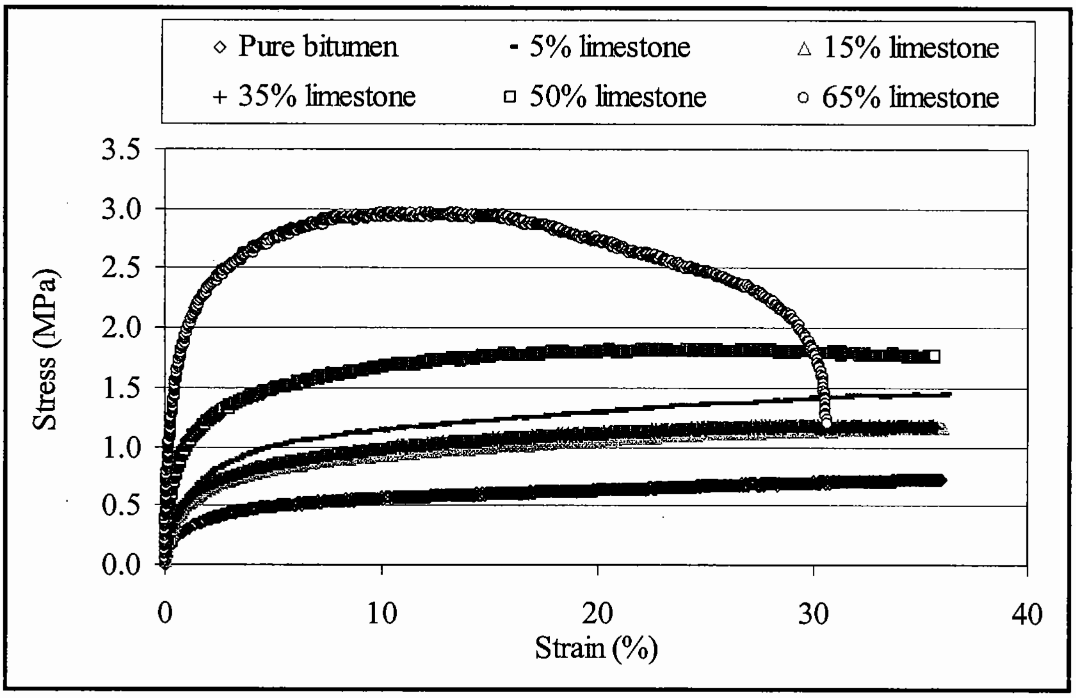


Figure 4.15 Stress Vs strain for bitumen and mortar containing different filler contents tested at 0°C and 5.9%/min

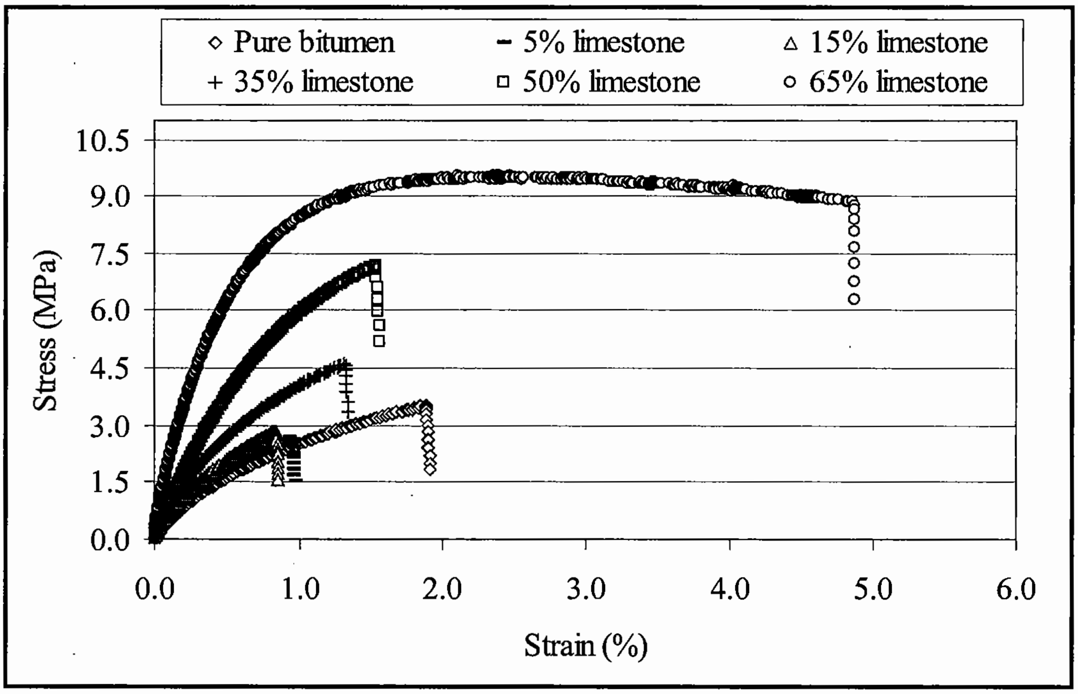


Figure 4.16 Stress Vs strain for bitumen and mortar containing different filler contents tested at –10°C and 5.9%/min

4.7.2.3 Effect of Mineral Filler Type

Tests were carried out using mortar containing 50% by mass of limestone, cement, and gritstone filler, tested at different temperatures and strain rates. The stresses and strains at failure were obtained. The effect of filler is seen to be similar for all the specimens tested, since they all increased the stress at failure. Figures 4.17 and 4.18 present a comparison of the different mortars containing the same filler concentrations (50% by mass), tested at temperature 0°C and strain rates 5.9%/min and 147%/min.

From the results, gritstone filler mortar gives higher peak stresses compared with the other mortars. More results are presented in the Appendix. Sewage sludge ash has also been used as an alternative filler to study how it affects failure stress. Because of the difficulty of preparing specimens containing 50% by mass of sewage sludge, a mortar of 15% by mass has been tested at different temperatures and strain rates. Figure 4.19 shows comparative results of limestone and sewage sludge ash mortars at different strain rates and at 0°C. Much higher stresses at failure were found when using sewage sludge mortar than with limestone mortar.

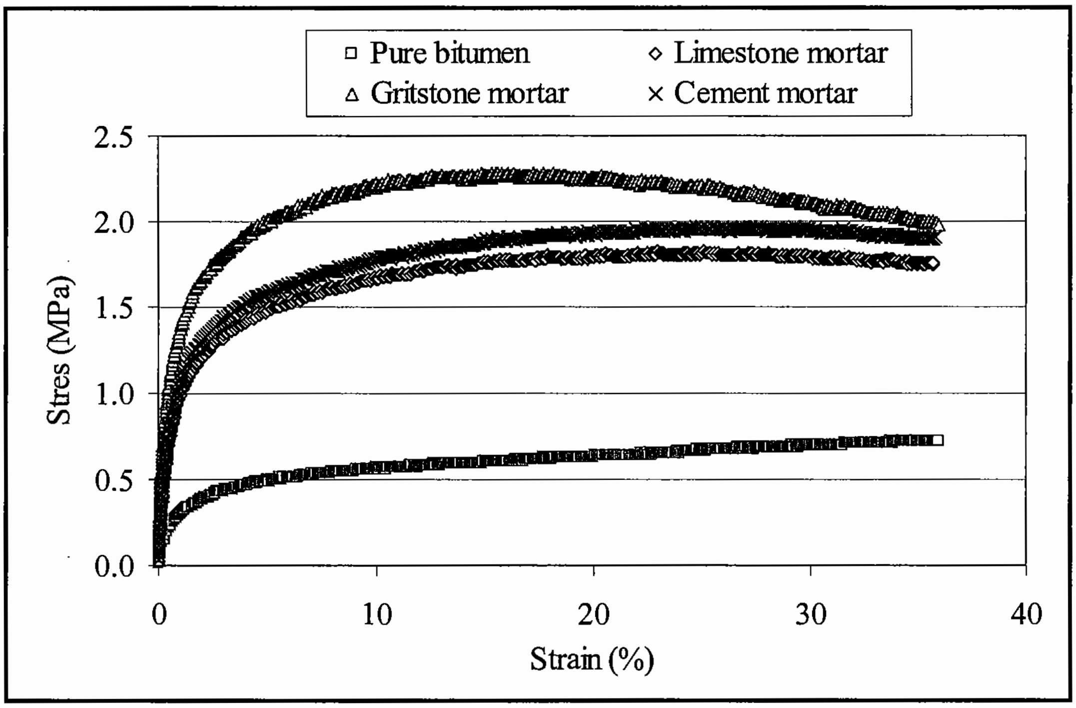


Figure 4.17 Stress Vs strain for bitumen and mortar containing 50% filler tested at 0°C and 5.9%/min

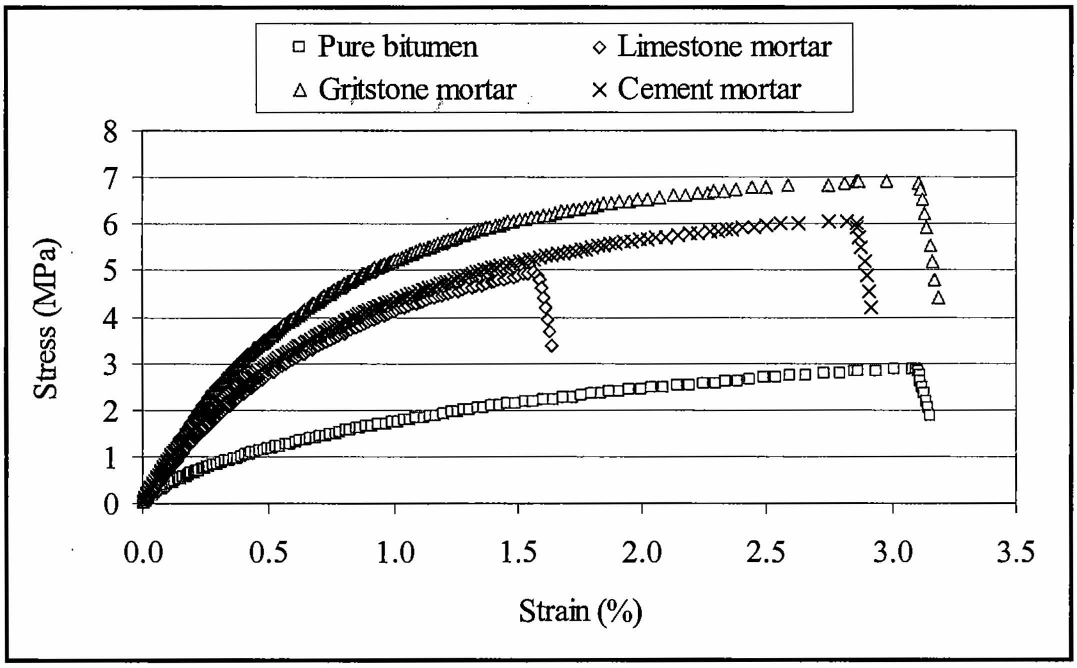


Figure 4.18 Stress Vs strain for bitumen and mortar containing 50% filler tested at 0°C and 147.9%/min

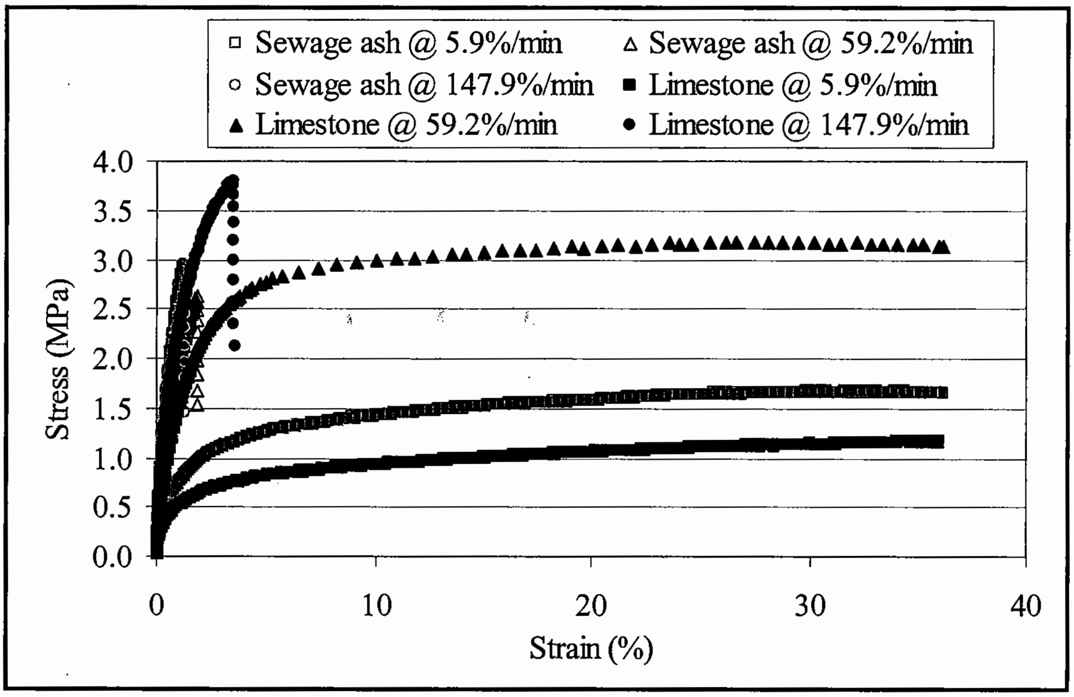


Figure 4.19 Stress Vs strain for mortar containing 15% filler tested at 0°C and different strain rates

4.7.2.4 Effect of Filler Particle Size

To study effect of filler particle size on failure stress a coarse limestone filler (particles passing 600μm sieve and retained on sieve 75μm) was used. Because of the limited amount of the filler, the mortar specimens were tested only at temperatures +10, +5 and –5°C and at different strain rates. Figures 4.20 and 4.21 show the comparative results of mortars containing 35% by mass of fine and coarse limestone filler tested at different strain rates and at –5°C and +5°C respectively.

The results indicate that the stress at failure is slightly increased for the mortar containing fine particles, but both have the potential to increase the failure stress, no statistically meaningful difference.

4.8 Discussion of the Results

For samples tested at higher temperatures (+5°C and +10°C), the samples elongated without rupture. This is because bitumens at this temperature are very ductile and the elongation is greater than 10% (up to the 36% limit of the machine). The bitumen-filler mortar stress at failure increases (at a given strain level) with increasing mineral filler content at low temperatures. At 0°C, +5°C, and +10°C the failure points are less well defined, but the results also show that the increase in filler content increased the peak stress value.

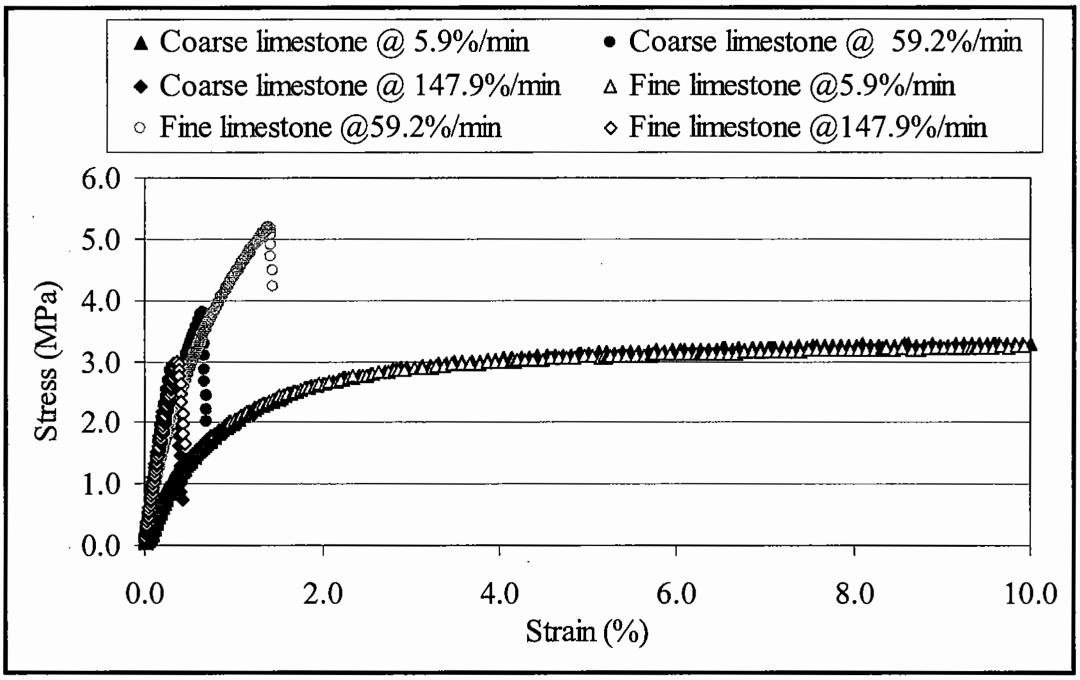


Figure 4.20 Stress Vs strain for mortar containing coarse and fine limestone particles tested at –5°C and different strain rates

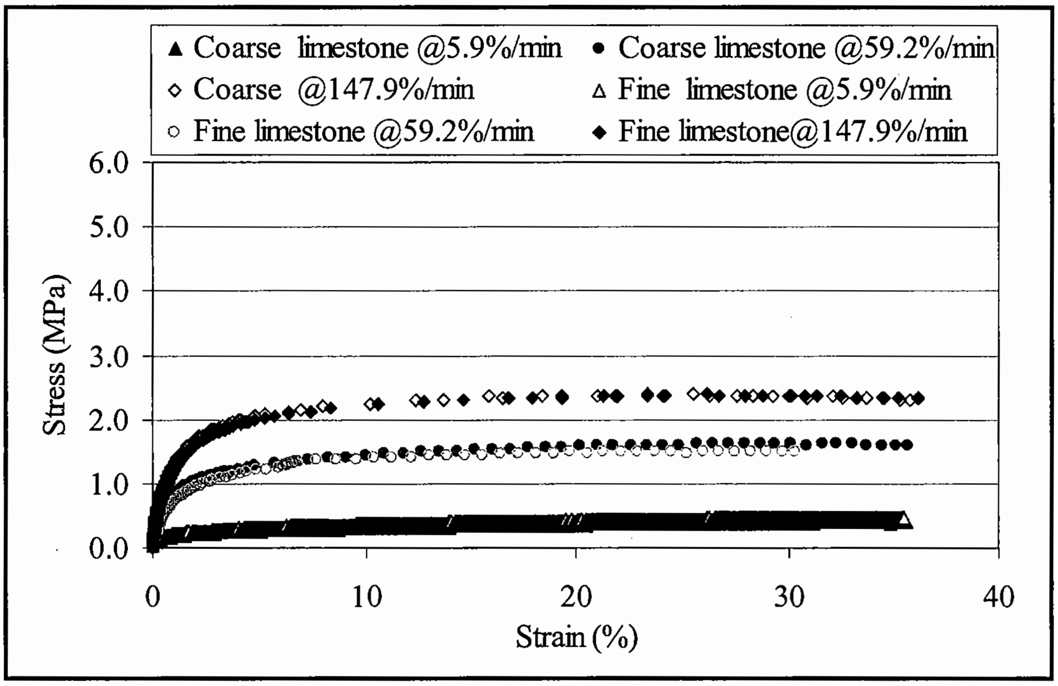


Figure 4.21 Stress Vs strain for mortar containing coarse and fine limestone particles tested at +5°C and different strain rates

Figure 4.22 illustrates the effect of temperature on the failure stresses for bitumen-limestone mortar tested at a low strain rate of 5.9%/min. It shows that an increase in temperature decreased the tensile stress at failure. Higher stresses and lower strains were obtained when limestone mortar was tested at –10°C. Charles and Moreland [121] also observed that, when testing bitumen in tension, as the temperature increased the tensile strength of the specimen at a given rate of deformation decreased. Figure 4.23 shows the strain at failure for bitumen/limestone mortar tested at a strain rate of 5.9%/min. The results show that adding large quantities of mineral filler decreased the tensile failure strain of bitumen-filler mortar. The result also shows that as the temperature increased, the failure strain increased.

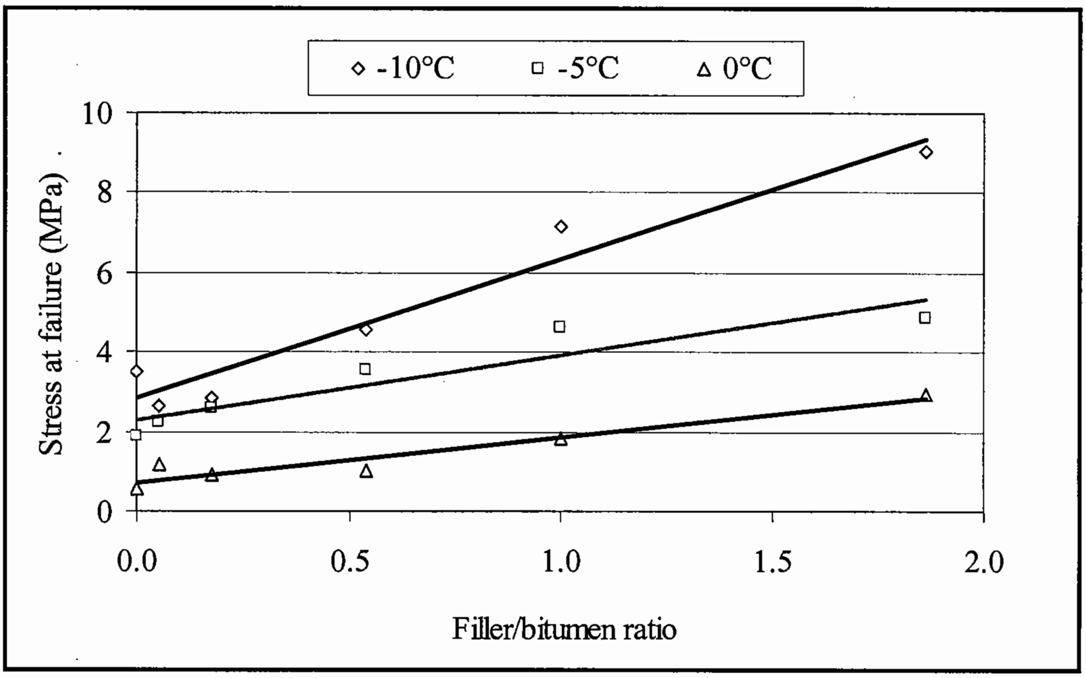


Figure 4.22 Effect of temperature on stress for bitumen/limestone mortar at strain rate 5.9%/min

The specimens fractured at tensile strains of 11% or less (see also Figure 4.28). Other researchers [122] postulated that mineral fillers may cause mastics to exhibit excessively brittle behaviour, and result in cracking at low temperatures due to stiffening of the bitumen. A research done by Chen and Peng [73] used the DTT to analyse failure properties of bitumen-filler mastics. They used Silica as a mineral filler and tested with bitumen at a strain rate of 1mm/min (3.9%/min) and at temperatures of –10°C, – 15°C, and –20°C. They concluded that increased filler concentration increases the tensile strength and also increases the tensile failure strain, which is agreed to some extend to the results shown in Figure 4.23 when filler increased from 5% to 50% by mass. The reason for this increase in tensile strain was not known, but they assumed that this might be because mineral fillers acted as crack arrestors.

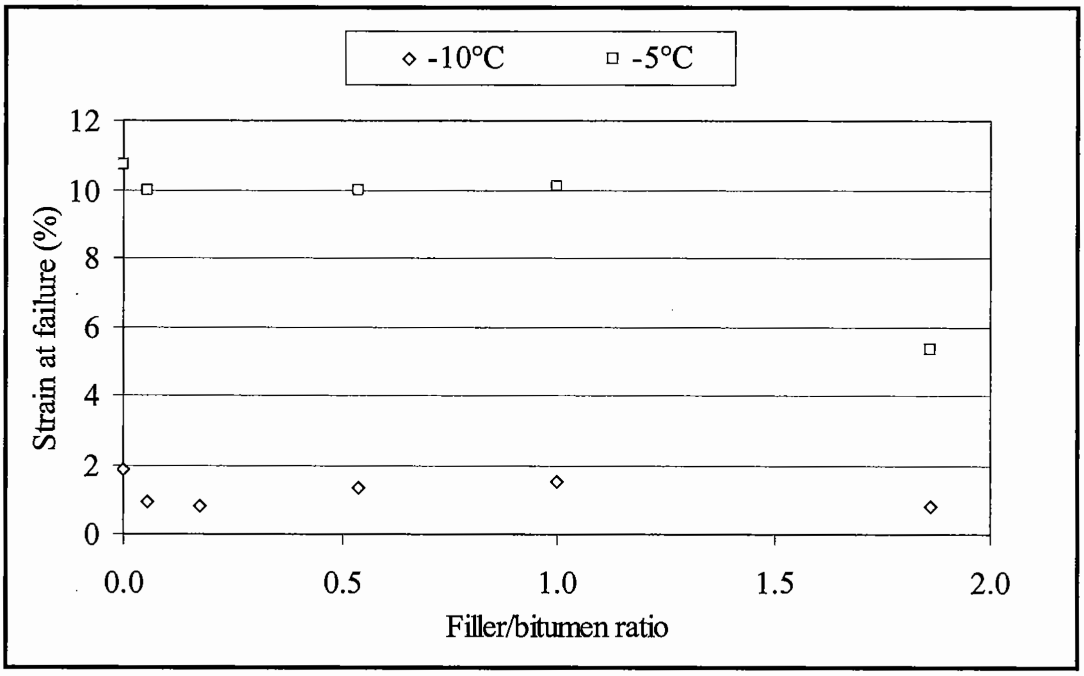


Figure 4.23 Effect of temperature on failure strain for mortar containing different limestone contents tested at strain rate 5.9%/min

The effect of strain rate on peak stress is presented in Figure 4.24 for bitumen/limestone mortar tested at 0°C. It is observed that in this, in this case increasing the strain rate increases the stress at failure. But at lower temperature (–10°C), and at higher strain rates (59.2 and 147.9%/min), the tensile failure stress is lower than at low strain rates. One possible explanation for effect may be because at lower temperature the specimens become very glassy and exhibit brittle behaviour, although the details beyond scope of the project.

Three types of filler, limestone, gritstone and cement, have been used in this research to investigate the effect of filler type on stresses at failure. Figure 4.25 presents the failure stresses for the filler types tested at a temperature of 0°C. It was found that gritstone gave highest stresses at failure at more temperatures. This might be because the gritstone contains finer particles as shown by the particle size distribution, and the surface area has an effect on the adhesion between filler particles and bitumen. Limestone and cement mortar have very similar failure stress levels.

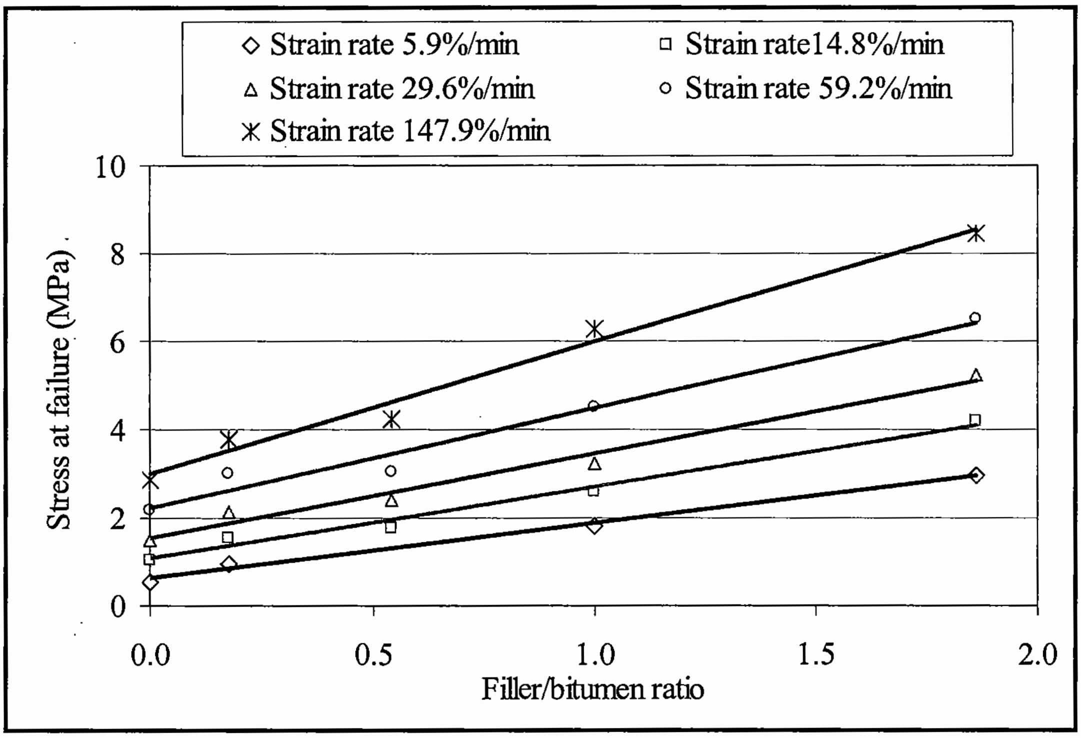


Figure 4.24 Effect of strain rate on failure stress for mortar containing different limestone contents at temperature 0°C

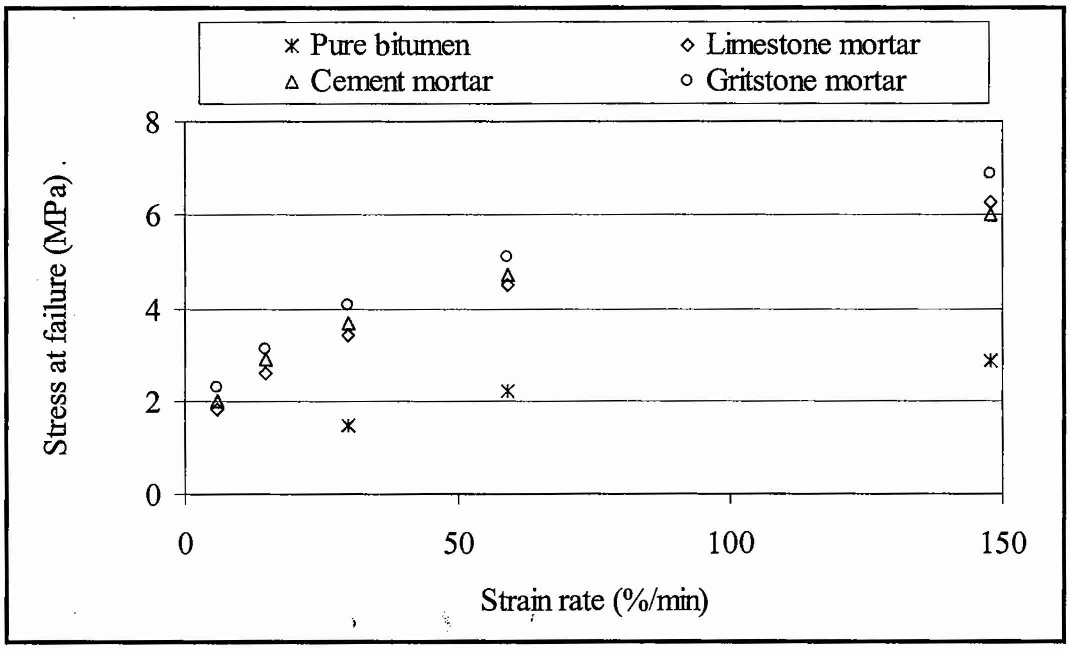


Figure 4.25 Stress at failure Vs temperature for bitumen and mortar containing different types of filler at 0°C

Figure 4.26 shows that for all filler types the stresses at failure decreased as the temperature increased. As discussed earlier, the strain at failure also increased as temperature increased or strain rate decreased.

The results for a mortar containing 35% by mass fine and coarse limestone filler particles are presented in Figure 4.27. The tests were carried out only at +5°C and –5°C. It seems to be that the mortar containing the coarse particles has lower failure stress compared with the normal limestone filler only at –5°C (i.e. fracture) not at +5°C (i.e. flow); the surface area may play a role in this case. But when sewage ash filler was used, which is coarser than limestone, it gave higher stresses at failure (see Figure 4.20), which means that this increase in failure stress is not only a surface area issue. Most likely the texture and the shape of the particle had a large effect as will be discussed later in Chapter 7.

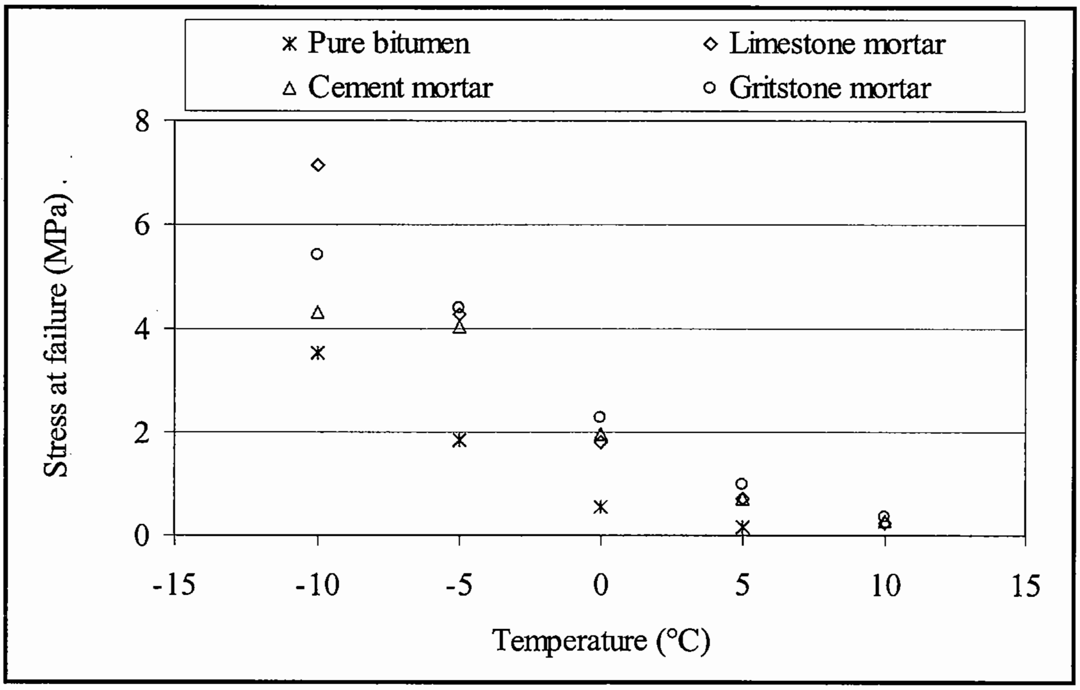


Figure 4.26 Stress at failure Vs temperature for bitumen and mortar containing different type of filler at strain rate 5.9%/min

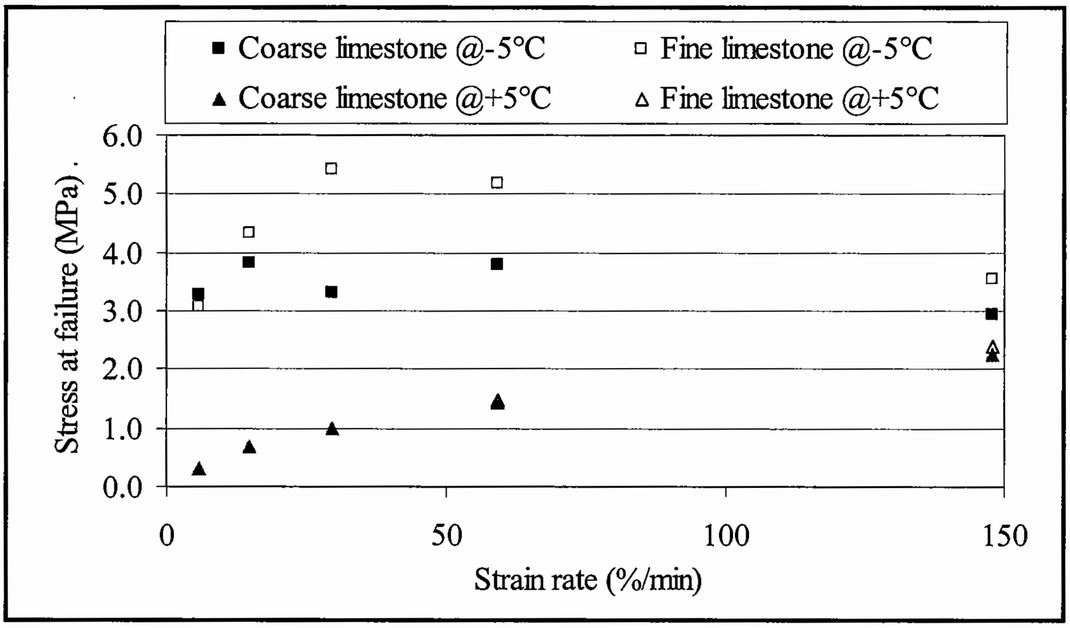


Figure 4.27 Effect of filler particles size on stress at failure for limestone mortar tested at different strain rate and temperatures

Figure 4.28 generated from limestone mortar data, presents stress and strain at failure for the limestone mortar containing different filler concentrations at different temperatures and strain rates.

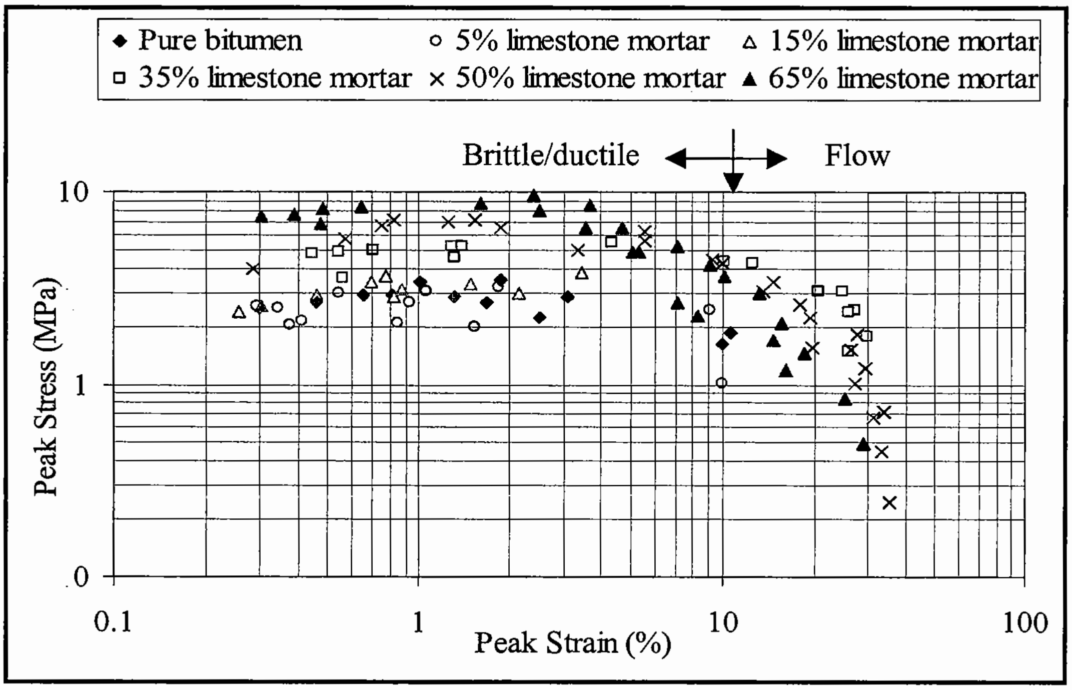


Figure 4.28 Peak stress and strain for bitumen and mortar containing different limestone contents tested at different strain rate (5.9 to 147.9%/min) and temperatures (-10 to +10°C)

It is clear that all of the specimens failed in brittle or ductile failure at a strain below 11% (the SHRP specification states that bitumen should fail in a brittle-ductile mode where the strain is less than 10%). The stresses at failure in the brittle region range between 3.0MPa and 9.0MPa depending on the filler concentration. Stresses at failure for pure bitumen and limestone mortar with 5% and 15% concentration were of the same order of magnitude, between 2.0 and 3.0MPa. Higher stresses were obtained when the mortar containing higher filler concentration was tested.

4.9 Summary

50pen bitumen and four types of filler have been tested in the DTT using different temperatures and strain rates. Different filler concentrations were used for limestone filler mortar. The results show that pure bitumen at higher temperature (+5°C and +10°C) does not usually show any clear failure point; instead the specimen flows up to the limit of the machine. However, it is possible for pure bitumen to fail when subjected to very high strain rates at high temperature (+10°C).

The key points are:

- When filler was added to bitumen at different concentrations, at high temperatures a maximum stress was found, followed by a decrease in stress up to the machine limit. At low temperatures, specimens fractured completely in a brittle mode. The fracture surface indicated that there was no void on the surface and the mortar seemed to be homogenous.

- The results for bitumen and mortar show that increasing strain rate will increase the stress at failure and increasing testing temperature will decrease the failure stress. 5% and 15% limestone concentration show little change to the behaviour of pure bitumen.

- Limestone mortar and cement mortar had the same failure behaviour, but gritstone mortar gave higher stresses at failure than the others under the same testing conditions. Sewage sludge mortar gave much higher stresses still.

- All brittle failures occurred at a strain less than 11%, and the stress at brittle failure ranged between 3MPa for bitumen and low filler concentration mortar, to 9MPa for high filler concentration. Brittle failure stress did not vary much over quite a wide range of strain; i.e. failure appeared to be stress- dependent.

5

Binder and Mortar Fatigue

5.1 General

This chapter describes new equipment, which has been developed to investigate the behaviour of binder in fatigue. A test apparatus using either flat or hemi-spherically shaped end 'platens' (rounded ends) was designed and used for fatigue testing of bitumen and bitumen/filler mortar. Bitumen filled with different fillers was tested using the new fatigue test facility under various operating conditions (frequency and loading rate). This chapter presents the testing protocol and some of the results obtained.

Fatigue cracking of a bituminous layer arises from repeated tensile strains due to traffic loading. The progressive damage to the asphalt layer leads to the occurrence of macro cracks at the bottom (or top) of the asphalt layer that propagate to the pavement surface. The observation of specimens tested in fatigue tests shows that cracks in an asphalt mixture tend to run through the binder. The binder, in practice, consists of a mixture of bitumen and fine aggregate or filler.

Although it is recognised that fatigue damage is mainly caused by cracking or damage within the bituminous binder and/or bitumen-filler mortar, fatigue testing has generally been limited to the full asphalt mixture. Hence different standard tests are used to characterise fatigue of mixtures. However, a number of research groups have recently started to study the fatigue performance of bituminous binder by means of time sweep tests using a dynamic shear rheometer (DSR) [49,88,91]. This test, one of the SHRP tests, can be used to study the fatigue of bitumen, but to date little information on fatigue properties of binder has been published. Also contributing to this is the lack of any ideal fatigue test for bitumen and bitumen/filler mortar. The DSR may not be the most suitable or convenient means of undertaking binder and bitumen/filler mortar fatigue testing, as the stress and strain conditions in the test specimen are not uniform.

To complete the picture of mixture fatigue and for better understanding of the fatigue phenomenon, it is necessary to determine the fatigue parameters for both binder and bitumen/filler mortar. Therefore, in order to evaluate the contribution of bitumen and its mortar to the mixture fatigue phenomenon, a new fatigue test has been developed. The new test equipment is used in the laboratory to simulate the fatigue of bitumen and mortar under repeated loading, attempting to simulate the in-service performance within a mixture.

To simplify the test, two testing geometries were designed and used, consisting of: firstly, a film of bitumen between two flat shaped blocks (conventional uniaxial configuration) and, secondly, bitumen between two steel convex (spherically shaped) protuberances simulating two aggregate pieces (hemi-spherical uniaxial configuration). Figure 5.1 is a schematic representation of the conventional and the hemi-spherical direct uniaxial fatigue testing geometries.

5.2 Choice of Equipment

In order to study the fatigue of binder, a film of binder has to be tested. Charles and Moreland [121] reported that the thickness of binder film coating the aggregate particles in the bituminous mixture varies between 20 and 600microns. However for the initial binder test, a 2mm film thickness was chosen. Binder testing requires accurate measurement of relatively small loads, and sensitive LVDTs. A uniaxial testing machine with a load cell capacity of 1000N was chosen, as it was the only available machine capable of doing this test. Figure 5.2 shows the uniaxial equipment used for binder fatigue testing.

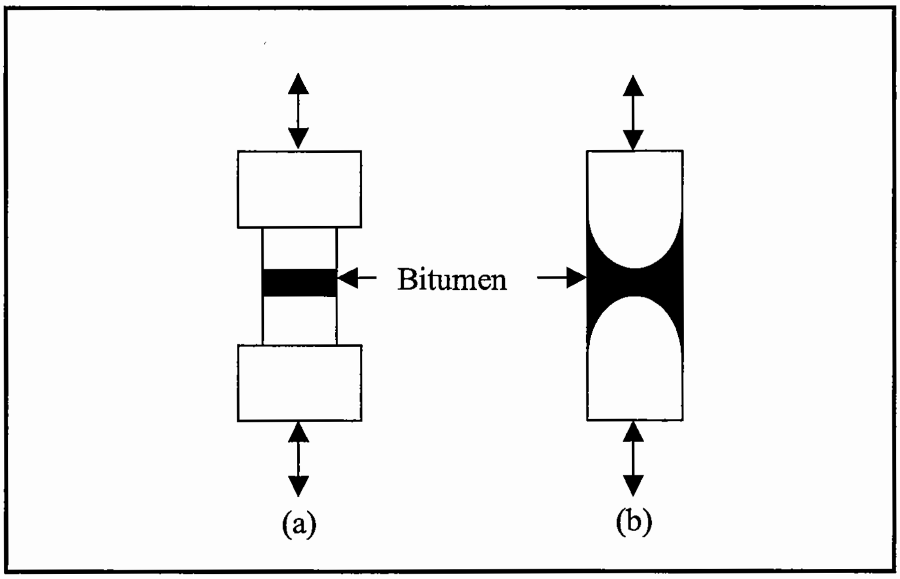


Figure 5.1 – Schematic representation of (a) conventional and (b) hemi-spherical direct uniaxial fatigue testing geometries

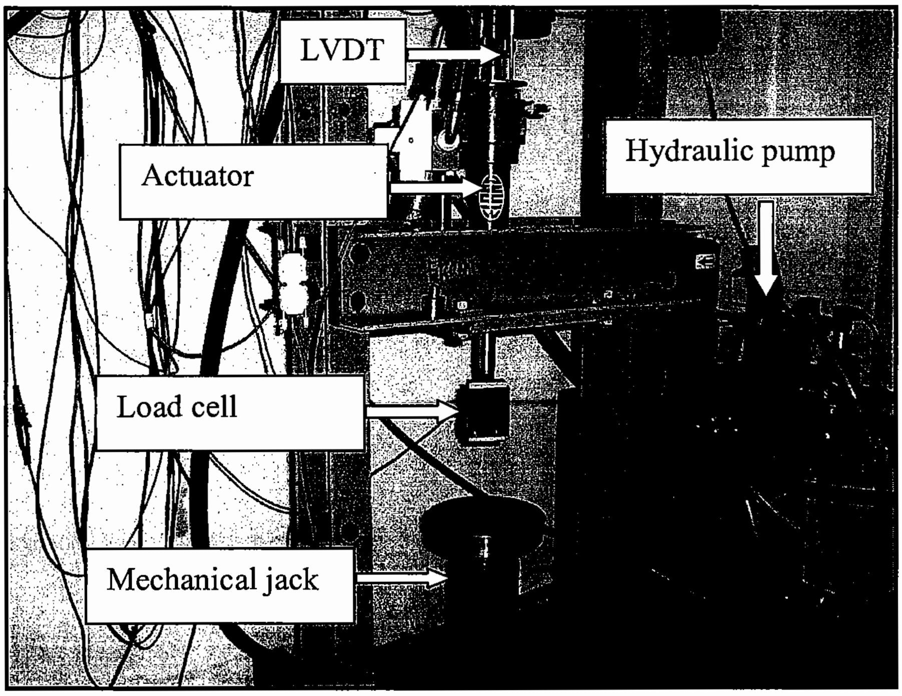


Figure 5.2 Uniaxial testing machine for binder fatigue testing

The dynamic uniaxial test is a repeated load test, from which the stresses and corresponding strains and number of load repetitions can be determined. This method was chosen for the new binder fatigue test because it is a quick, simple test and satisfies the requirements of the research (test conditions).

5.3 Design of the New Binder Fatigue Apparatus

The new binder fatigue apparatus was designed as five pieces taking into account simplicity and the availability of the materials.

- Load cell attachment
- Upper end platen
- Split mould with jubilee clip
- Bottom end platen
- Steel plate platform.

The new binder fatigue apparatus comprises the flat shaped end platens and hemi-spherically shaped end platens, both 16mm diameter as shown in Figures 5.3 and 5.4. Most of the tests were carried out using the flat end platens, to simplify computation of stress and strain (although in reality stresses are non-uniform). The hemi-spherically shaped ends were designed to simulate two aggregate particles with bitumen in between. Both of the platen geometries and the other accessories were made of steel. Steel platens were used to ease cleaning, give solid platens, and to provide good adhesion to the binder with the aid of roughening of the surface.

The load cell attachment was designed in order to be fixed onto the load cell permanently, and to allow the upper platen to be attached easily without damaging the specimen. The other accessories were removable for cleaning and preparing the specimens.

The upper end platen had a T hole for draining the excess bitumen so that the required thickness could be obtained. It was firmly fixed, before testing, to the load cell attachment using two locking screws to ensure the loads transferred uniformly into the binder specimen. The lower platen attached at the bottom to a steel plate with sufficient thickness so as not to deform under load, and which could also be adjusted horizontally in order to align the system in the vertical plane.

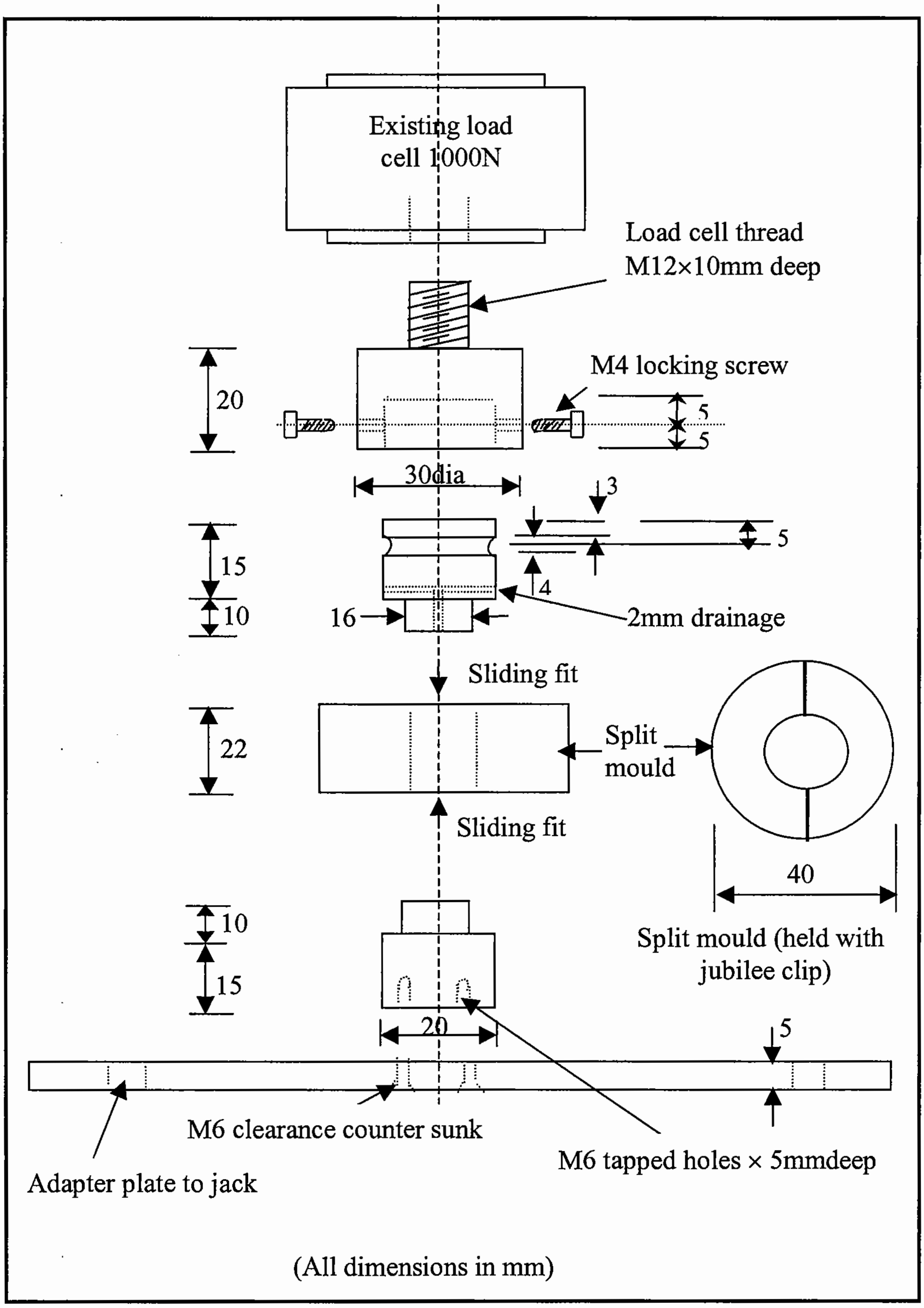


Figure 5.3 Design of the new binder fatigue tester (flat shaped end platens)

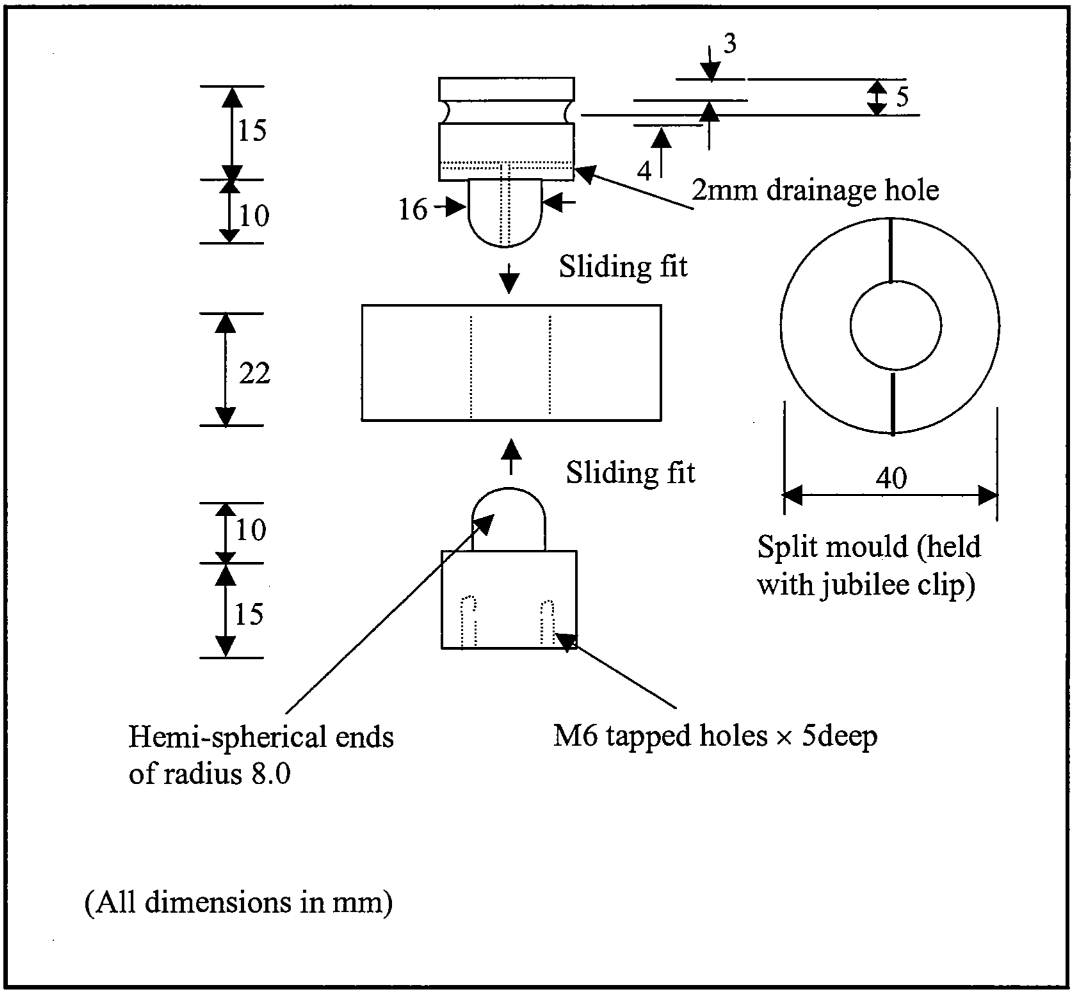


Figure 5.4 Hemi-spherically shaped ends platens

The upper and lower end platens were initially fixed together using a split mould (consisting of half of a cylinder). A jubilee clip was used to hold the split mould tightly together with the platens so that after pouring the specimen they would not move relative to each other even when they were fixed to the machine. The split mould was removed prior to the test. Figures 5.5 to 5.7 show the arrangement of the new binder fatigue apparatus.

The advantages and disadvantages of this apparatus can be summarised as follows:

Advantages:

- The test is relatively simple to conduct
- It has a simple design using available materials
- The measured properties can be used for binder fatigue evaluation

Disadvantages

- Bonding problems can arise between the platens and the binder

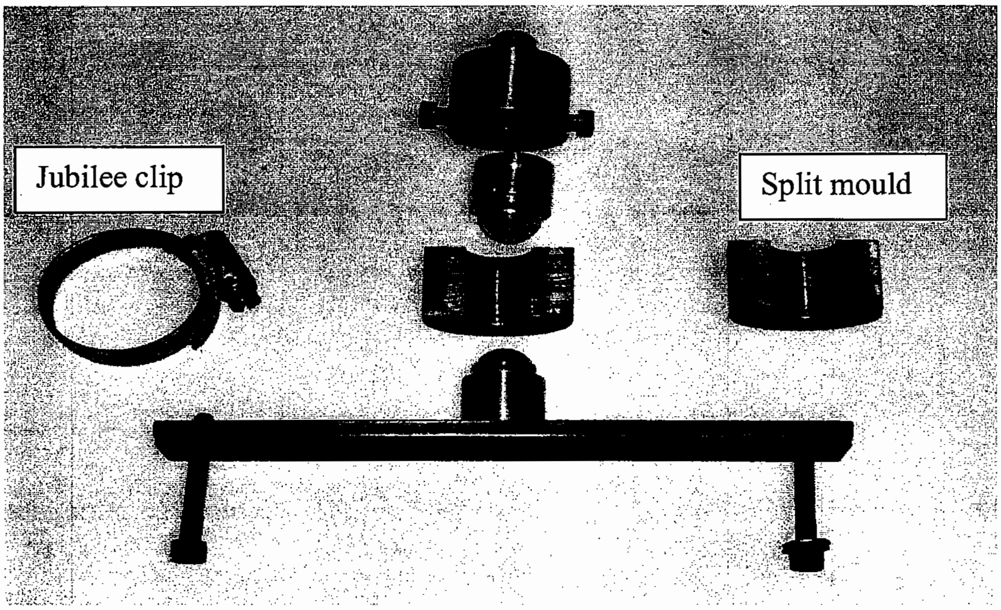


Figure 5.5 Hemi-spherically shaped ends platens for binder fatigue testing

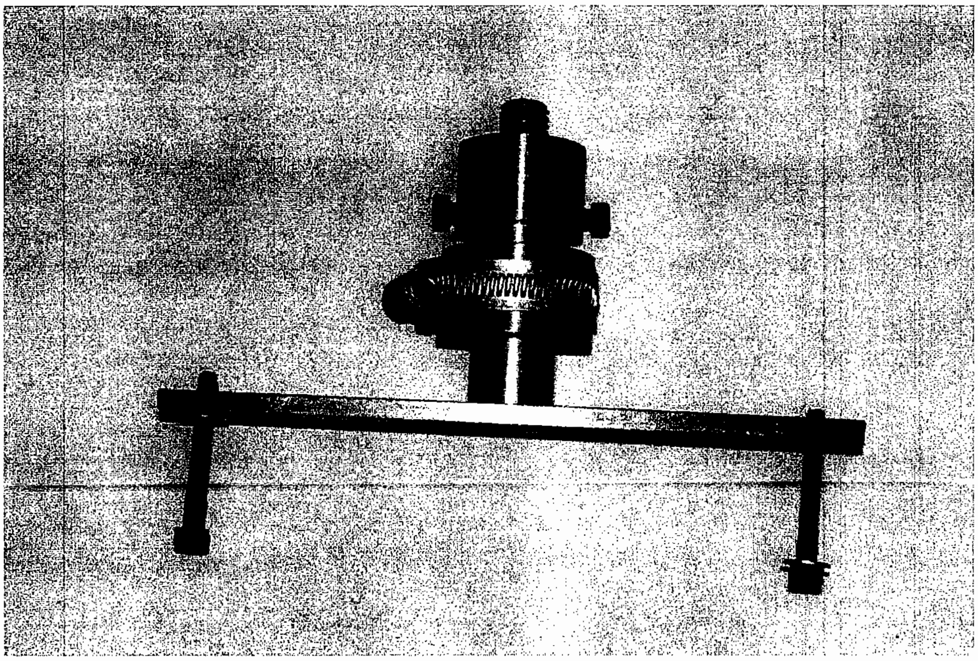


Figure 5.6 Arrangement for binder fatigue testing

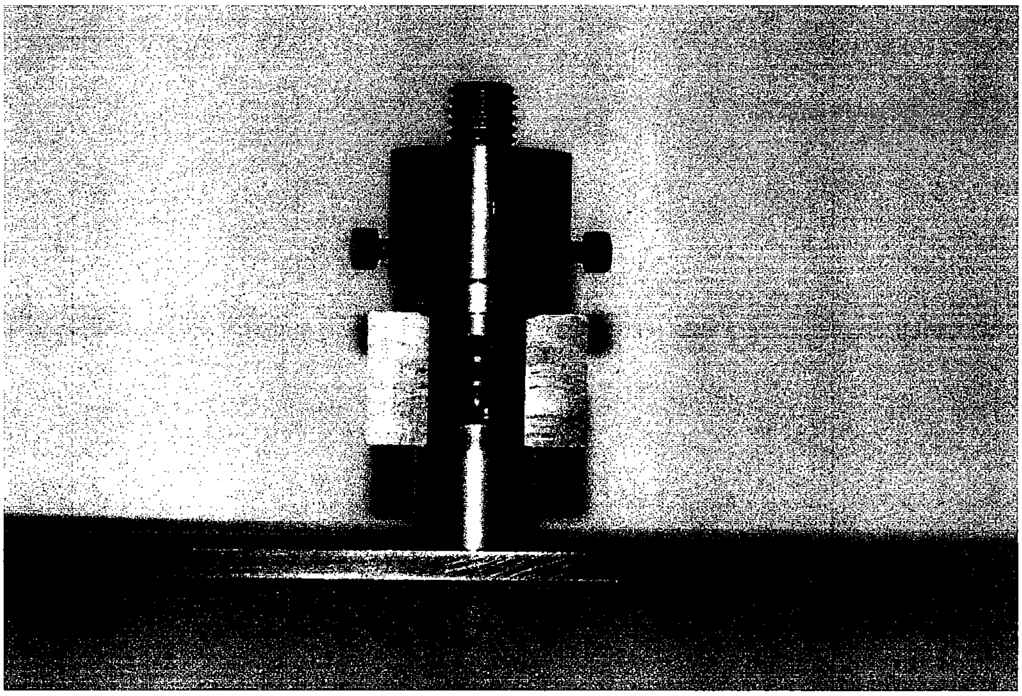


Figure 5.7 Side view of the hemi-spherically shaped end platens for binder fatigue testing

5.3.1 Test Configuration

Figure 5.8 shows the details of the test apparatus arrangement. In order to accurately control the test temperature, all the test equipment is located in a temperature-controlled room. The rig is able to produce dynamic loading, which induces stresses and strains in the bitumen specimen. It consists of a loading frame and axially mounted hydraulic actuator. As the system operates using a computer-controlled servo-hydraulic closed-loop feedback system, it was important to have a fast and accurate data acquisition system.

A controller with a high speed data acquisition system is mounted outside the temperature controlled room. Hydraulic power was supplied by a pump. The dynamic loads were applied in this testing configuration to the top of the flat end platens along the central vertical plane. The load applied to the bitumen specimen was monitored by a sensitive load cell (1000N capacity). The vertical deformation on the specimens was measured using a linear variable differential transformer (LVDT) attached to the top of the load actuator.

The bottom platens of the new testing equipment were fixed to the top of the steel plate of a raised platform (mechanical jack) and the upper part of the equipment was attached to the load cell and loading rig. The load cell provides the feedback signal required for the control system.

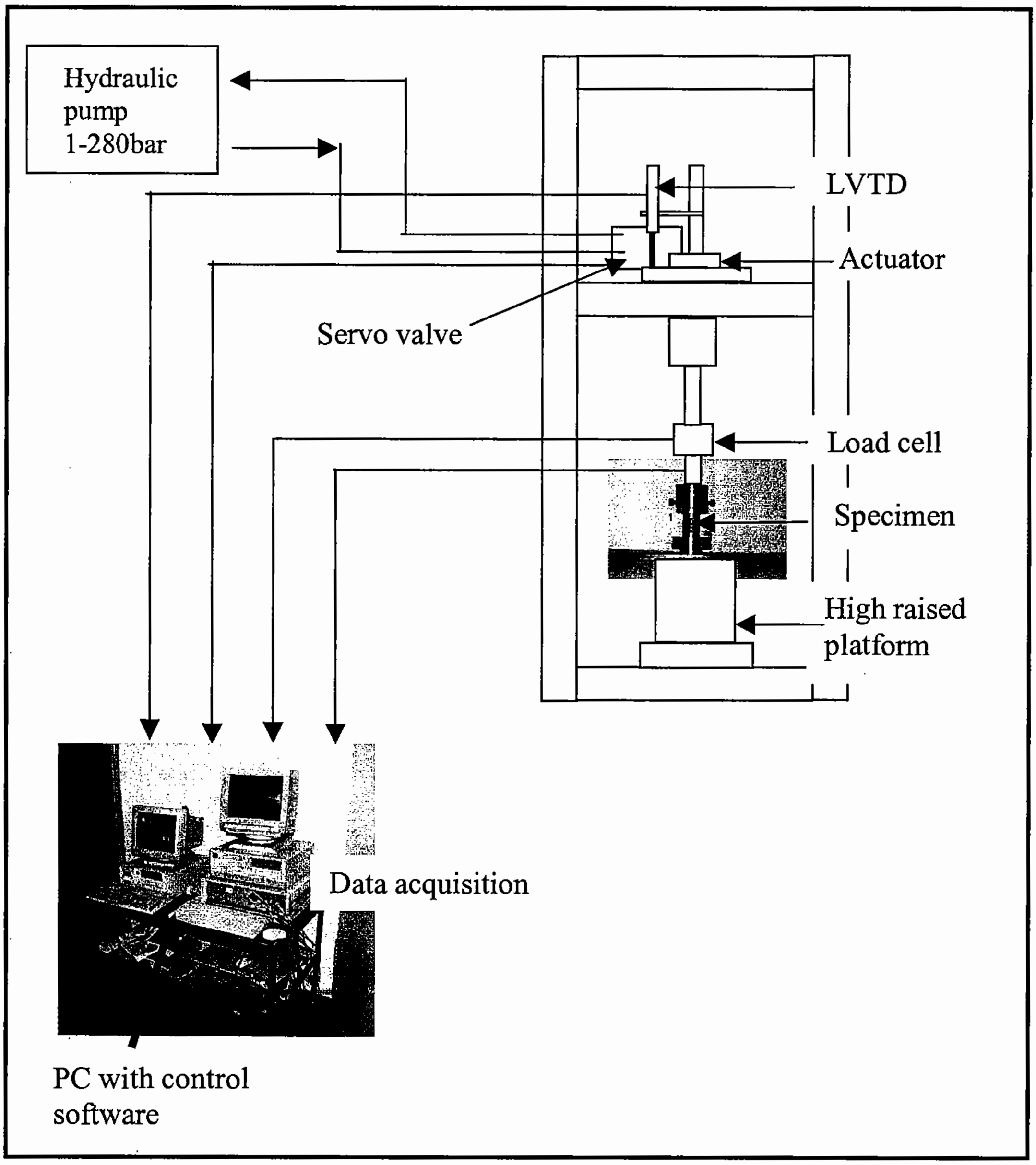


Figure 5.8 Configuration of overall hydraulic testing equipment

5.4 Testing Methodology

5.4.1 Test Temperature

As mentioned before, bitumen is a viscoelastic material. Its properties are very sensitive to temperature and load application rate. Airey [65] stated that the temperatures experienced in a road pavement cover a wide range (-10°C to +60°C) depending upon location and climate. In the UK, typical average annual air temperatures are between 8.5°C and 10°C, with pavement temperature ranging from about -5°C to 45°C.

As temperature is an important variable for bitumen behaviour, a test temperature range of 0°C to +20°C has been chosen for the binder fatigue test. While most of the fatigue tests were carried out at +10°C, some tests for pure bitumen were carried out at 0°C and some mortar tests at +20°C. In order to accurately control the temperature, the entire test assembly was located inside a temperature-controlled room capable of operating between –5°C to + 70°C.

5.4.2 Rate of Loading and Loading Mode

Dynamic loading simulates the in-service use of the pavement materials. Increasing rates of loading lead to increased stiffness of the bitumen, whereas decreasing rates of loading lead to decreased stiffness. The following loading times are representative of typical situations [65]:

Fast road traffic	= 0.01-0.1sec
Parking and slow moving traffic	= 0.1-1.0sec

Therefore a frequency range of 1.0Hz to 15Hz has been used for the fatigue tests. Most of the tests were carried out using a frequency of 10Hz, except for pure bitumen where additional frequencies were used. Researchers [4,102] have found that loading waveform affects the fatigue life of a bituminous mixture and have explained this by the energy put into the system per loading cycle. For this research, a sinusoidal waveform was chosen in a tension-compression arrangement, although

Read [4] suggested that a haversine wave with a rest period is the closets waveform to simulate the loading condition in the field.

There are two main modes of loading, which can be applied in a fatigue test: stress controlled and strain controlled loading. Brown [102] explained the different modes of loading in a whole pavement structure and stated that the pavement as a whole system is subjected to stress controlled loading. Thick and stiff layers, which have considerable structural significance will also be stress controlled.

Since, the stress controlled mode there is a definite point of failure, the stress controlled mode of loading was chosen for these fatigue tests. For each fatigue test carried out in this research, the number of cycles to failure was determined as the point of complete specimen fracture.

5.4.3 Materials

The materials used for fatigue testing were the same materials used in the direct tension test (DTT) as discussed in chapter 4, section 4.4. 50pen bitumen, supplied by Nynas, was also used for the fatigue testing.

Four types of mineral filler were used for the investigations of fatigue characteristics of bitumen-filler mortar. They are:

- Limestone filler
- Cement
- Gritstone filler
- Sewage sludge ash

The bitumen/filler mortars were divided into three groups according to the filler concentration:

- **Bitumen/limestone mortar**: limestone concentrations used were: 5%, 15%, 35%, and 65% by mass (percentages by volume: 2%, 6%, 17%, and 41% respectively).

- **Bitumen/gritstone (and bitumen/cement) mortar**: Filler concentration was 35% by mass (percentage by volume: 17% for gritstone and 15% for cement)
- **Bitumen/sewage sludge mortar**: Filler concentration was 15% (percentage by volume: 6.3%).

5.4.4 Specimen Manufacturing Technique

For consistency, the following procedure was used for mixing and preparing the bitumen specimen for the new fatigue test equipment:

1- Determine an estimated amount of bitumen (or bitumen-filler mortar) required to produce the correct film thickness.

2- Place the bitumen in an oven at a temperature of 165°C±5.

3- To prepare a bitumen/filler mortar follow the same procedure as discussed in chapter 4.

4- Fix the bottom platen to the base plate, and remove the upper platen.

5- Apply a release agent carefully to the inner faces of the split mould (specimen holding device) to prevent the adhesion of binder.

6- When the bitumen is fluid and ready for pouring, warm the whole assembly for a few minutes, so that the release agent is not affected by heating. The aims of warming are to avoid loss of temperature in the bitumen specimen, and to allow the bitumen to cool over a period of time.

7- Place the assembly on a flat surface.

8- Place and tighten the spilt mould into the right position on the bottom platen so that it will allow the required thickness of the bitumen.

9- Carefully and slowly pour the estimated amount of hot bitumen (or mortar), to give the required thickness, on top of the bottom platen through the split mould.

10- Carefully and quickly insert and push the upper part of the equipment (upper platen) into the split mould until it touches the mould, so that the correct film thickness is obtained. Tighten the jubilee clip carefully and any excess amount of bitumen will come out through the drainage hole in the upper part of the equipment.

11- Clean excess bitumen from the split mould in order to ease the removal of the mould prior to testing.

12- Allow the whole assembly to cool at room temperature for at least one hour. This allows the bitumen to solidify without the development of voids due to rapid cooling.
13- Transfer the whole assembly to the temperature controlled room for conditioning.
14- The jubilee clip should not be removed until both the upper part of the equipment and the lower part have been fixed completely to the machine.

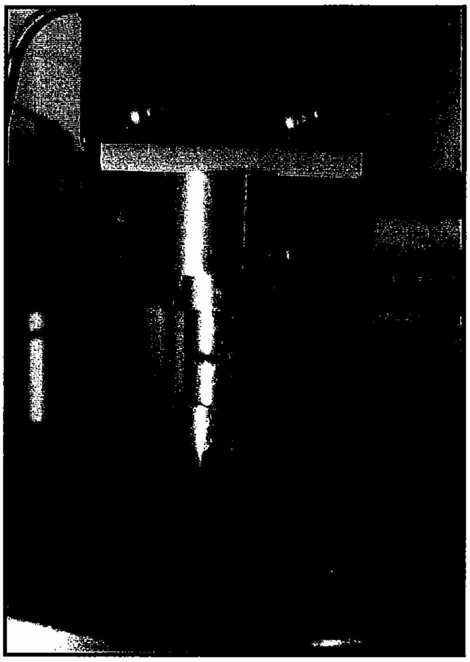
5.4.5 Testing Procedure

The test procedure consisted of conditioning the specimen in the temperature-controlled room for at least 1 hour at the required testing temperature. At the end of this period, the assembly is placed on the machine, and the bolts for fixing the bottom platen to the raised platform put in place but without tightening so as to allow precise alignment of the assembly with the loading cell.

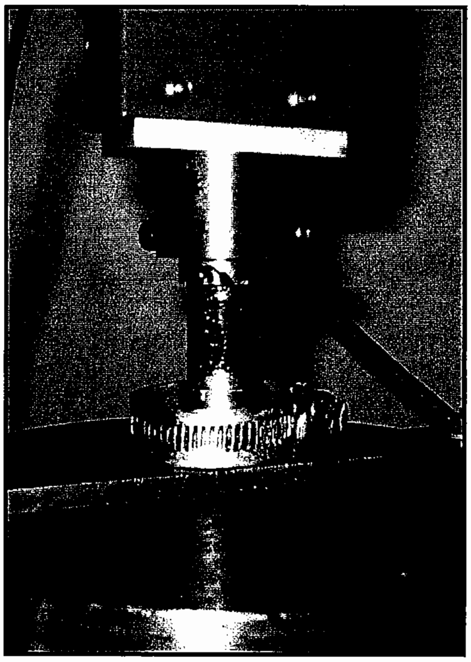
The platform is raised and the assembly moved horizontally until it is aligned with the load cell in the vertical plane. At this moment, the bolts of the bottom plates are tightened carefully. Then the hydraulic pump is switched on, and the load adjusted to as near zero as possible. The locking screw for the upper platen is fixed carefully and the split mould removed ready for testing. The LVDT is adjusted to as near zero as possible in order to measure the displacement.

The data capture rate was programmable and it was found to be capable of taking data every millisecond. The input data for the controlling computer are the time delay for each scan and number of scanning points required. These values depend on the frequency and number of cycles to be recorded. When all these were set, the specimen was subjected to sinusoidal loads (stress) over a range of amplitudes and frequencies. The axial displacements (strains) were measured accordingly. The data captured by the data acquisition system was collected and converted from voltage output to load in Newtons (N) and displacement in millimetres (mm).

Several sinusoidal tension-compression tests were initially carried out at different frequencies 1, 5 and 10Hz to test the equipment and the output results. Figure 5.9 shows both the flat and hemispherical end platens during binder fatigue testing. To improve the accuracy of the sinusoidal waveform, the proportional and differential feedback gains of the servo-hydraulic system were adjusted.



(a)



(b)

Figure 5.9 Photographs of the (a) flat and (b) hemi-spherical uniaxial fatigue testing geometries

Because of the limitation in the capacity of the controlling computer (storage capacity) and the difficulties of recording continuous data cycles, only five cycles were recorded every 400 cycles. The load and displacement data for the recorded cycles were used to generate sinusoidal stress and strain waves as shown in Figure 5.10. The peak stress and strain values were then determined.

5.4.6 Stress and Strain Calculations

The stress distribution within the specimen is very complicated for both the flat end and hemi-spherical end cases. This is due to the change in shape during testing as shown in Figure 5.11, which will affect the stresses. Because of the complexity of this problem, to simplify the calculations the average stress was calculated and will be presented.

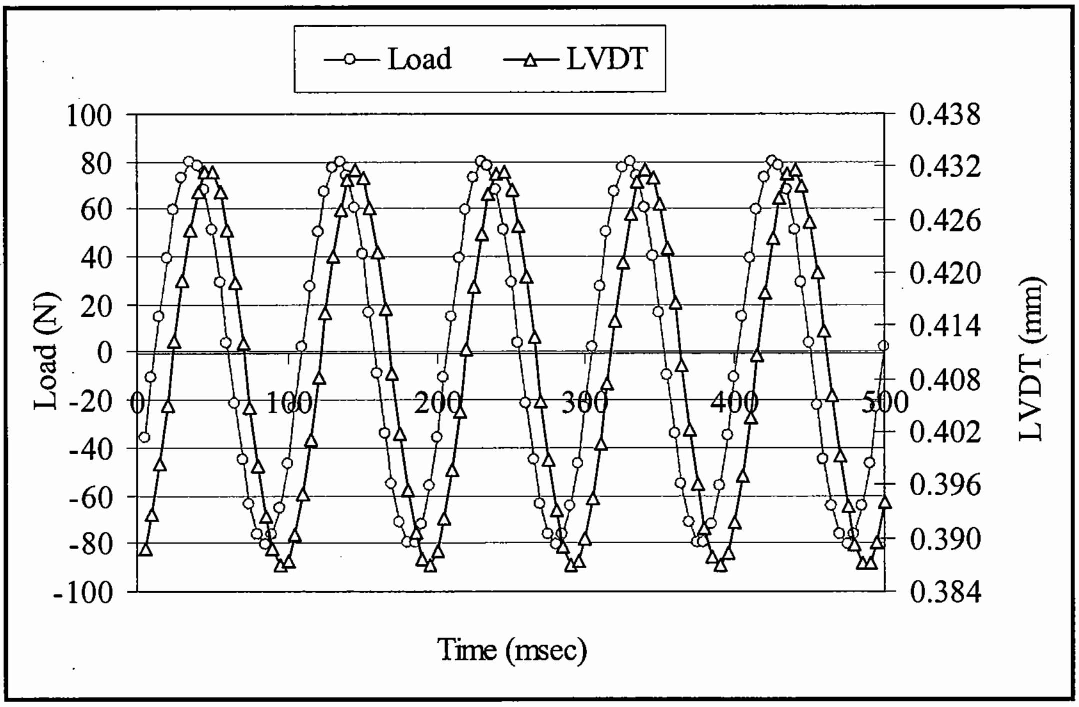


Figure 5.10 Load and LVDT response for binder fatigue testing at +10°C and 10Hz

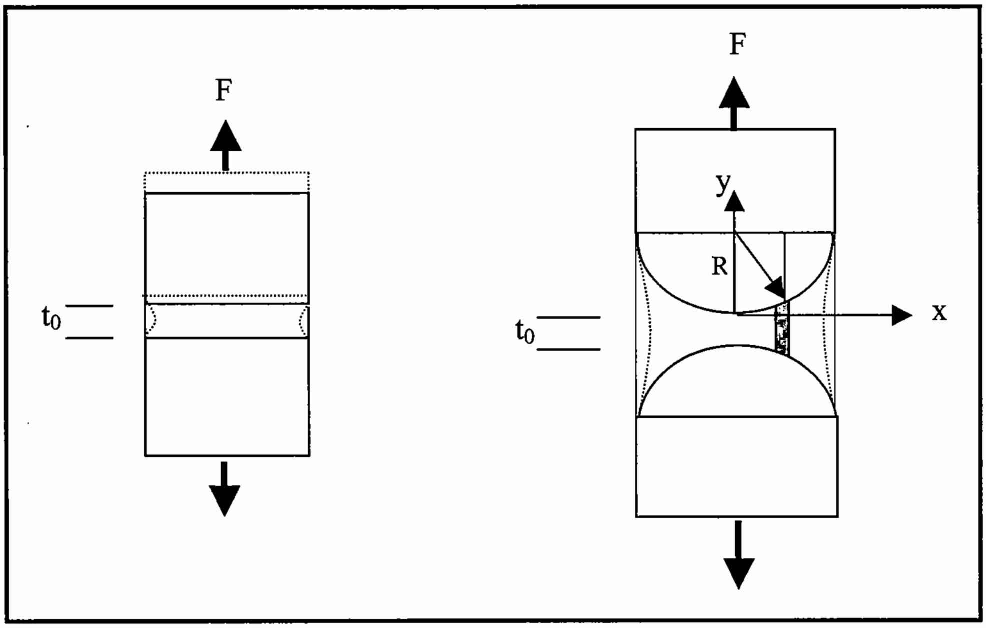


Figure 5.11 Applied load on specimen for both flat and rounded ends

If the gap opens by Δt then:

the strain (ε) and the average stress (σ) for the flat ends can be calculated using the following normal stress and strain equations:

$$\epsilon = \frac{\Delta t}{t_0} \tag{5.1}$$

$$\sigma = \frac{F}{\pi . R^2} \tag{5.2}$$

Where

t_o = original thickness of the specimen

F = the measured load

R = radius of the platen

Δt = measured displacement due to the applied load F

the strain (ε) and the average stress (σ) for the rounded ends can be estimated from the following equations since the strain is a function of distance between platens, which varies with radial distance:

$$\epsilon(t) = \frac{\Delta t}{t} \tag{5.3}$$

$$t(x) = t_0 + R - \sqrt{R^2 - x^2} \tag{5.4}$$

at $x = 0$ $\qquad t(0) = t_0$

Therefore, the strain at any distance x is approximately equals to:

$$\epsilon(t) = \frac{\Delta t}{t_0 + 2(R - \sqrt{R^2 - x^2})} \tag{5.5}$$

$$\sigma = S_b . \frac{\Delta t}{t_0 + 2(R - \sqrt{R^2 - x^2})} \tag{5.6}$$

where

t = the thickness at any radial distance x

S_b = bitumen stiffness

5.5 Testing Results and Discussion

5.5.1 Flat End Testing

Bitumen has been subjected to fatigue testing using flat end platens at temperatures of 0°C and +10°C and frequencies of 1, 5, 10, and 15Hz. Different load levels were applied and the generated strains were collected. Because the testing machine was not capable of controlling the load actuator to stop the test after specimen failure occur, necessitating regular observation of test progress, therefore a minimum load value of 50 N was used to prevent excessive test duration.

The results of the fatigue testing show that frequency has little effect on binder fatigue life in a stress controlled loading test. Figures 5.12 and 5.13 show fatigue data at four different frequencies plotted against initial stress and initial strain respectively. It is clear that when plotted against stress the data lie on a single line and, therefore, that fatigue of bitumen is much nearer to being stress dependent than strain dependent. Similar results obtained from the DTT test where the brittle failure stress doesn't vary much over a range of strain, i.e. stress dependent. But because bitumen stiffness is frequency dependent, hence different fatigue lines are expected when plotted against initial strain.

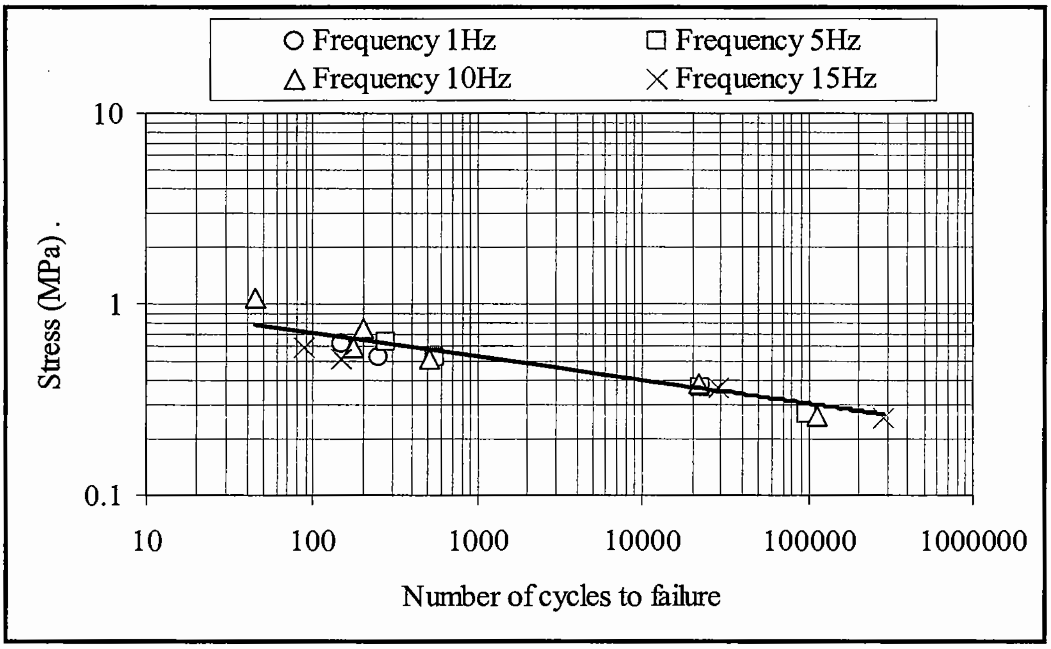


Figure 5.12 Effect of different frequency on bitumen fatigue tested at +10°C and 10Hz

The fatigue resistance of a bituminous mixture is generally defined as the ability of the mixture to respond to repeated traffic loads without significant cracking. It is commonly found that a reasonably linear relationship exists between applied stress and number of cycles to failure when plotted on logarithmic scales [4]. The results generated using this equipment for studying binder fatigue can also be approximately represented as straight lines in logarithmic space, implying a power law type relationship. This relationship can be expressed as:

$$N = k_1 \text{ x } (\frac{1}{\sigma})^{k_2} \tag{5.7}$$

where

N = number of cycles to failure obtained at a particular stress level;

k_1 = constant which determines the position of the line;

k_2 = slope of the fatigue line; and

σ = maximum amplitude of applied stress

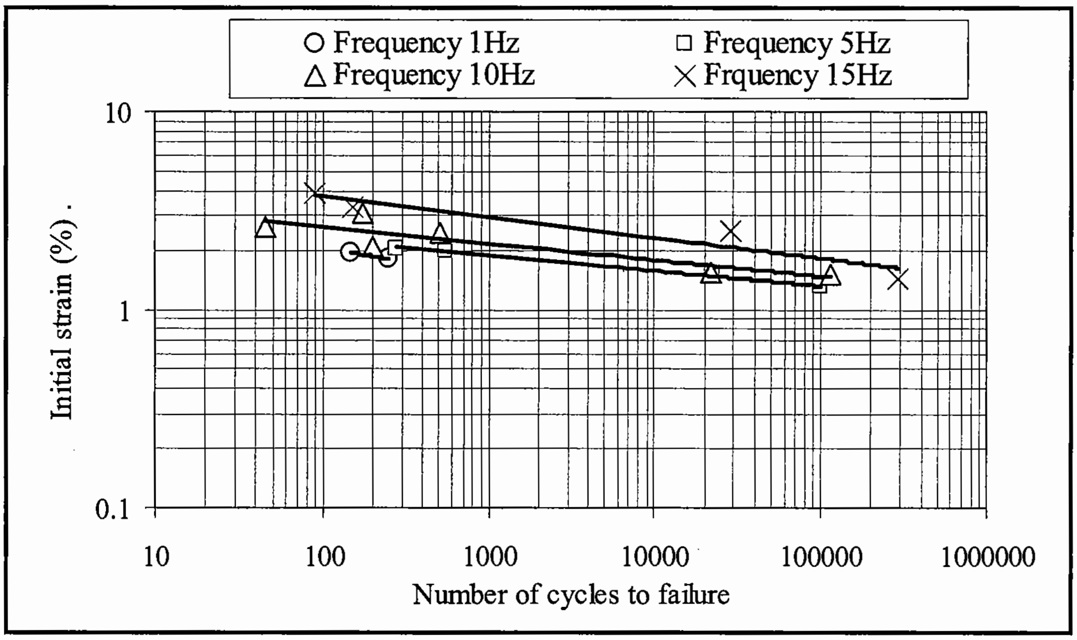


Figure 5.13 Initial strain Vs number of cycles to failure for bitumen fatigue tested at +10°C

The initial results obtained are in terms of stress against the number of cycles to failure. As the tensile strain generated in the specimen was clearly also an important parameter, therefore this was also plotted against number of cycles to failure for some tested materials.

The effect of temperature on fatigue life of binder is shown in Figure 5.14. The bitumen was tested only at 0°C and +10 °C, but the results indicate that fatigue life of the bitumen increased slightly (for a given stress level) as the temperature decreased. On the basis of these results it can be seen that temperature, may play a different role from that of frequency, with the fatigue lines for the same material being parallel but separated at different temperatures.

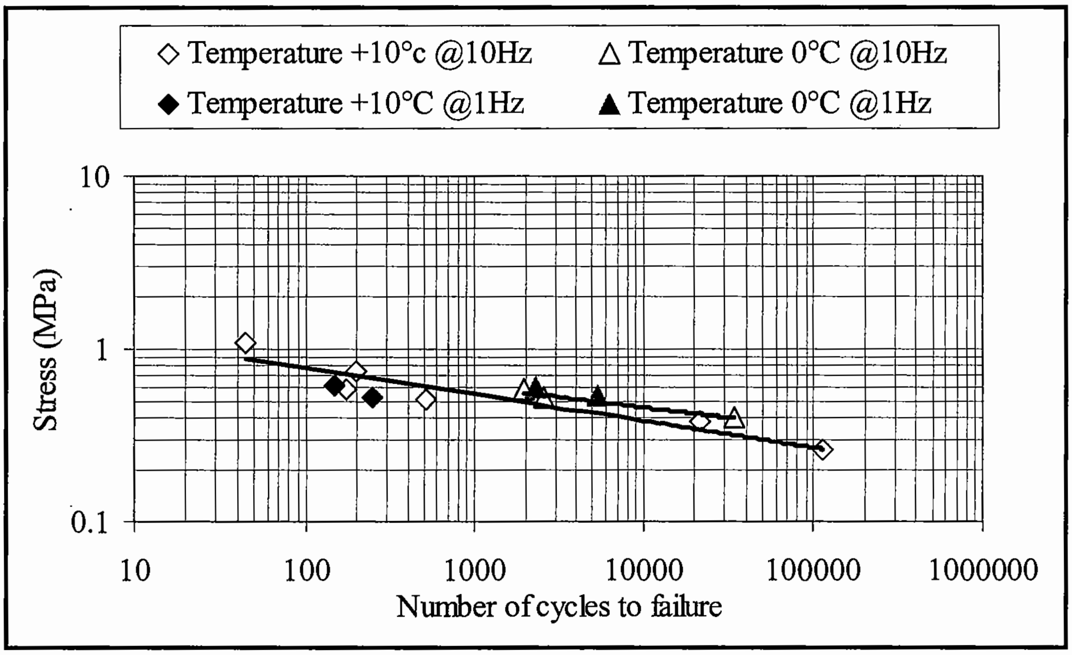


Figure 5.14 Effect of temperature on bitumen fatigue life tested at frequency 10Hz and 1Hz

Different limestone filler concentrations were added to study the fatigue of bitumen/filler mortar. Figure 5.15 shows the results of the stresses plotted against number of cycles to failure for limestone mortars tested at +10°C and 10Hz. It is clear that there are some differences in the fatigue characteristics of the mortars. The results show that high filler content mortar gave more fatigue life compared to pure bitumen and different fatigue lines were generated with different slopes. 65% limestone mortar generated a steep fatigue line compare to the others, whether plotted against stress or strain.

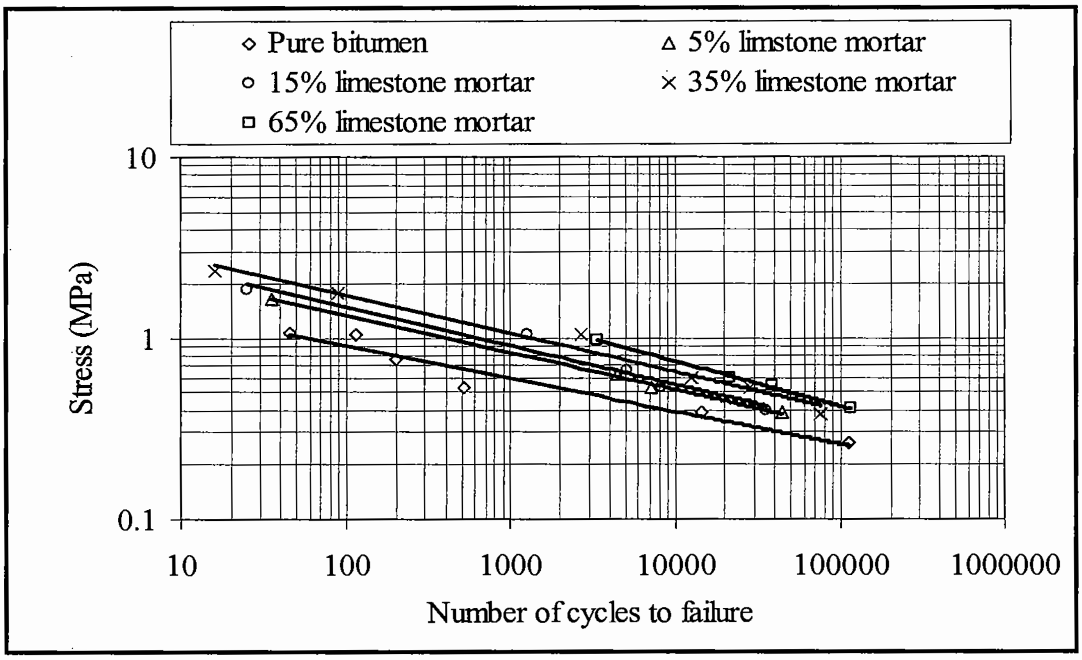


Figure 5.15 Effect of filler content on the fatigue life for mortar tested at +10°C and 10Hz

A much flatter fatigue line was obtained from the pure bitumen compared to the mortars when initial strain was plotted against number of cycles to failure. This indicates that bitumen is more sensitive to strain developed inside the specimen as shown in Figure 5.16. Another investigation into the effect of temperature on fatigue life of mortar was carried out on a mortar containing 65% limestone filler tested at +10°C and +20°C at 10Hz. Figure 5.17 shows the temperature effect on the fatigue life. The results are in the form of parallel lines, similar to those obtained when pure bitumen was tested at 0°C and +10°C. Only specimens containing 65% limestone filler were used because in trials on pure bitumen at 20°C the specimens deformed excessively under the self weight of the actuator.

As the fatigue life of the mortars tested can be presented as a line having a power law relationship (see equation 5.7) therefore, from test data the constants or the fatigue parameters (k_1 and k_2) can be determined. It can be seen from the equations that the slopes of the fatigue lines for the mortars are very similar except for 65% limestone filler concentration which has a steep slope.

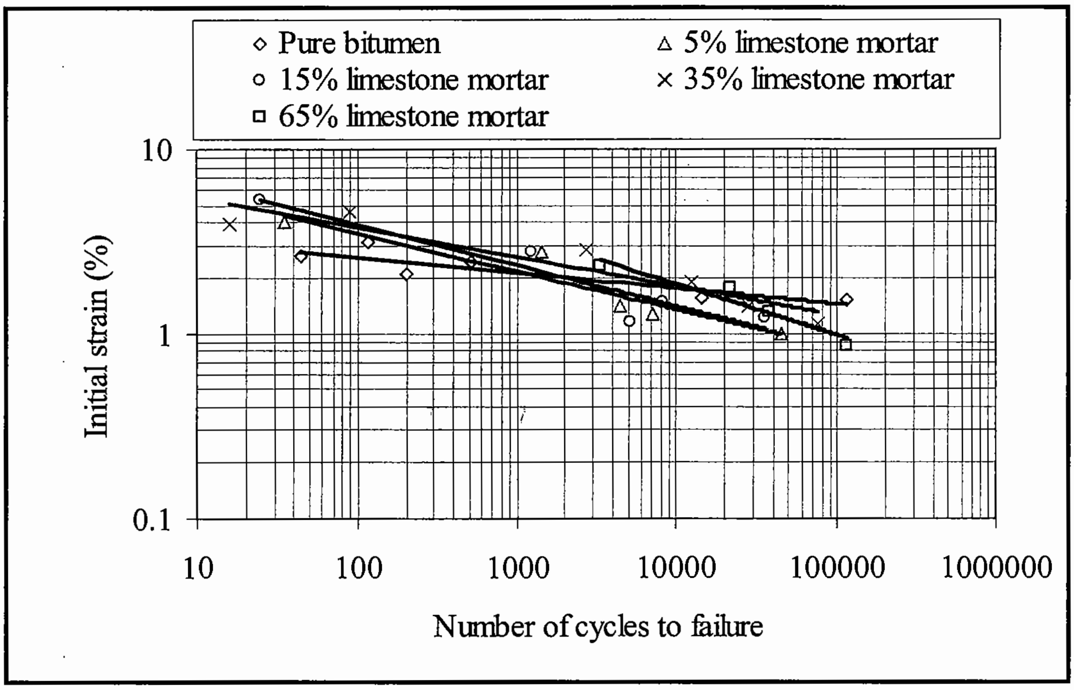


Figure 5.16 Initial strain Vs number of cycles to failure for limestone mortar tested at +10°C and 10Hz

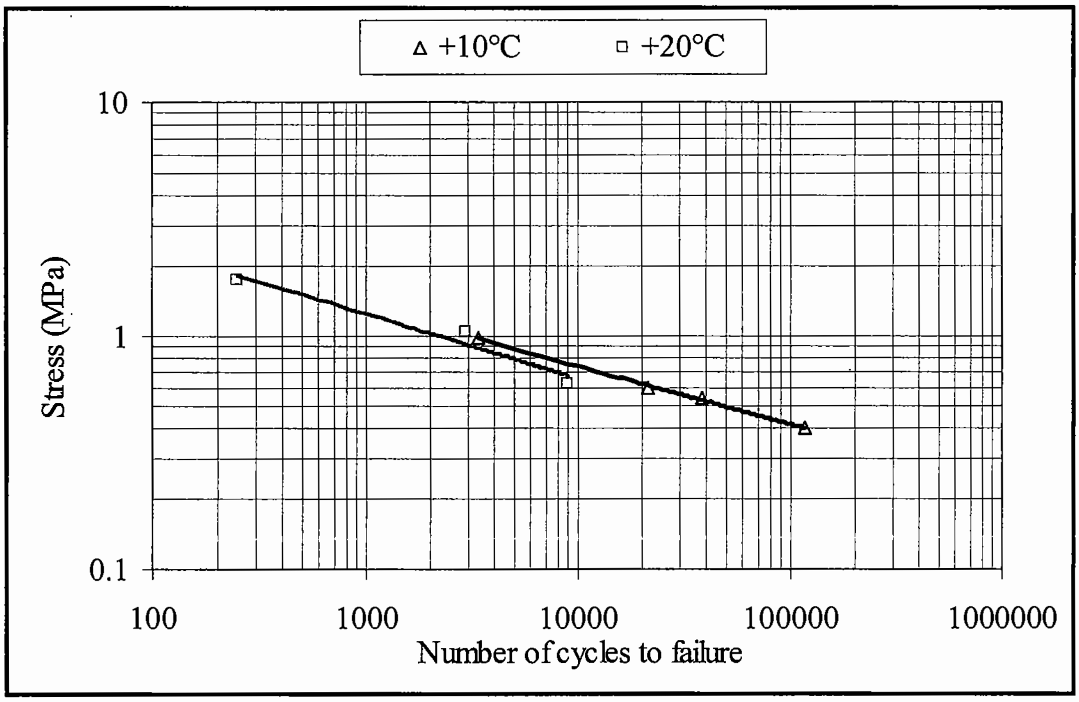


Figure 5.17 Effect of temperature on mortars containing 65% limestone filler tested at 10Hz

Table 5.1 presents the stress and strain after 100 cycles and one million cycles of testing using the fatigue equations as reported above. Pure bitumen has a high strain value at one million cycles as the material had low stiffness compared with the mortars. The inclusion of the filler particles into the bitumen as solid bodies forces the strain to occur in the bitumen around the solid particles, which enables the mortar to take higher stress as the crack surface area necessarily increases.

Table 5.1 Predicted stresses, strains and number of cycles to failure for pure bitumen and mortar containing different limestone filler contents

Material	**Stress (MPa)**		**Strain (%)**		**N_f**
	$N_f = 100$	**$N_f = 10^6$**	**$N_f = 100$**	**$N_f = 10^6$**	**$\varepsilon = 100x10^{-6}$**
Pure bitumen	0.8919	0.1715	2.551	1.170	3.69 E + 22
5% mortar	1.3228	0.1996	3.50718	0.5149	1.34 E +13
15% mortar	1.4721	0.2051	3.934	0.5000	1.54 E +11
35% mortar	1.7284	0.2426	3.779	0.86416	2.42 E +16
65% mortar	2.3437	0.2335	6.638	0. 5153	1.03 E + 10

To study the effect of filler type on fatigue life, gritstone filler and cement were used in the research with a concentration of 35% by mass. The results shown in Figure 5.18 indicate that in terms of stress filler in general increases the fatigue life of the binder by more than 25 times (at high stresses) regardless of the filler type. The filler type generally had little effect on the fatigue life of the tested specimens in this research as shown in Figure 5.18, where the fatigue lines lie very close together. A plot of initial strains were against number of cycles to failure is shown in Figure 5.19.

There is a slope difference between the fatigue lines of the bitumen and the mortars. But it can be said that the slopes of the lines of the mortars are almost the same against stress, although different slopes are generated when considering the initial strains. The results show that the filler type has only a small effect on fatigue life and it seems to be the filler concentration, which has a greater effect on fatigue life rather than the filler type.

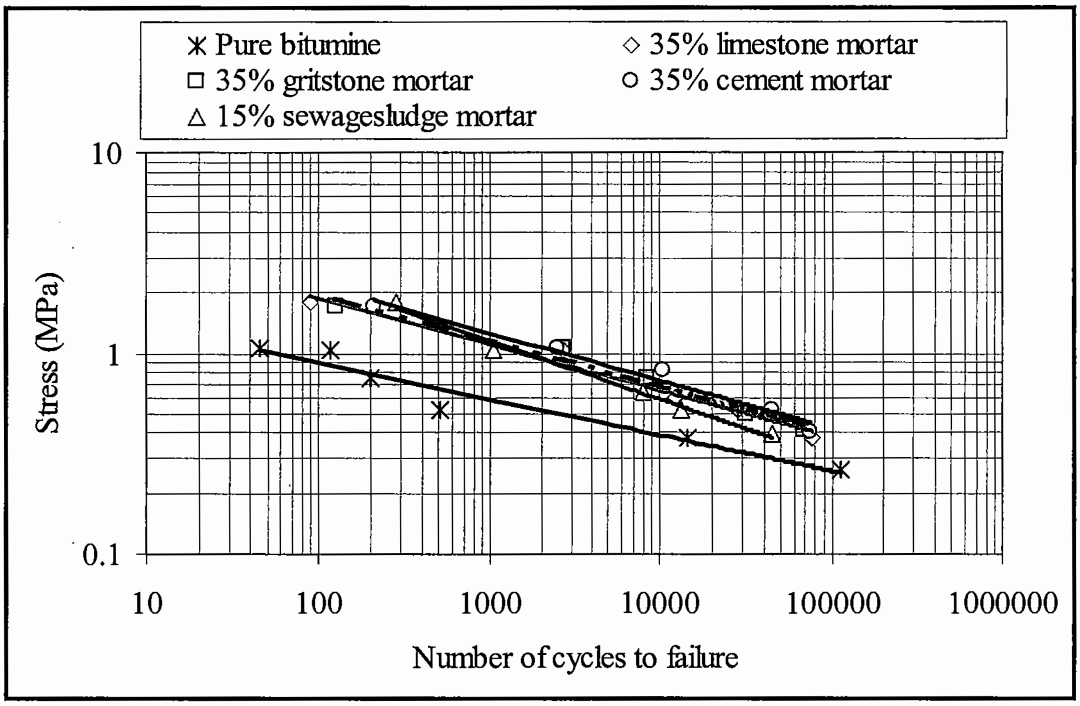


Figure 5.18 effect of filler type on fatigue life for mortar containing different fillers tested at +10°C and 10Hz

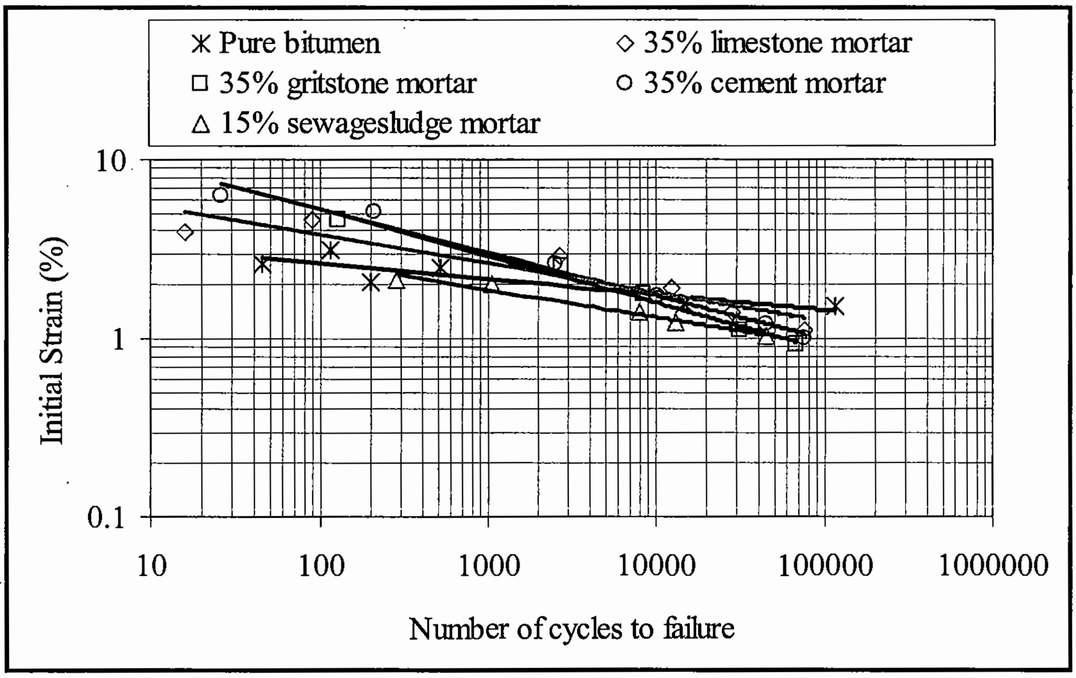


Figure 5.19 Initial strain Vs number of cycles to failure for mortar containing different fillers tested at +10°C and 10Hz

A mortar containing 15% sewage sludge ash, by mass, was also used in this research to see its effect on fatigue life. Figures 5.18 to 5.20 show the results of sewage sludge ash mortar compared to other mortars. It is clear that at the same concentration (Figure 5.20), both limestone and sewage sludge ash filler gave reasonably close fatigue lines, slightly higher for sewage sludge mortar. However, the sewage sludge fatigue life with this concentration is very close to the mortars containing 35% limestone, cement or gritstone. The particle size distributions for the fillers indicate that sewage sludge ash is coarser than the other filler types, which means this effect is not only a surface area issue. The main cause of this extra fatigue life of sewage sludge ash is probably due to the particles' shape or surface texture.

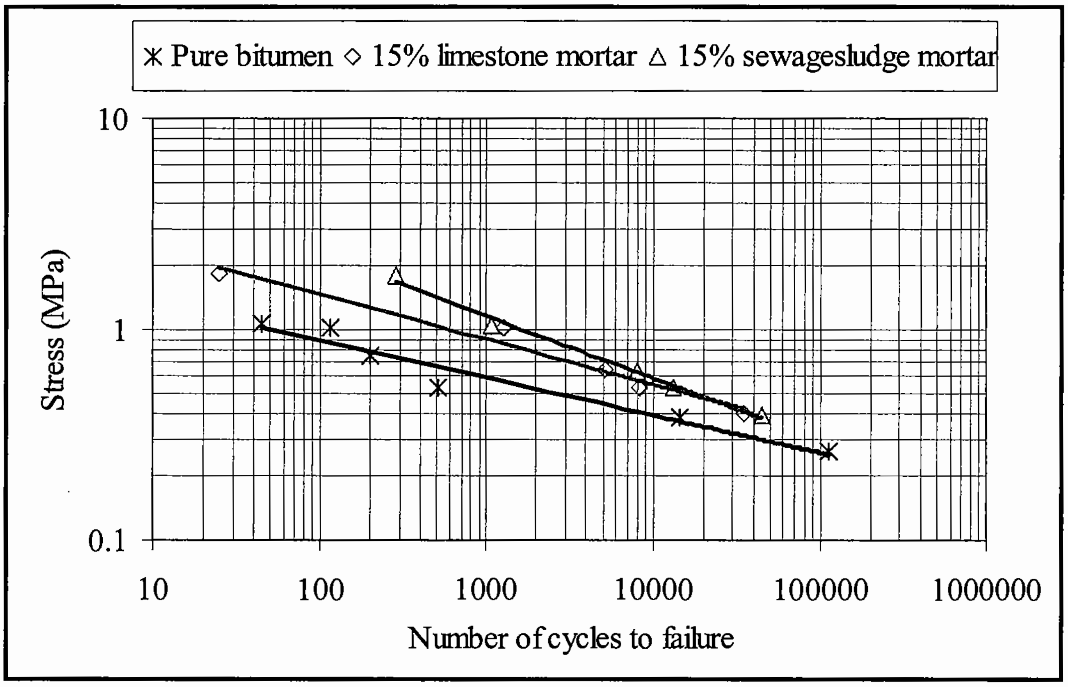


Figure 5.20 fatigue life for mortars containing 15% limestone and sewage sludge ash tested at +10°C and 10Hz

As discussed above, the fatigue life of the mortars tested can be presented as a line having a power law relationship. It can be seen from the equations that the slopes are very similar except for sewage sludge filler which is slightly different from the others.

Table 5.2 presents the stress and strain after 100 cycles and one million cycles of testing using the fatigue equations. The results indicate that filler mortars have similar stress to failure after one million cycles, although limestone mortar had a higher strain value at one million cycles compared with others containing the same amount of filler concentration (35% by mass).

Table 5.2 predicted stresses, strains and number of cycles to failure for different mortars

Material	Stress (MPa)		Strain (%)		N_f
	$N_f = 100$	$N_f = 10^6$	$N_f = 100$	$N_f = 10^6$	$\varepsilon = 100x10^{-6}$
Limestone mortar	1.7284	0.242577	3.7791	0.86415	2.42 E +16
Gritstone mortar	1.9626	0.238802	5.3205	0.470279	1.60 E +12
Cement mortar	2.1728	0.244467	5.24296	0.57224	1.28 E +13
Sewage sludge mortar	2.2858	0.151857	2.644477	0.651538	3.36 E +17

Generally for a fatigue test under controlled load, the strain increases as loading is applied and this results in decreasing stiffness of the material until the specimen fails. Figure 5.21 shows the results of 35% gritstone mortar tested at +10°C and 10Hz. It was observed that there was a stiffness reduction for all levels of loading. The reduction in stiffness was also observed when different filler mortar types were tested as shown in Figures 5.22 and 5.23, which show the results for load levels of 75N and 100N respectively.

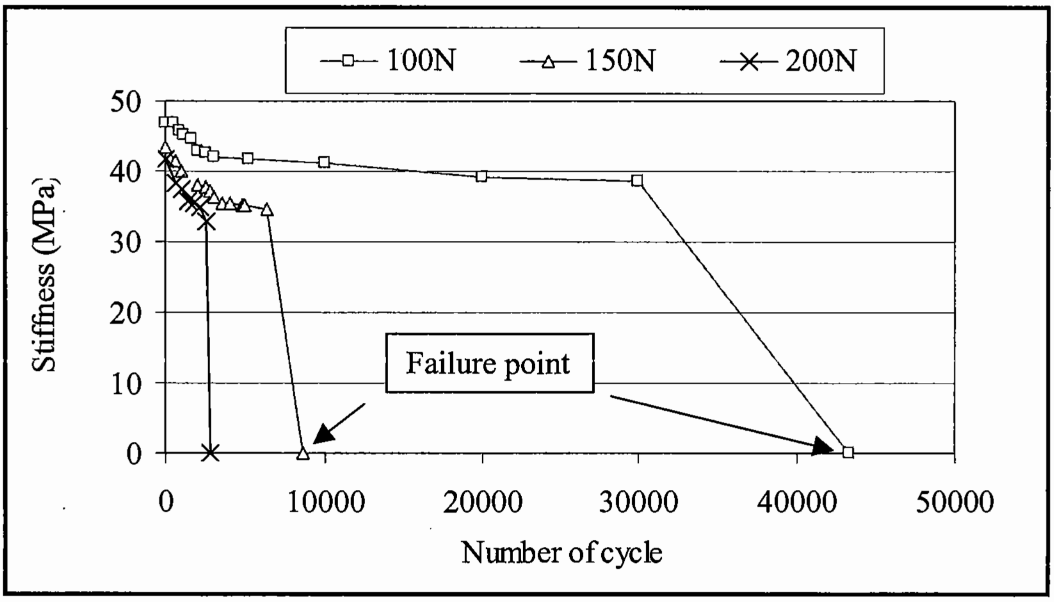


Figure 5.21 Stiffness Vs Number of cycles for a mortar containing 35% gritstone filler tested at +10°C, 10Hz and different load levels

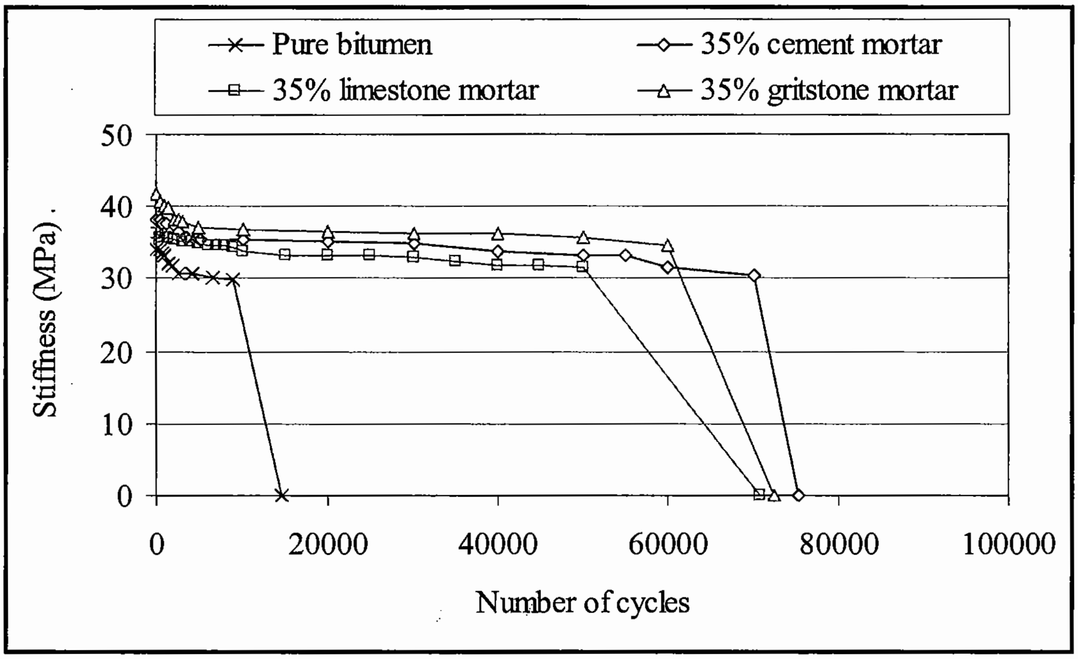


Figure 5.22 Stiffness Vs number of cycles for mortar tested at +10°C, 10Hz at a load of 75N

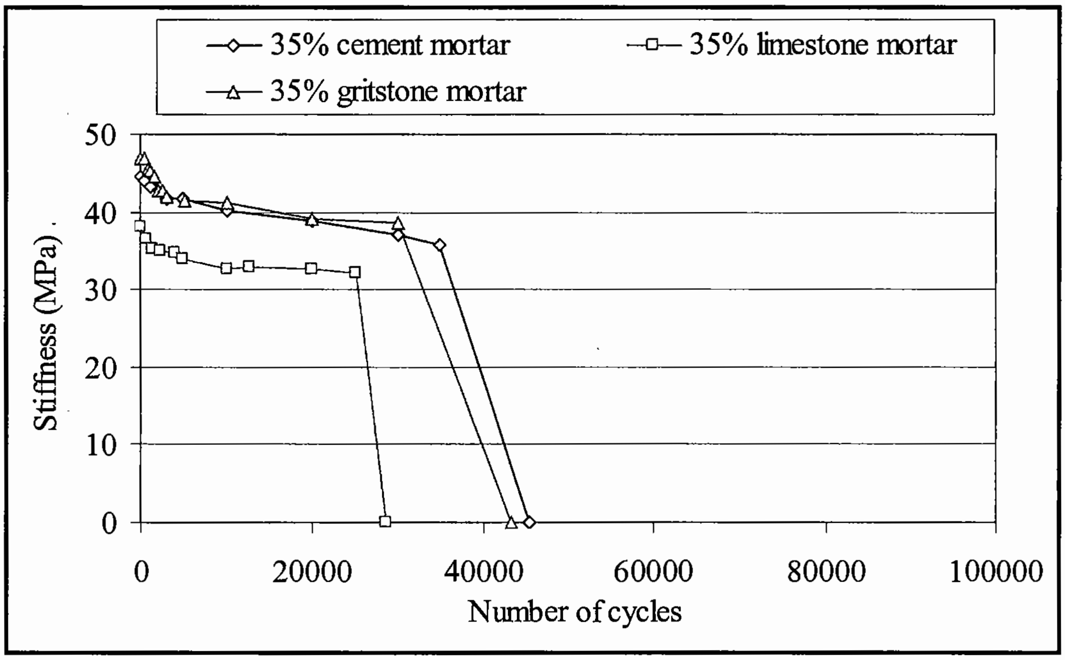


Figure 5.23 Stiffness Vs number of cycles to failure for bitumen/filler at +10°C, 10Hz at a load of 100N

Figure 5.24 shows the stiffness reduction of different concentrations of limestone mortar compared to bitumen when tested at a temperature of +10°C and frequency 10Hz. Other researchers also observed the stiffness reduction phenomenon in asphalt mixture fatigue tests. This reduction in stiffness is assumed to be due partially to self-heating of material generated during loading.

The failure surface appeared visually different for pure bitumen and bitumen-filler mortar. Figures 5.25 and 5.26 show surface failure for bitumen and mortar tested at +10°C. Both failed clearly but the bitumen was slightly more ductile as shown by the raised area at the centre of the failure surface.

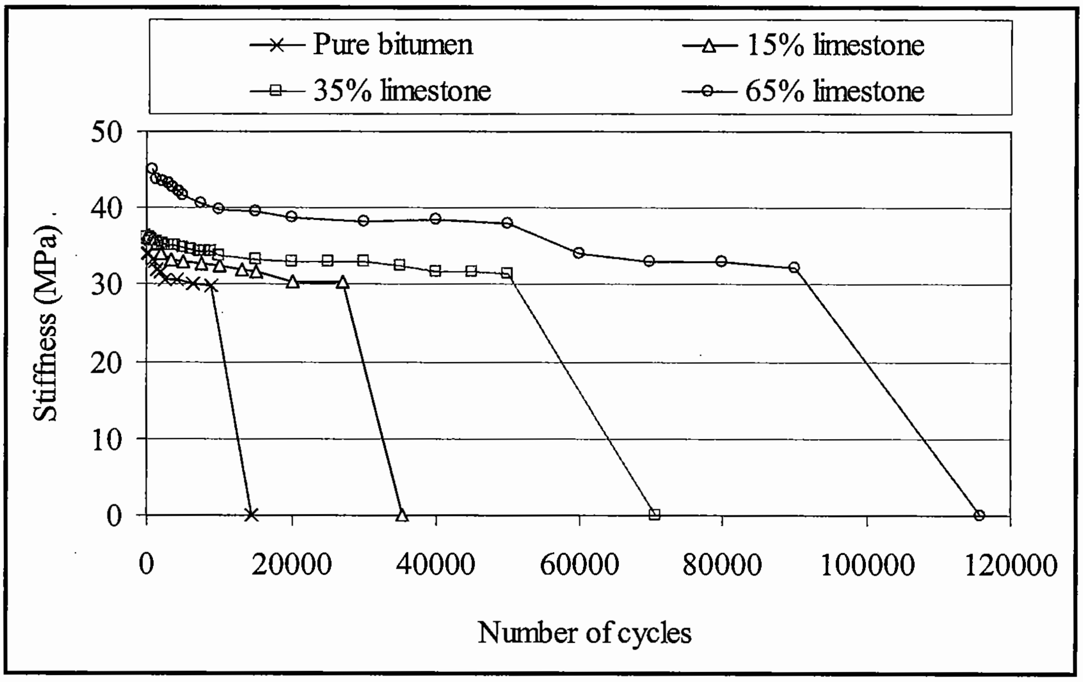


Figure 5.24 Stiffness Vs number of cycles for limestone mortar tested at +10°C, 10Hz at a load of 75N

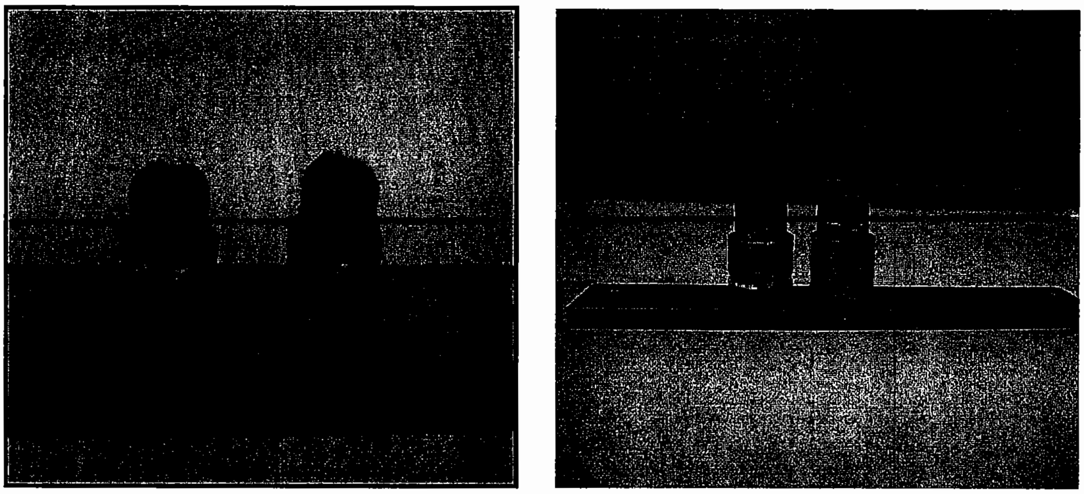
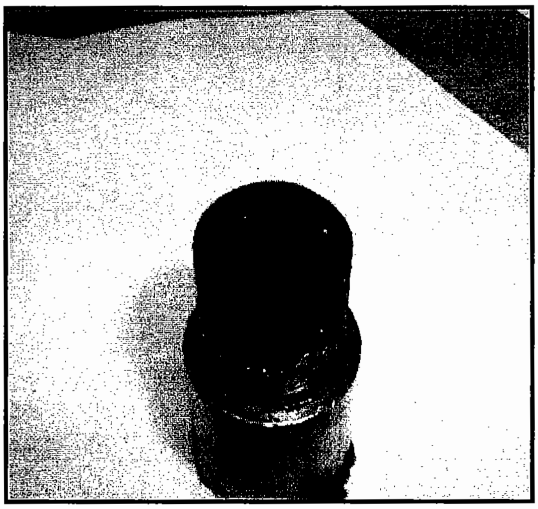


Figure 5.25 Surface failure of pure bitumen tested at +10°C and 10Hz

(a) (b)

Figure 5.26 Surface failure of mortars: a) 35% limestone mortar, b) 65% limestone mortar tested at +10°C and 10Hz

5.5.2 Hemi-spherical End Testing

The hemi-spherical end tests have been carried out for pure bitumen and limestone filler mortar only with 15%, 35% and 65% filler concentration by mass. The tests were carried out at a temperature of +10°C and a frequency of 10Hz and at different load levels. Figure 5.27 shows the results of testing pure bitumen and limestone mortar, plotting stress against number of cycles to failure. The results indicate that filler content has a great effect on fatigue life of the binder. The limestone mortar with 65% filler concentration has a much higher influence on fatigue life of the binder compared to 15% and 35% filler contents.

5.5.3 Flat and Hemi-spherical Ends Comparison

Figures 5.28 and 5.29 show the comparison of flat end and hemispherical end test results for bitumen tested at +10°C and 10Hz. The fatigue life of bitumen for both types of tests is almost similar when plotted against average stress. It was expected that lower fatigue life would be seen from the hemi-spherical ends due to the high stress concentration at the centre. The stresses calculated, using Equation 5.7, indicate that the average stress at the centre of the hemi-spherical end is 5 times that on the edge. As a research done [7] indicate that 99% of the stress is concentrated in the first 3mm radius around the vertical symmetric axis of the specimen at hemi-spherical end platens.

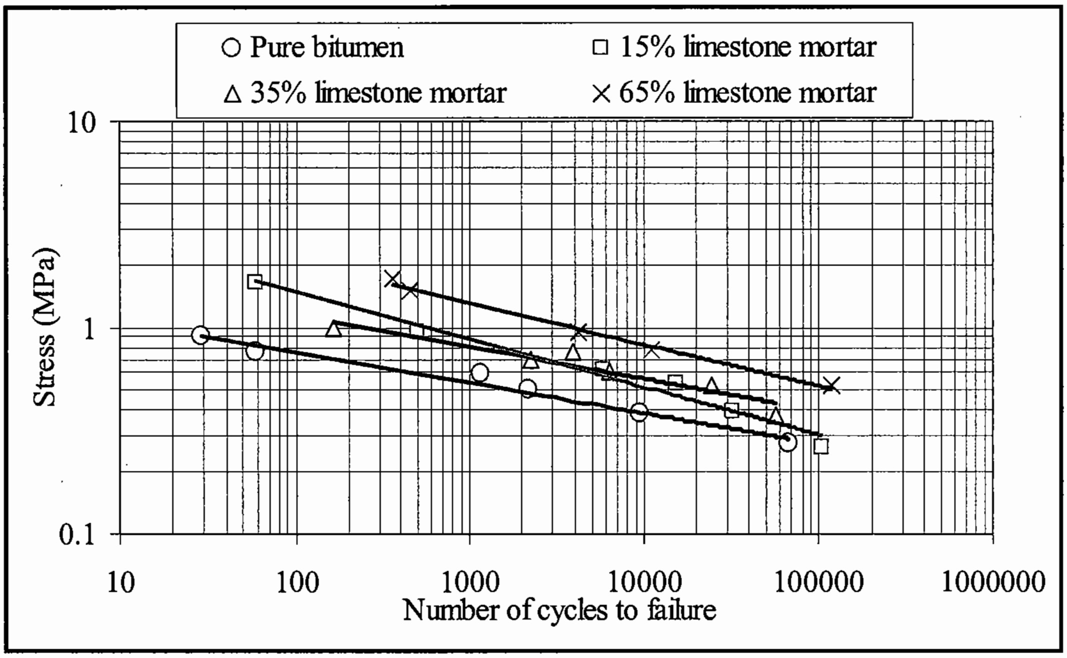


Figure 5.27 Stress against number of cycles to failure for mortars tested using hemi-spherical ends at +10°C and 10Hz

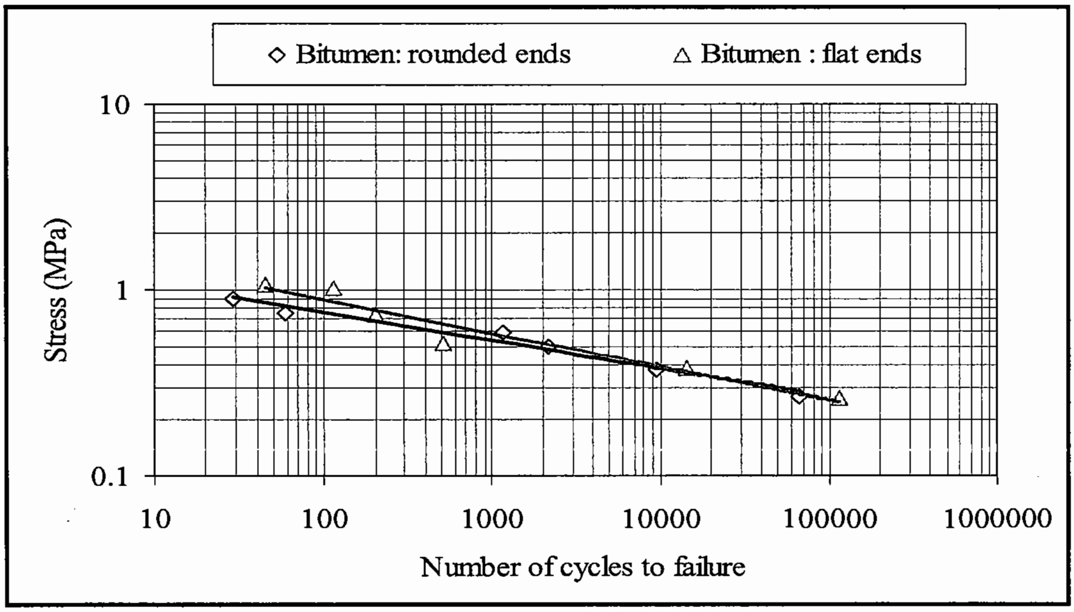


Figure 5.28 Stress against number of cycles to failure for pure bitumen tested on both flat and rounded ends at +10°C and 10Hz

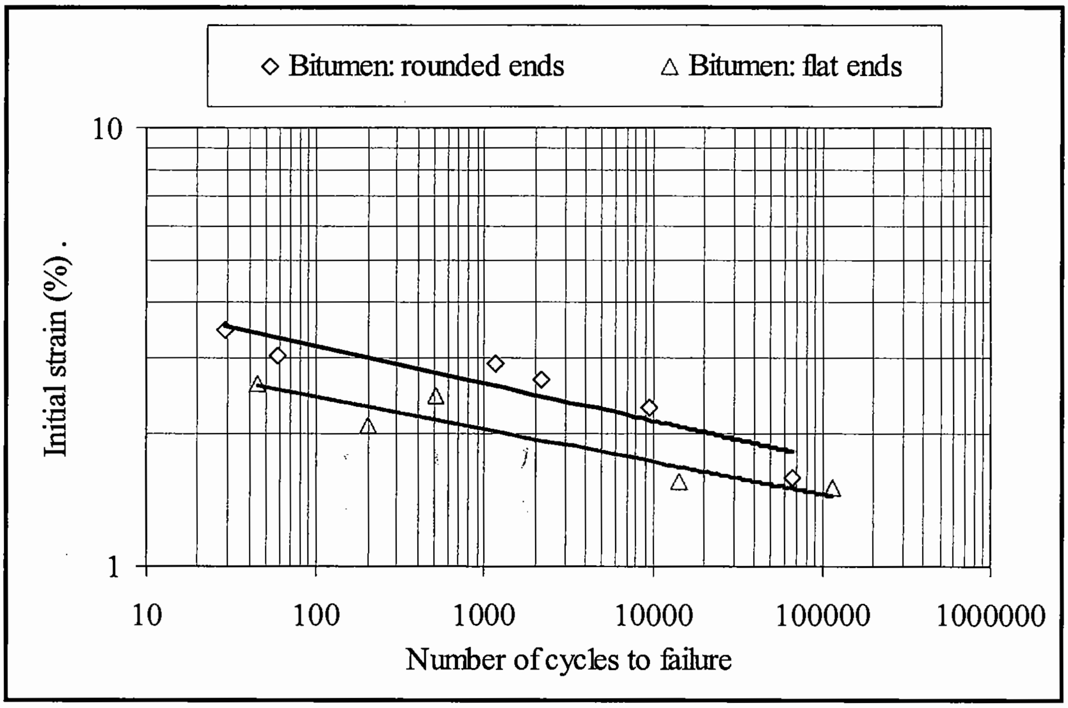


Figure 5.29 Initial strain against number of cycles to failure for pure bitumen tested on both flat and rounded ends at +10°C and 10Hz

The lack of difference in Figure 5.28 might be because of the bitumen molecular structure. Bitumen is a complex material with a complex and varied molecular structure. Under repeated loading, it is to be expected that the individual molecules would re-orientate themselves and that breakdown of cohesion would occur at those points within the material which happen to have the weakest molecular arrangement. For pure bitumen this may mean that fracture will take place immediately after the initiation of the crack on the flat end platens. But on rounded end platens much lower stresses applying away from the centre line of the test, if the crack initiate on the centre the molecules re-orientate themselves for thousands of cycles until the breakdown of the cohesion take place for all the molecules on the crack line.

For both flat and rounded end platens, the test results also show that for the mixtures containing 15% and 35% limestone filler there is no big difference in fatigue life (against stress); both increased the resistance to fatigue failure of the binder as shown in Figures 5.30 and 5.31.

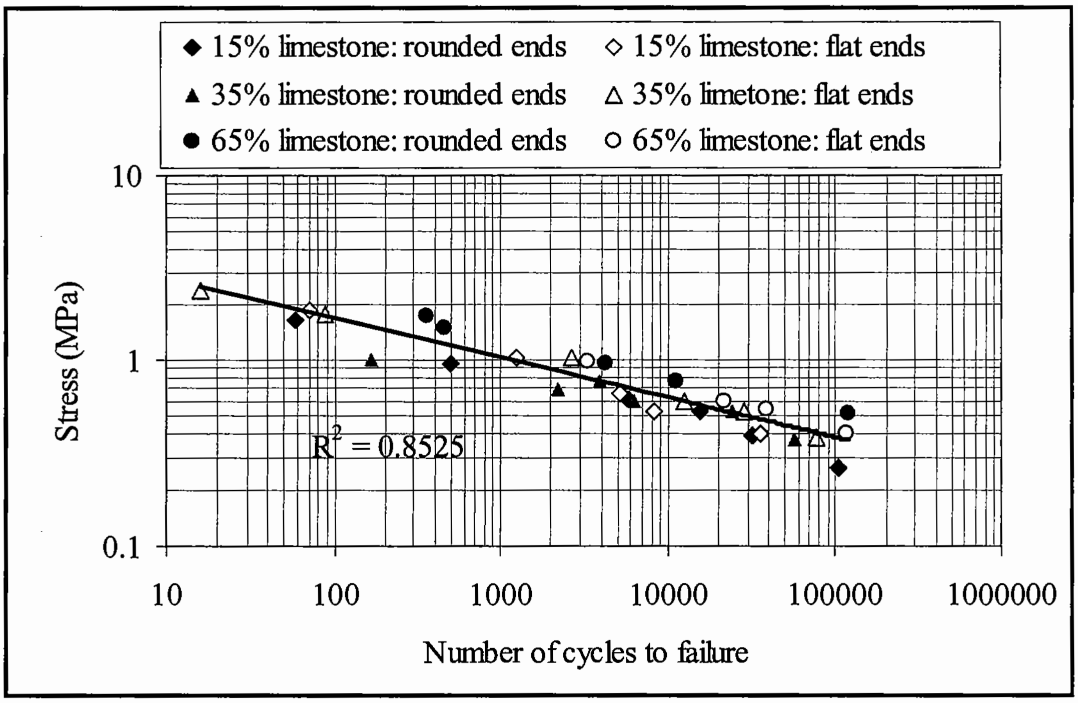


Figure 5.30 Stress against number of cycles to failure for bitumen filled with limestone filler tested on both flat and rounded ends at +10°C and 10Hz

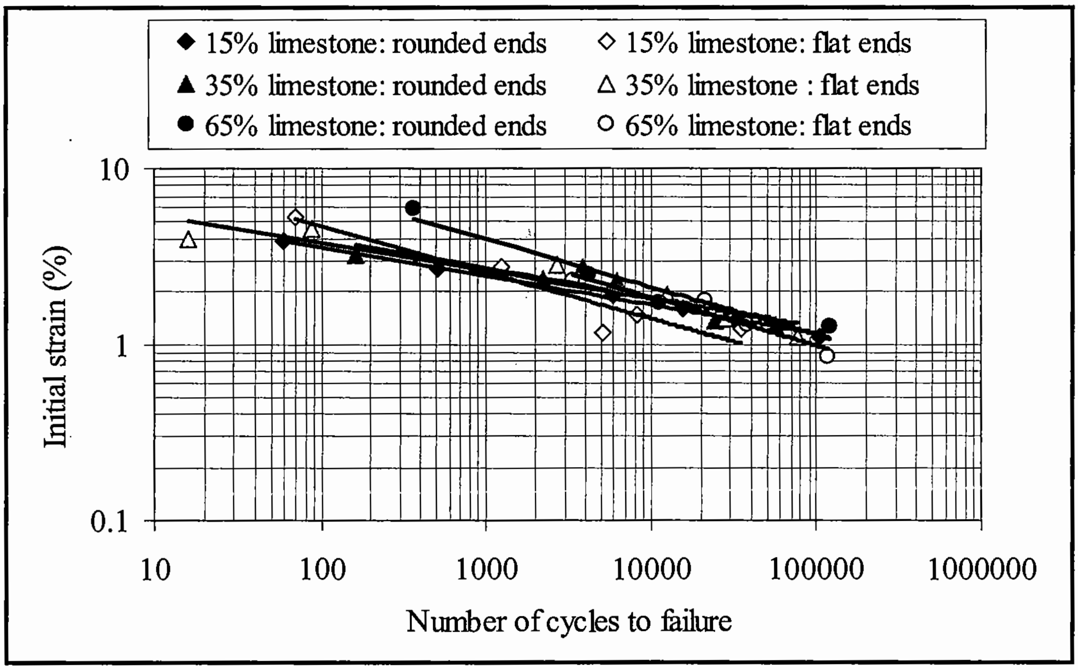


Figure 5.31 Initial strain against number of cycles to failure for bitumen filled with limestone filler tested on both flat and rounded ends at +10°C and 10Hz

Table 5.3 presents the stress and strain after one million cycles of testing using the fatigue lines equation as reported above. It can be observed that after one million cycles the stresses generated on both flat and rounded ends are fairly similar but strains obtained from rounded ends are relatively higher than on the mortar tested on flat end platens. If a higher value of strain (after one million cycles) as an indicator for better fatigue resistance [134], the 35% limestone mortar gave better fatigue resistance compared with other mortars.

Table 5.3 Predicted stresses, strains and number of cycles to failure for different mortars

Material	Stress (MPa) $N_f=10^6$		Strain (%) $N_f=10^6$	
	Flat end	Rounded end	Flat end	Rounded end
Pure bitumen	0.1715	0.192	1.170	1.4409
15% Limestone mortar	0.2051	0.176	0.500	0.7869
35% Limestone mortar	0.2426	0.273	0.864	0.8585
65% Limestone mortar	0.2335	0.324	0.515	0.68041

It was observed that a reduction in stiffness occurred during the hemi-spherical end tests at each level of load, the same phenomenon as was observed in the flat end tests. A rise in temperature due to energy dissipation has been suggested. However, computations carried out for the uniaxial test equipment indicate that a 0.8°C rise is the maximum likely. It may be that the true explanation for much of the stiffness reduction is related to activity on a molecular scale.

5.6 Summary

Bitumen and bitumen/filler mortar have been tested using both flat and rounded end platens at a temperature of +10°C and a frequency of 10Hz at different load levels. Four types of filler have been used in the testing programme for the flat end tests with different filler concentrations, but only limestone filler has been used in the rounded end tests with 15%, 35% and 65% filler contents by mass.

The key points are:

- The frequency effect on fatigue life has been checked for the binder only. The results show that frequency does not have any measurable effect on the fatigue life for the materials tested in this research, when plotted against stress.
- The results of the flat end tests show that filler type does not have a great effect on fatigue life, as the mortars containing cement, limestone, and gritstone gave almost the same fatigue life with no significant difference. But filler content has an appreciable effect on the fatigue life; limestone with different filler contents shows this effect. However 15% sewage sludge ash has a large effect on binder fatigue life compared with the other fillers, which were tested with 35% concentration.
- In general, the fatigue life of the binder was increased by the addition of filler, but different fatigue lines were obtained with different slopes depending on the filler content. The strain increased during fatigue tests and this generated a reduction in stiffness that was observed at all load levels. This reduction in stiffness is considered to be partially due to the self-heating of the materials due to cyclic loading.

6

Fatigue of Bituminous Mixtures

6.1 General

This Chapter presents tests for the fatigue of bituminous mixtures with a detailed description of the equipment used in this research, the 4-point bend test, developed at the University of Nottingham. It also includes fatigue testing using the ITFT test method. Description of the materials used, the mix design and method of testing are given. The results of both testing programs are presented with discussions and summary.

Fatigue failure is known to occur in an asphalt pavement under repeated loading at in-service temperature. Considerable research has been done to develop theoretical, experimental laboratory testing techniques, and data analysis methods to predict the fatigue performance of asphalt mixtures [9,113,123,124,125].

The fatigue behaviour of an asphalt mixture is generally established using different fatigue tests as described in Chapter 3, section 3.7.2. Failure (or the number of cycles to failure) in fatigue testing has been defined in various ways and the value cited depends on the mode of loading.

In strain control mode, since the maximum strain stays constant and the stress decreases during the fatigue test, defining fatigue failure is harder. Again researchers have adopted several different fatigue failure definitions. The most common and widely used definition for fatigue failure is that of 50% reduction in the initial stiffness.

According to the proposed research program [115], a laboratory fatigue test other than the ITFT was supposed to be used, such that the test should simulate a pavement layer under traffic load. SHRP carried out a survey of the most recognised and widely used methods for measuring fatigue properties of asphalt mixtures. They ranked the methods based on governing parameters. The survey showed that repeated flexure tests were ranked highest followed by the ITFT test in second place.

The 4-point bend test is one of the flexural tests developed to study the fatigue of an asphalt mixture. The advantages of these tests are that they simulate to a certain extent the stress state in the lower part of the bound layer (large horizontal tensile stresses combined with small vertical compressive stresses).

Oliveira [126] conducted research at Nottingham University to study the fatigue of Densiphalt (a proprietary grouted macadam system) using the principle of simple flexure. He developed a new test equipment to study the fatigue of the mixture using the 4-point bending test (also known as third point flexure). This equipment has been used in this research to study the fatigue of asphalt mixtures in parallel with the ITFT test. In this test, the failure of the specimen occurs in the uniform bending moment zone.

6.2 Materials

6.2.1 Binder

The binder used was supplied by Nynas Ltd, and was the same as that used in the DTT and mortar fatigue tests. It was a straight run 50pen grade bitumen. See Chapter 4 section 4.4 for the binder properties.

6.2.2 Fillers

Filler can influence a mixture by modifying the grading of the fine aggregate thus producing a denser mixture with greater aggregate contact. This will also influence the demand of binder.

For this study, three types of filler were used to study the fatigue properties of asphalt mixture. Limestone, gritstone and cement were used in the testing program. The material properties and gradations were described in detail in Chapter 4 section 4.4.

6.2.3 Aggregate

The aggregate used in this research was crushed limestone. The aggregate gradation is shown in Table 6.1. The aggregate size was selected to meet the specification of BS 4987-1 [127] for Dense Bitumen Macadam (DBM).

6.3 Mixture Design

The principal objective of a mixture design procedure is to determine the proportions (within specification limits) of the materials comprising the mixture. Mixture design involves selection of an appropriate aggregate gradation, selection of grade and amount of bitumen.

The mixture designs for the DBM materials are shown in Table 6.2. Limestone aggregate, 50pen grade bitumen and three types of filler were used to produce the mixture, with a binder content 5.2%. The DBM mixtures were designed according to the specification for 10mm size close graded wearing course BS 4987-1 [127]. The target air voids content was set at 4%. The grading curves for the DBM mixtures are shown in Figure 6.1.

Density of Compacted Mixture

Bulk density of a bituminous mixture can be calculated by dividing the mass of a specimen by its volume. It is used to calculate the maximum void content of the mixture. In this study, the dry method was used to find the bulk density according to BS EN 12697-6:2003, using the following equation and determining the mass of the specimen in air and water.

$$\gamma_b = \frac{M}{V} \tag{6.1}$$

where

γ_b = bulk density of the asphalt mixture

M = mass of the asphalt mixture

V = volume of asphalt mixture

Theoretical Maximum Density of Compacted Mixture

This is used to calculate the air voids in the asphalt mixture. It can be obtained using either the theoretical method or from the laboratory. Theoretically, it can be calculated from the following equation.

$$\gamma_{max} = \frac{100 \times \gamma_w}{\frac{P_s}{G_{se}} + \frac{P_b}{G_b}} \tag{6.2}$$

Where

γ_{max} = maximum density of the asphalt mixture (zero air voids)

P_s = aggregate, percentage by total mass of mixture

P_b = bitumen, percentage by total mass of mixture

G_{se} = specific gravity of the aggregate

G_b = specific gravity of bitumen

6.3.1 Sample Preparation

The required mass of each aggregate fraction was worked out according to the predetermined gradation as shown in Figure 6.1.The mass of the materials was calculated for each slab to give the required density when compacted. The slab dimensions were 305 × 305 × 50mm (the maximum dimension of the square moulds used in the roller compactor).

Table 6.1 DBM mixture design for different filler types and contents

Constituent	Percentage by mass (%)				
	15% Limestone Filler	**35% Limestone Filler**	**65% Limestone Filler**	**35% Gritstone Filler**	**35% Cement Filler**
10mm aggregate	36	37	40	37	37
6mm aggregate	27	26	26	26	26
Dust	36	34	24	34	34
Filler	1	3	10	3	3
Bitumen	5.2	5.2	5.2	5.2	5.2

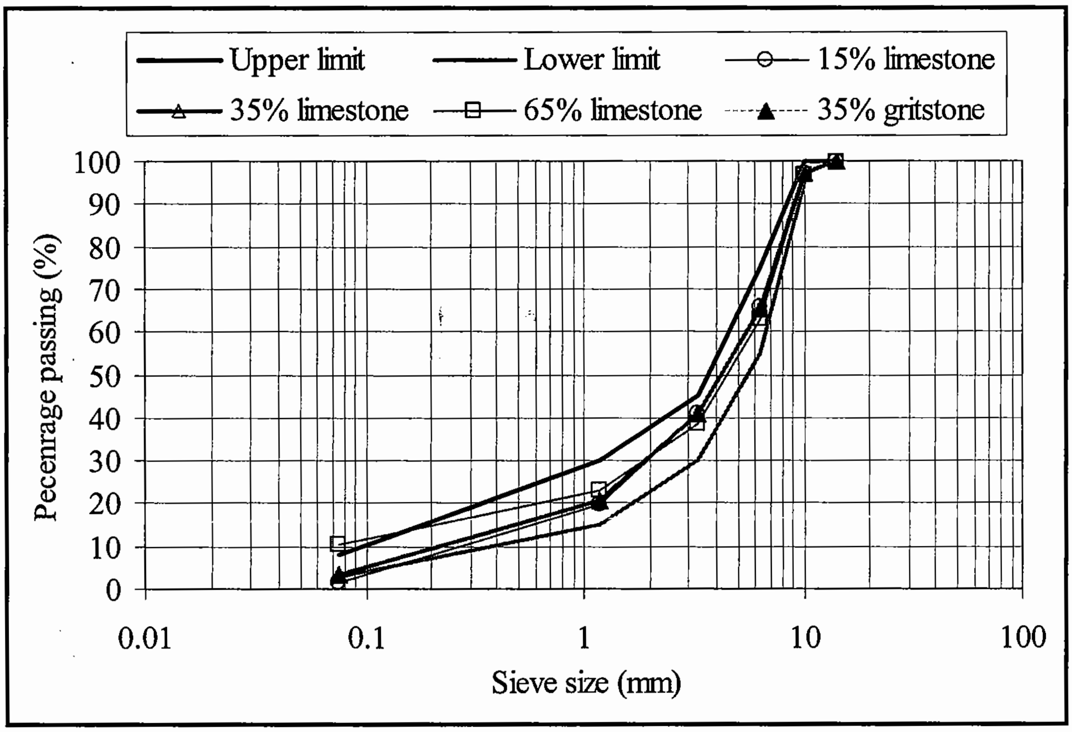


Figure 6.1 Aggregate grading for the mixture containing different filler type and contents

The dried aggregate blend was placed in containers and heated in a thermostatically controlled oven to the required temperature for mixing. The binder was also heated to the required mixing temperature. Both bitumen and aggregate were maintained at the mixing temperature for at least 4 hours. The appropriate mass of heated bitumen was mixed with the heated aggregate in a preheated mixer in proportion according to the job mix formula to achieve the filler/bitumen ratio required for the specific mix design. The mixing continued until the aggregate was fully coated with bitumen and there were no signs of uncoated particles.

The mixture was then transferred to the slab mould. The slab mould was preheated and sprayed with lubricant to prevent adhesion between the mould and the mixture. All mixtures were compacted in a slab mould using a slab roller compacter. The compactor was designed to simulate the action of a roller compactor on site. After mixture compaction, the slabs were left at room temperature to cool and then stripped from the moulds. For the 4-point bend testing program, five beams were then cut from each slab, having the dimensions 305 × 50 × 50mm as shown in Figure 6.2. The actual air void level in the sawn beam specimens was determined (by measuring the bulk density and the maximum density of each beam). For the ITFT testing program, cylindrical specimens were cored from the slabs and trimmed, having diameter 100mm and thickness 40mm. All the specimens were then stored in a temperature controlled room at +5°C until required for testing.

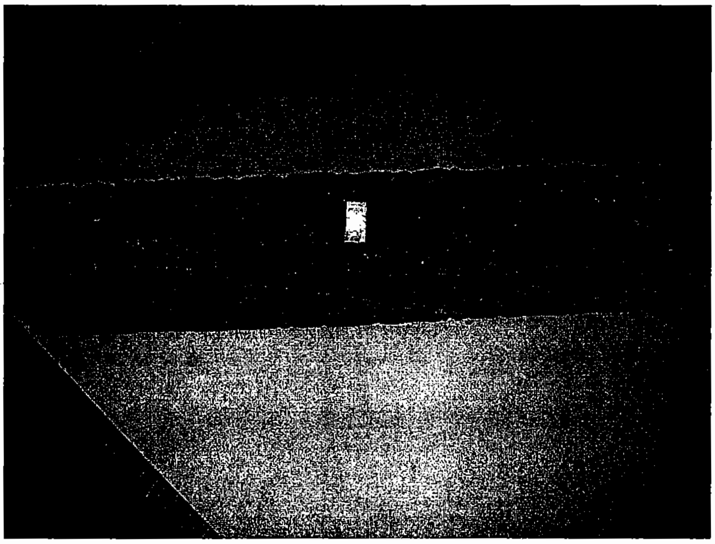


Figure 6.2 Asphalt mixture specimen for 4-point bend test

6.4 Testing Equipment

The 4-point bend test apparatus was designed by Oliveira [126] and manufactured at the University of Nottingham, in the workshop of the School of Civil Engineering. A MAND servo-hydraulic testing apparatus was used to apply load to the 4-point bend equipment. The machine enables vertical compressive and tensile forces to be applied to a specimen under static or cyclic mode. Each piece of the frame was designed in order not to deform significantly under load, see Figure 6.3. Figure 6.4 shows the MAND machine used for the 4-point bend test. The MAND machine consists of a temperature control cabinet mounted on the loading frames capable of controlling temperature in the range –5°C to 50°C. It also consists of a 100kN servo-hydraulic actuator and an axially mounted load cell. A Rubicon digital servo control system is used to operate the frame. An external pump is used to supply the hydraulic power to the frame. A digital computer is used for data acquisition.

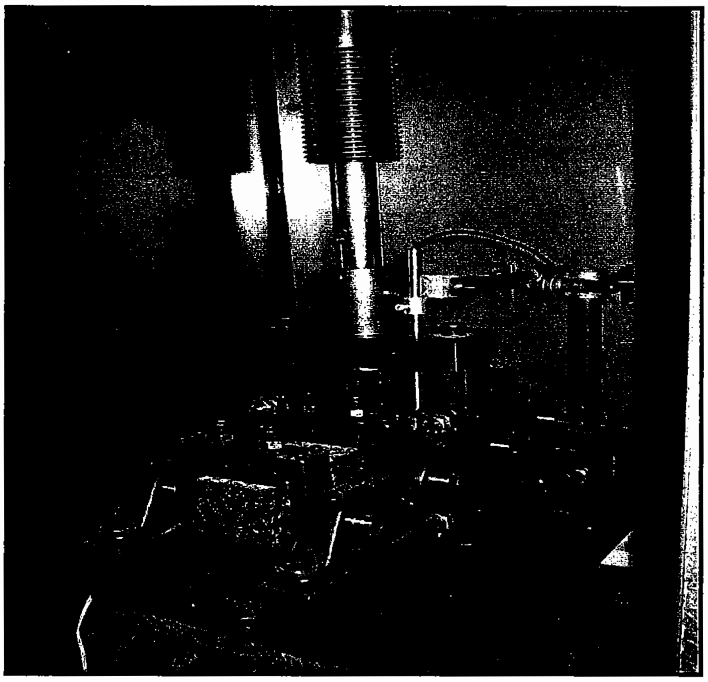


Figure 6.3 Four-point bend frame and specimen ready for testing

The tests were conducted in the strain-controlled mode. During the test, a displacement is applied to the specimen through the hydraulic actuator. A precision linear variable differential transformer (LVDT) connected to the actuator piston continuously monitors the crosshead stroke during a test, and provides a feedback signal for the control system. The control system compares this feedback signal with the input command signal and the difference is amplified and fed to the servo-valve, which adjusts the oil flow to the actuator to reduce the difference between the input and the output signals, such that the actuator responds precisely to the command signal. To connect the 4-point bend frame to the axial MAND testing machine, special pieces were designed and manufactured to fit to the existing actuator and reaction plate [126]. Then the actuator piston was connected to a load cell to record the applied load during a test. To measure the vertical deformation of the beam on the neutral axis, one LVDT was used and positioned at the centre of the neutral axis of the beam resting on an L-shaped piece of metal glued onto the side of the beam as shown in Figure 6.2. Figure 6.5 shows a schematic of the apparatus used for the four-point bend tests.

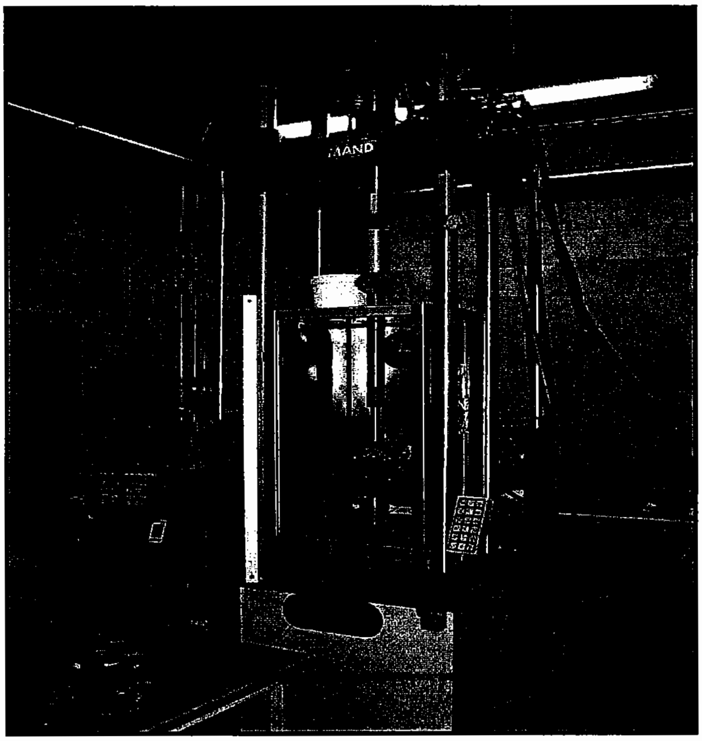


Figure 6.4 MAND testing machine for the 4-point bend test

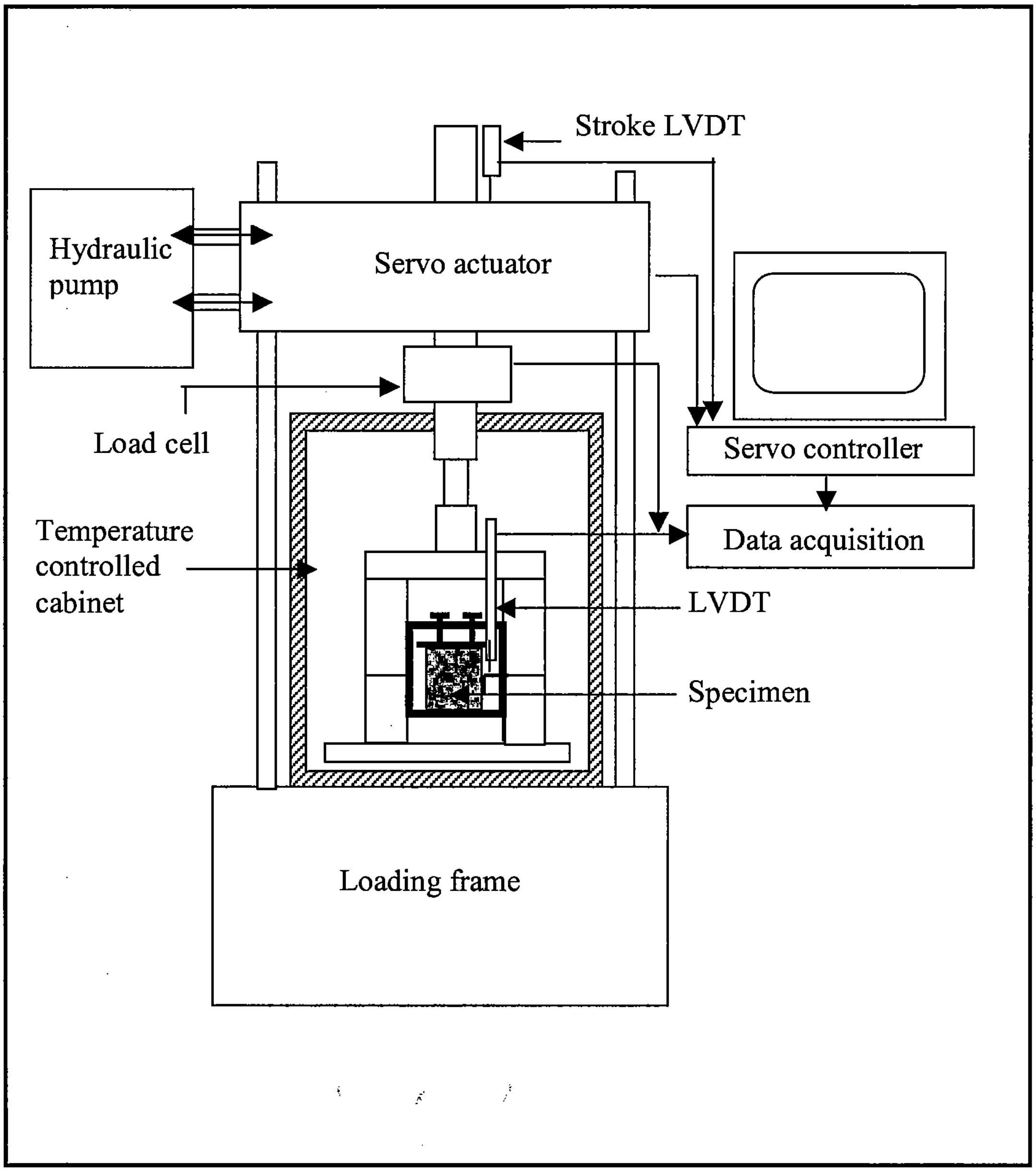


Figure 6.5 Schematic of 4-point bend test apparatus

6.5 Calibration of the equipment

The 4-point bend test frame was attached to the testing machine. Calibration of the LVDT was carried out in an LVDT calibrator. The LVDT used in the tests has a very limited range, but provides high accuracy. All the attached equipment was checked and the necessary alignment was done accordingly. To check the software, the input and output data for the load and displacement of the machine was assessed.

6.6 Test Procedure

Figure 6.5 shows the experimental apparatus used for 4-point bend testing. Before the specimens were installed for testing, they were placed in a temperature-controlled cabinet overnight, to gain the required testing temperature. When ready to begin the test, a specimen was taken and located centrally and straight on the testing frame. The position of the LVDT was adjusted to be at the centre. The top steel plates were placed on the top of the beam at each loading point and fixed using bolts to the outer supports. The specimen was then instrumented as detailed in Figure 6.5. The load was checked to be at the zero position and then the middle supports adjusted; only then were the bolts of the middle supports fixed. Only when all the bolts are fixed correctly and the LVDT adjusted to its position, is the specimen ready for testing. Before starting to test, the temperature-controlled cabinet door was closed and the apparatus was left for a period of not less than 20 minutes to ensure constant temperature conditions within the cabinet.

The tests were conducted in the displacement-controlled mode at strain levels up to 700 microstrain. All the specimens were tested at a temperature of 10°C. In the test, a repeated sinusoidal displacement was applied at a frequency of 10Hz.

The test machine enables measurement of the mixture stiffness with very low deformation applied. Once the initial stiffness had been measured, the fatigue test was carried out on the same beam but changing the input data as required. During each test, the LVDT measurements and the axial load were logged by a digital computer. The machine was fully controlled by the computer and commanded to stop when the load reached 30% of its initial value. During each test, a file was created on the hard disc of the computer, which could be copied and transferred for data analysis. To make sure that the beam remained in the right position and straight, the bolts which fix the beam needed to be checked regularly and tightened if necessary. This is because an asphalt mixture is viscous and will deform permanently under load.

The strain was determined by measuring the deformation on the specimen using the LVDT. The typical load and deformation readings obtained are shown in Figure 6.6.

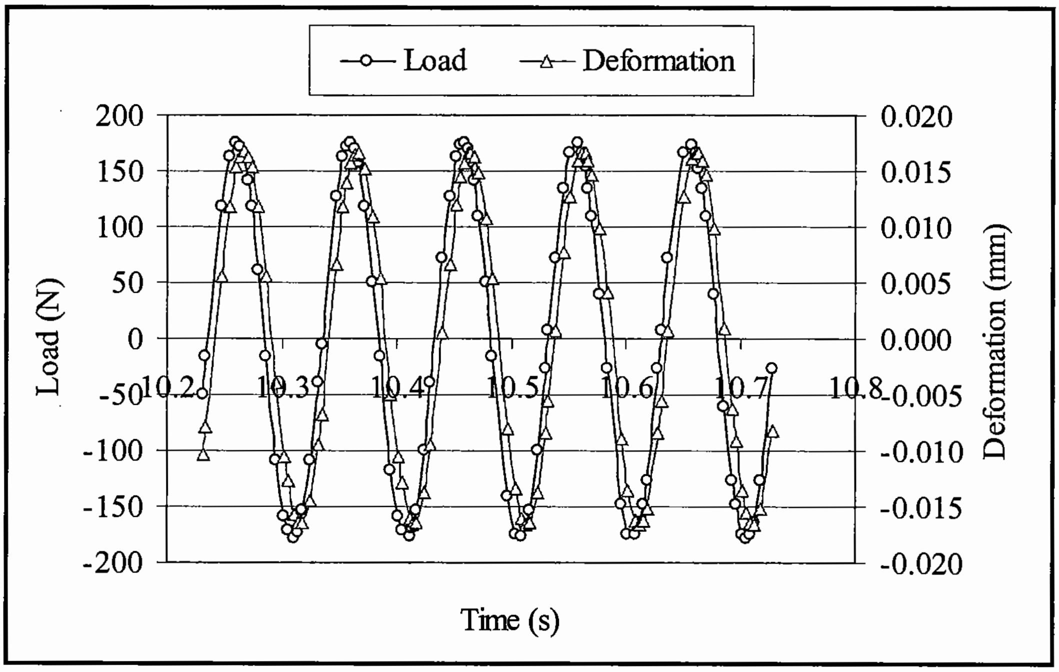


Figure 6.6 Load and deformation response for limestone mixture tested at 0.0166mm deformation, 10°C and 10Hz

6.6.1 Stress, strain and stiffness calculations

Two symmetrically positioned loads were applied to the specimen, subjecting it to 4-point bending. The test schematic, loading and geometry are shown in Figures 6.7 and 6.8.

The data acquisition system of the device recorded the load and deformation of the specimen. Then the tensile strains and stresses were calculated using the following equations.

For the stress calculation:

$$\sigma = \frac{P \times L}{b \times h^2} \qquad (6.3)$$

Where

σ = maximum tensile stress.

P = load applied by the actuator on the specimen.

L = distance between outer supports.

b = average width of the specimen cross-section.

h = average height of the specimen cross-section.

For the strain calculation the following equation [126] is used which considers both bending and shear deformation:

$$\varepsilon = \frac{108\,\delta\,h}{23\,L^2 + 36\,h^2\,(1+\nu)} \tag{6.4}$$

Where

ε = maximum tensile strain.

ν = Poisson's ratio

δ = vertical deformation of the neutral axis in the middle of the specimen.

Flexural stiffness modulus (S) was calculated using the equation:

$$S = \frac{\sigma}{\epsilon} \tag{6.5}$$

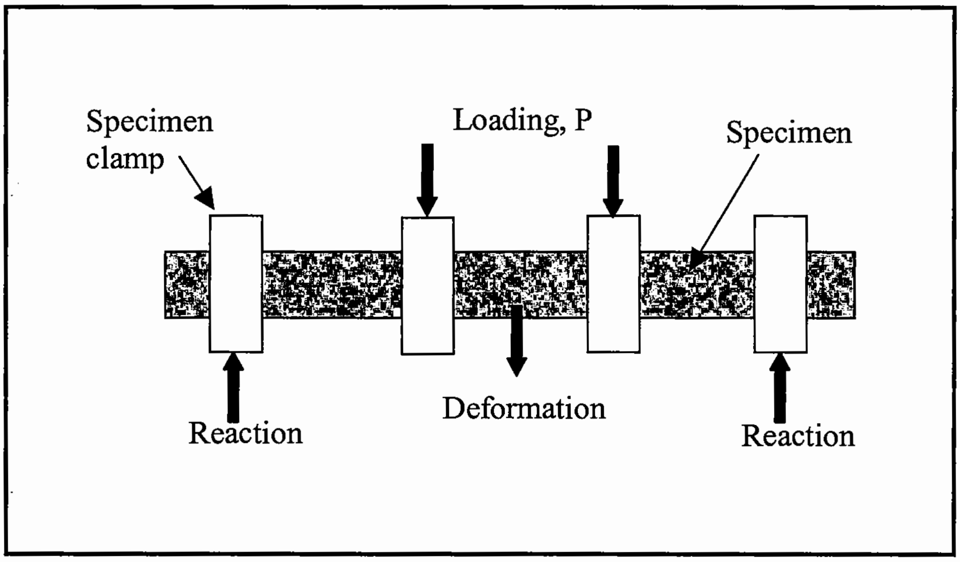


Figure 6.7 Schematic diagram of the 4-point bend fatigue test

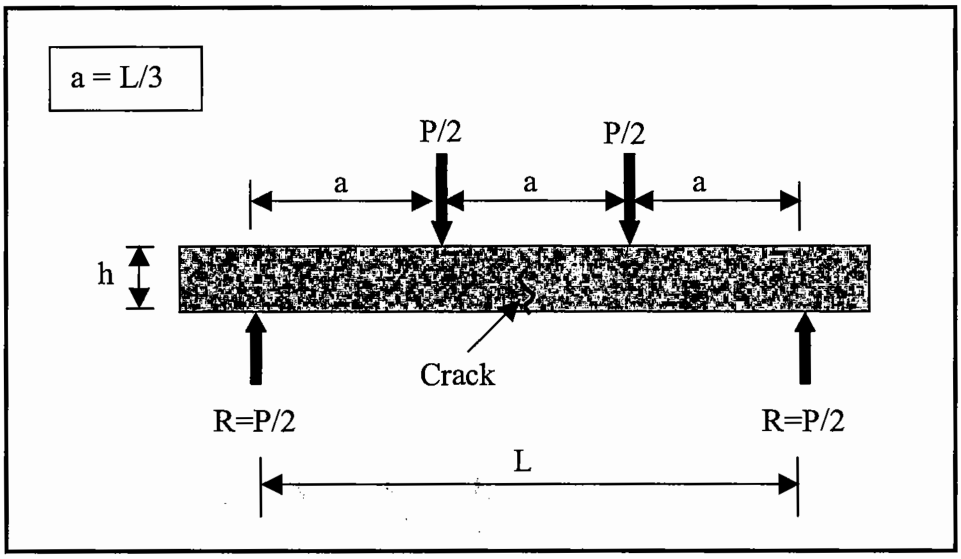


Figure 6.8 Loading and geometry of the specimen in the 4-point bend test

6.7 Mechanical Properties of the Mixtures

Asphalt mixtures in flexible pavements are used as surface or base layers, so that they must withstand the effect of air and water, resist permanent deformation and resist fatigue cracking caused by traffic loading and the environment.

6.7.1 Stiffness Modulus

Generally, the stiffness is the ratio of a stress and a corresponding strain. In pavement engineering, the stiffness modulus is a ratio of stress to strain under axial loading conditions. Stiffness modulus is a function of load, deformation, time, specimen dimensions, temperature and Poisson's ratio. Therefore, different stiffness values are obtained at different loading times and/or temperatures.

The stiffness modulus of bituminous mixture is determined in the laboratory by several methods. In this study, two methods were used; the 4-point bend test and the Indirect Tensile Stiffness Modulus (ITSM) test using the NAT machine, both of which are non-destructive test methods.

6.7.1.1 Measurement of stiffness from 4-point bend testing

The 4-point bend test is a strain-controlled test. To measure the stiffness of the specimens without damaging them requires using very low strain. The X-Y plot program supplied by 'Rubicon Company' is for low strain application. 50 microstrain was used and applied to the beams, which generated a 0.0166 mm deformation in the middle of the beam. The corresponding stress was determined using equation (6.3). From the measured value of the strain and stress, the stiffness was calculated using equation (6.5). All the specimens were tested at a temperature of +10°C and a frequency of 10Hz.

6.7.1.2 Measurement of ITSM

ITSM tests were carried out on specimens using the Nottingham Asphalt Tester (NAT). The test was carried out according to BS DD213: 1993. The standard target parameters throughout testing were as follows:

Test temperature	= 10°C
Rise time	= 124 ± 4 ms
Horizontal deformation	= 5 ± 2 μm
Poisson's ratio	= 0.25

The specimens were conditioned at the testing temperature of 10°C for at least 2 hours before testing.

6.7.2 Bituminous Mixture Fatigue

Fatigue is a common form of failure in an asphalt pavement resulting in the formation of cracks from repeated traffic loading. It is important to have a measure of fatigue characteristics of specific mixtures over a range of traffic and environmental conditions so that fatigue considerations can be incorporated into the process of designing asphalt pavements.

The fatigue characteristics of asphalt mixtures are usually expressed by the relationship between the initial stress or strain and the number of cycles to failure. The fatigue properties of the mixture can be characterised by the slope and the relative level of the stress or strain versus the number of cycles to failure in log-log space. In addition to the 4-point bend test, discussed above, the Indirect Tensile Fatigue Test (ITFT) using the NAT machine was also used for comparison purposes. It is a stress-controlled test, therefore different ranges of stress levels were used.

The result is expressed as a relationship between the initial maximum tensile horizontal strain and the number of cycles to failure. The maximum tensile strain is calculated from the equation:

$$\epsilon_{xmax} = \frac{\sigma_{xmax} \times (1 - 3\nu)}{S_m} \text{ x} 1000 \qquad (6.6)$$

Where

ϵ_{xmax} = maximum tensile strain (microstrain)

σ_{xmax} = maximum tensile stress at the centre of the specimen (kPa)

ν = Poisson's ratio (0.25)

S_m = indirect tensile stiffness modulus (MPa)

The ITFT test was performed to measure the resistance to fatigue failure of the bituminous mixtures. The NAT apparatus was used for the test. The specimens were conditioned at the testing temperature of 10°C overnight. Different stress levels were selected and used to perform the test. Pulse loads were applied to the specimen repeatedly until the specimen failed by complete cracking. LVDTs were used to measure the deformation along the diameter. Each specimen was tested at a different target stress level. The maximum tensile horizontal strain at the centre of the specimen was then calculated using Equation (6.6). The results were plotted as the maximum tensile strain (ϵ_{xmax}) against the number of cycles to failure using logarithmic scales and applying linear regression analysis.

6.8 Results and Discussion

6.8.1 4-point Bend Testing

To study the effect of filler content on the bituminous mixture, different mixtures containing limestone filler contents (15, 35 and 65% by mass) were tested in the 4-point bend machine.

The stiffness modulus of the mixtures indicates that the mixture containing 65% limestone filler had high stiffness (at 10°C), between 13000 and 15000 MPa, whereas for the other mixtures it ranged between 11000 to 13000MPa. These results reflect the effect of filler concentration on the stiffness modulus of the mixture.

A 50% stiffness reduction from the initial stiffness was used to identify the failure point of the mixture. The fatigue data obtained for the various asphalt mixtures was then analysed using the approach of plotting the stress or strain against number of cycles to failure in log-log space.

Figure 6.9 presents the results obtained plotting initial stress against number of cycles to failure. The slopes of the fatigue lines of the three mixtures are similar slopes except for the mixture containing 15% limestone filler, which has a slightly steeper line. The mixture containing 65% limestone filler gave a higher fatigue life (plotted against stress), whereas the 15% filler mixture gave a lower fatigue life.

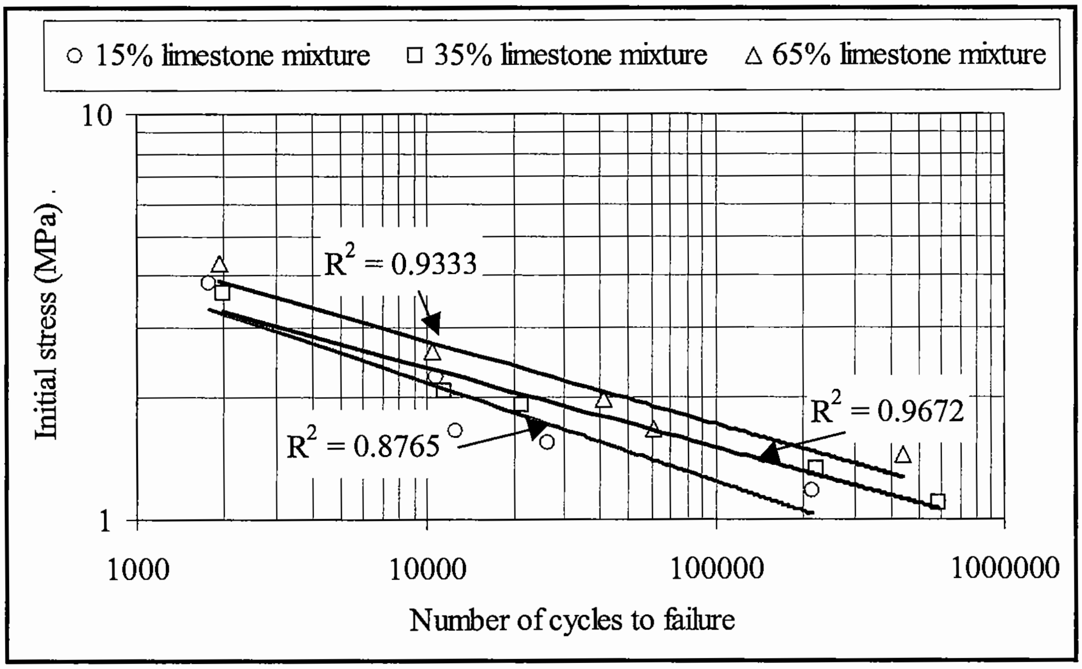


Figure 6.9 Initial stress against number of cycles to failure for limestone mixture tested in the 4-point bend test

The same data is shown plotted against strain in Figure 6.10. Similar fatigue lines were obtained, that for the mixture containing 15% limestone filler, again being slightly different. Mixtures containing 35% and 65% limestone have almost the same fatigue life plotted against strain.

To study the effect of filler types on bituminous mixture fatigue, the types of filler tested were: limestone, gritstone and cement filler. The mixtures were tested under the same conditions as the above, but with only 35% by mass filler concentration used.

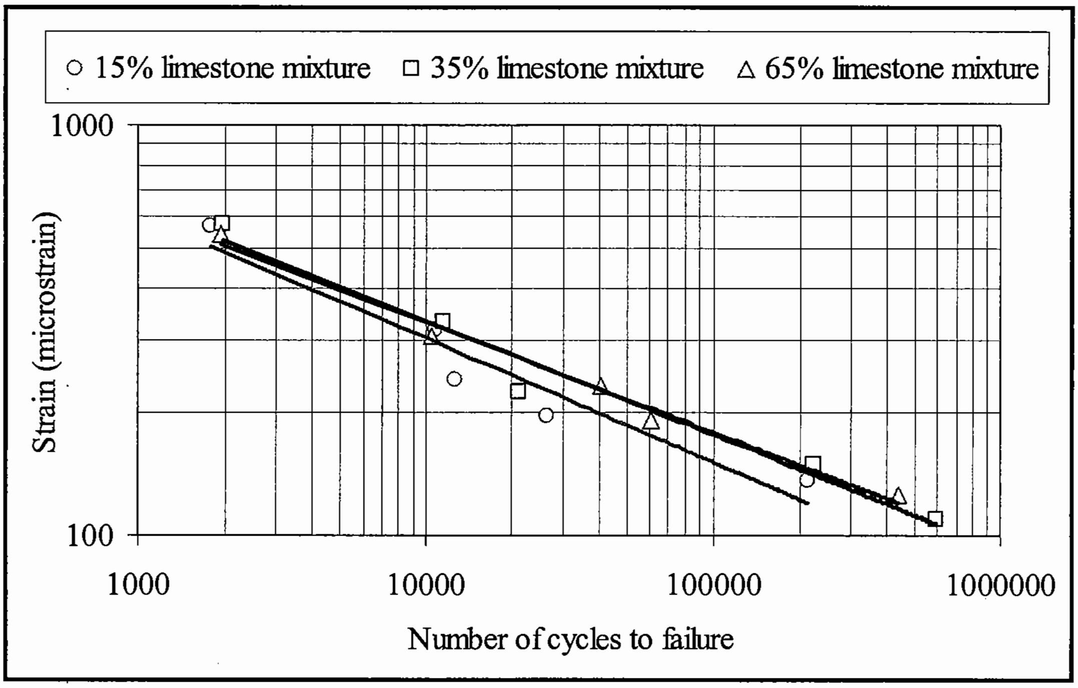


Figure 6.10 Strain against number of cycles to failure for limestone mixtures tested in the 4-point bend test

The results of the tests indicated that fatigue lines of the three mixtures have similar slope, as shown in Figure 6.11, the mixture with gritstone filler being slightly different. However, the three mixtures generated approximately a single fatigue line, which indicates that the filler type has very little effect on the fatigue life of a bituminous mixture. This means that filler content in the mixture plays the more major role in bituminous mixture fatigue rather than filler type.

Figure 6.12 presents the results of the 4-point bend test on the mixtures on the basis of strain against number of cycles to failure. Again the results showed no effect due to filler type on the bituminous mixture fatigue life. Referring to Figures 6.10 and 6.12 the clear impression is that mixture fatigue is primarily strain dependent.

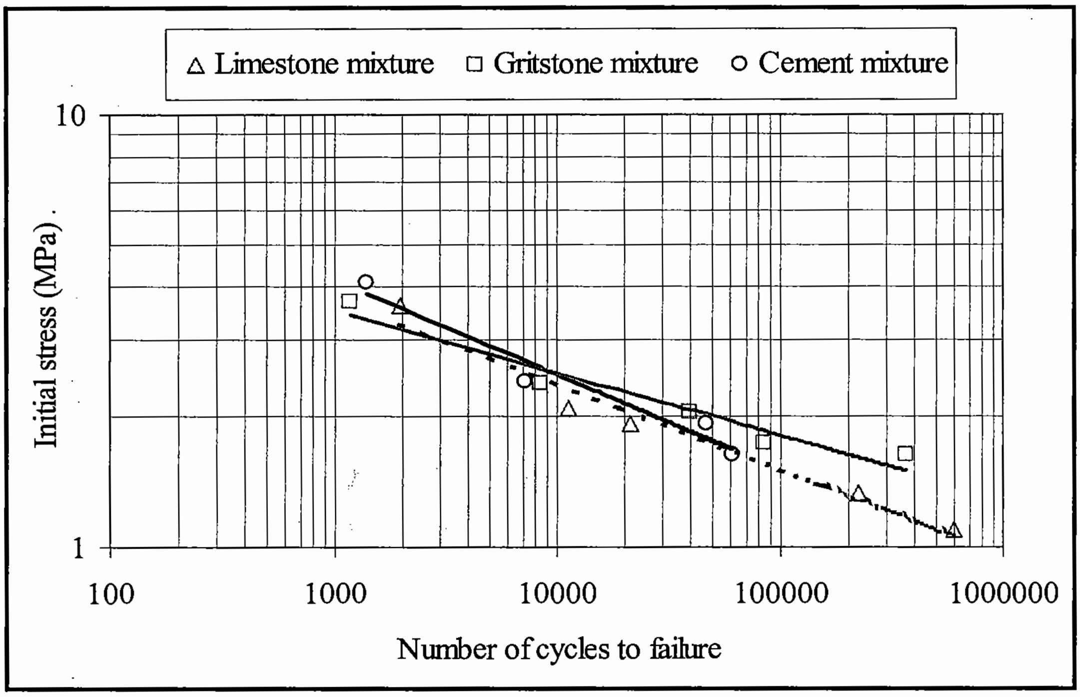


Figure 6.11 Initial stress against number of cycles to failure for mixtures tested in the 4-point bend test

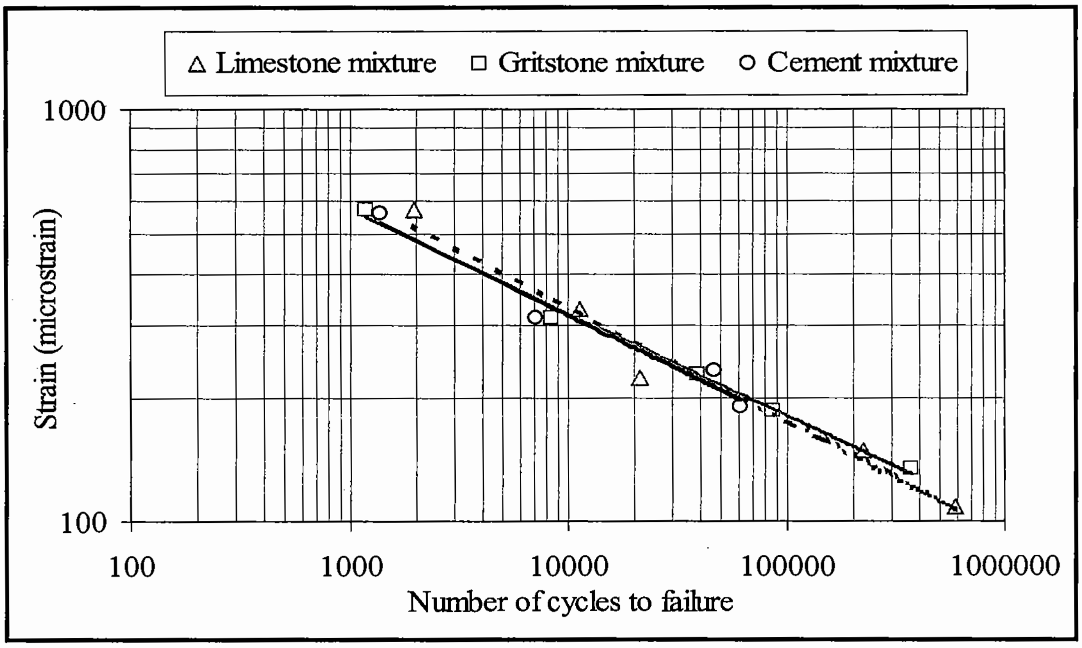


Figure 6.12 Strain against number of cycles to failure for mixtures tested in the 4-point bend test

Table 6.2 presents the stress and strain after 100 cycles and one million cycles of testing using the fatigue equations. The mixture containing 65% limestone filler has higher resistance to fatigue failure compared to the other limestone mixtures, by almost all measures. The lowest resistance to fatigue was exhibited by the mixture containing 15% limestone filler.

Table 6.2 Predicted stresses and strains for mixtures containing limestone filler

Material	Stress (MPa)		Strain (microstrain)	
	$N_f = 100$	$N_f = 10^6$	$N_f = 100$	$N_f = 10^6$
15% limestone mixture	6.70	0.7081	1203.95	75.336
35% limestone mixture	5.87	0.9574	1210.7	92.52
65% limestone mixture	7.10	1.0731	1137.28	96.35
Gritstone mixture	4.895	1.2958	1015.13	101.887
Cement mixture	6.867	0.90697	1048.65	95.9906

Stiffness reduction was observed on all specimens tested in the 4-point bend fatigue test. Figure 6.13 shows the stiffness results for a mixture containing 35% limestone filler tested at different strain levels. These stiffness curves can be divided into three zones as illustrated on the curve for the specimen containing 65% limestone filler and tested at 300 microstrain as shown in Figure 6.14. Zone one indicates a reduction in the stiffness, which happens early in the test. In zone two, the test is progressing and the stiffness is reducing but at a slower rate. The final zone is the failure zone, where the specimen fails after reaching 50% reduction from its initial stiffness.

Figure 6.14 shows the reduction in stiffness of specimens containing different limestone filler contents at different strain levels. Similar results are shown in Figure 6.15 for mixtures containing different filler types. This reduction in stiffness might be due partially to the heat generated during testing as an accumulation of the dissipated energy per cycle. The same phenomenon has been reported by other researchers [86,107,108].

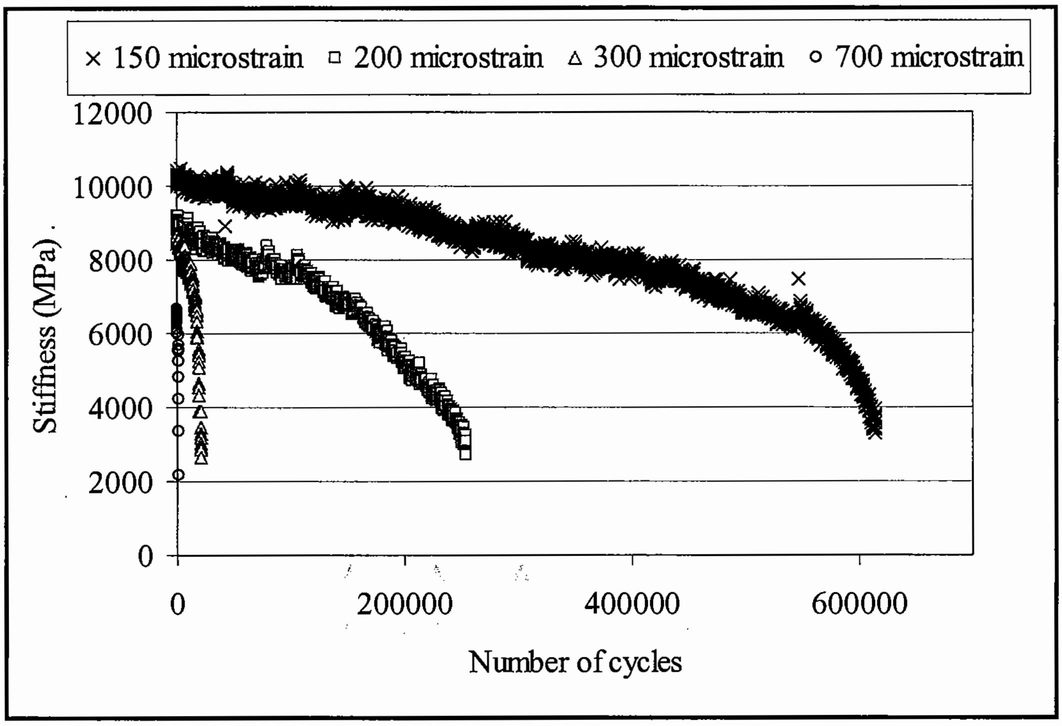


Figure 6.13 Stiffness against number of cycles for a mixture containing 35% limestone filler tested by the 4-point bend test

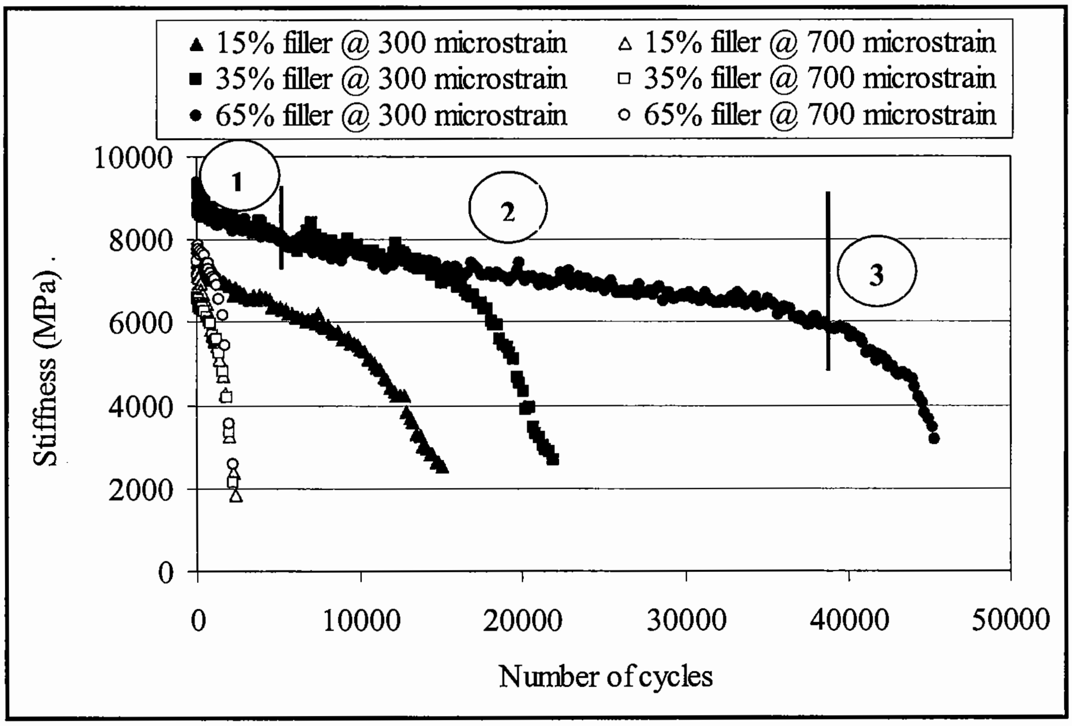


Figure 6.14 Stiffness against number of cycles for mixtures containing different limestone filler contents tested by the 4-point bend test

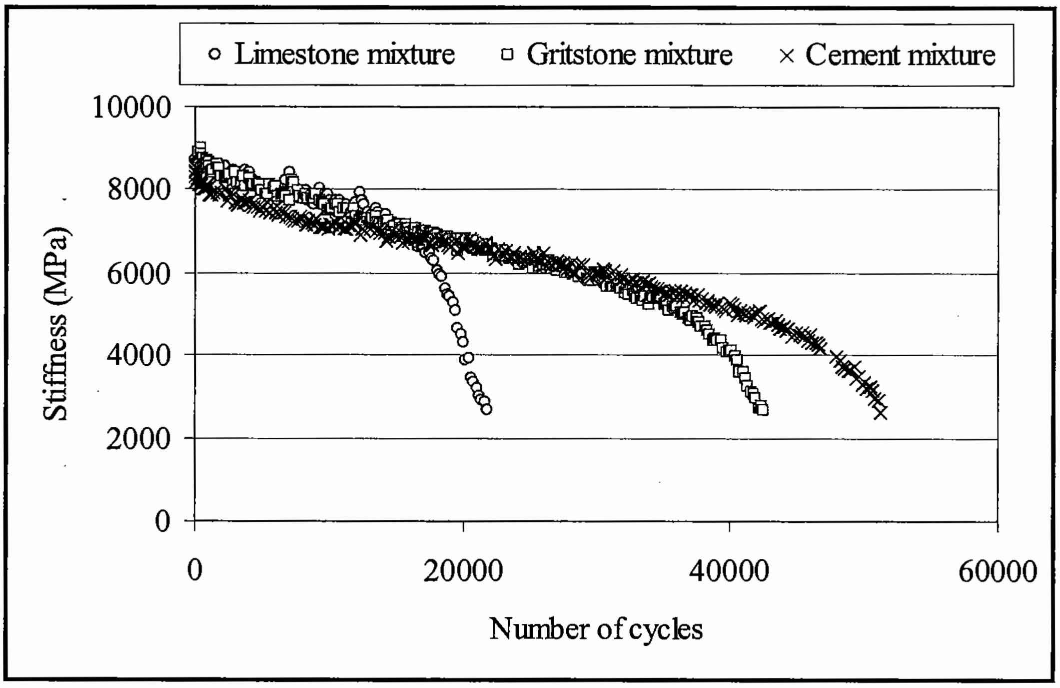


Figure 6.15 Stiffness against number of cycles for mixtures containing different filler types tested by4-point bend test at the 300 microstrain level

6.8.2 Indirect Tensile Fatigue Test

Fatigue of bituminous mixtures was also investigated using the ITFT method. Three mixtures were tested. Each mixture contained 35% filler concentration by mass (limestone, gritstone and cement). The test was carried out using the NAT apparatus after conditioning the specimens at a temperature of +10°C. The tests were carried out in stress-controlled mode. Table 6.3 presents the stress and strain to cause failure at 100 and one million cycles.

Figures 6.16 and 6.17 show the fatigue life of mixtures tested using the 4-point bend test and ITFT method. Different fatigue lines were obtained with slightly different slopes. However, the results indicate that on either a stress or strain basis, the data can fit a single fatigue line with a high regression coefficient (> 0.81). These results support the earlier finding of this research that filler type does not play a major role in the fatigue life of a bituminous mixture, within the limited range of materials tested on this research.

Both test methods can generate single fatigue lines for the mixtures containing different filler types and have similar slopes. The results show that the ITFT gives shorter fatigue lives compared to the 4-point bend test. This difference can be explained by the difference in testing method, since ITFT is a stress-controlled test, while the 4-point bend test is a strain-controlled test. Another reason for the difference is the significant accumulation of permanent deformation during ITFT tests.

Table 6.3 Predicted stresses and strains and for the bituminous mixtures tested in the ITFT test

Material	Stress (MPa)		Strain (microstrain)	
	$N_f = 100$	$N_f = 10^6$	$N_f = 100$	$N_f = 10^6$
Limestone mixture	4447.5	216.63	702.16	26.35337
Gritstone mixture	2875.7	313.27	428.222	46.86715
Cement mixture	3743.4	254.49	372.382	40.79351

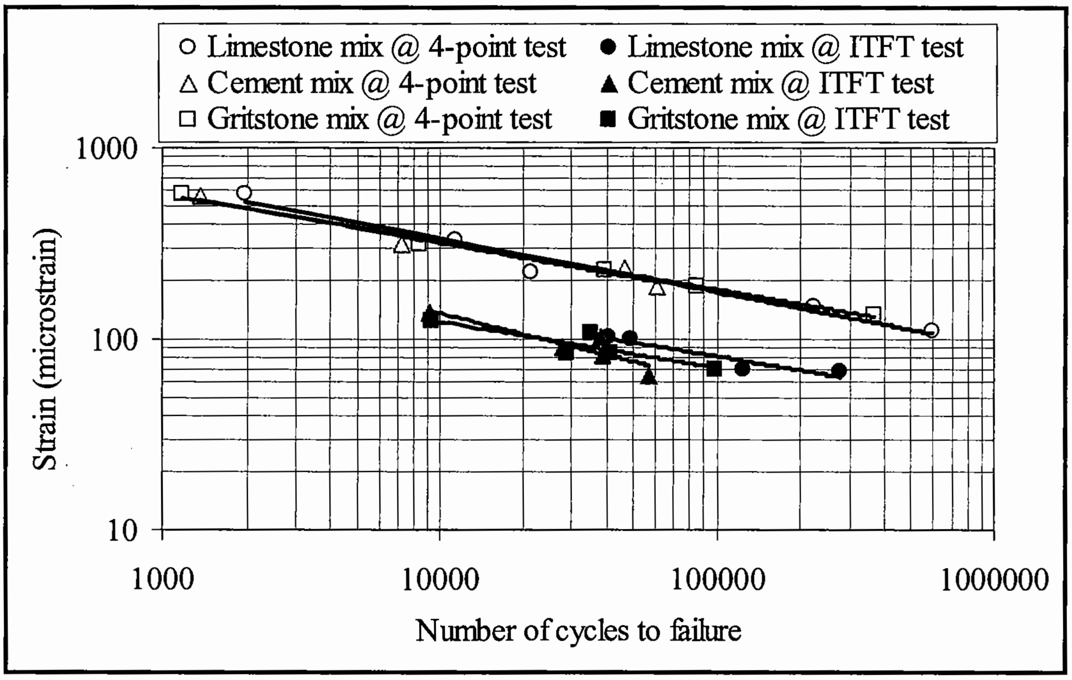


Figure 6.16 Strain against number of cycles to failure for bituminous mixtures tested at +10°C and 10Hz

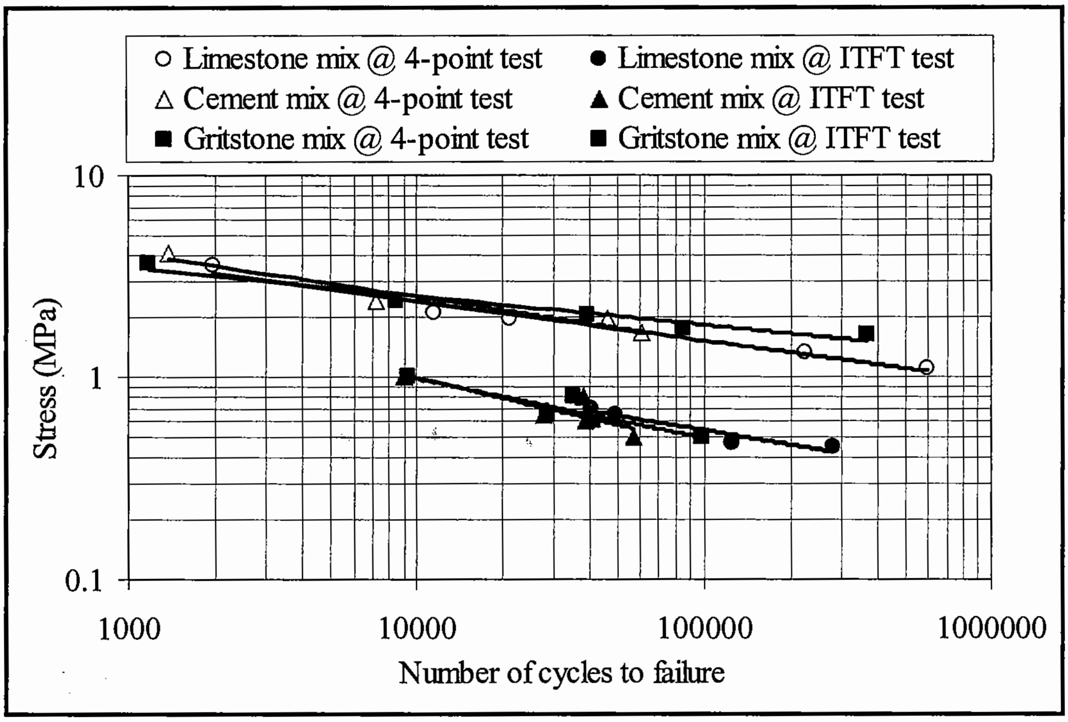


Figure 6.17 Stress against number of cycles to failure for bituminous mixtures tested at +10°C and 10Hz

6.9 Summary

Three mixtures containing different limestone filler contents were tested in the 4-point bend test, and three mixtures containing different filler types were tested in both the 4-point bend test and the ITFT test at a temperature of +10°C.

The key points are:

- The stiffness modulus measured for the mixture containing 65% limestone filler by mass is higher than for the other mixtures, while the 15% filler mixture is least stiff.

- Both stress (and strain) were plotted against number of cycles to failure to characterise the fatigue properties of the mixtures. The results indicate that filler concentration has an effect on the fatigue resistance, whereas the fatigue life of the mixture is not filler type dependent within the range for the materials tested in this research.

- Both of the test methods used can generate a single fatigue line for mixtures containing different filler types, for the range of materials tested in this research. The ITFT method gave shorter fatigue life because of the difference in testing mode.

- Initial stiffness reduction was observed in the 4-point bend test for mixtures containing both different filler types and filler contents. This was explained as due partially to the heat generated during loading cycles.

7

Analysis and Evaluation of Laboratory Results

7.1 General

This chapter presents an analysis of the results of the tests carried out for pure bitumen, bitumen-filler mortar and the bituminous mixtures. The idea is to compare the fatigue properties of bitumen and bitumen-filler mortar and then those of the mortar and the bituminous mixture so that a clear picture of the factors affecting pavement fatigue at every stage from bitumen to mixture can be established.

A bituminous mixture normally contains aggregate and filler, together with sufficient bitumen to reduce the voids between aggregate particles. The filler and bitumen together can be considered to form a mortar that both lubricates and binds the aggregate to form a bituminous mixture. Therefore, the properties of the mixture partly depend on the properties of the aggregate and partly on the amount, viscosity and fracture resistance of the bitumen/filler mortar. Some researchers [128] have shown that the ratio of the viscosity of mortar and bitumen is a function of the volume of the fraction of the filler.

7.2 Direct Tension Test Results

The results obtained from the DTT were used to study the tensile properties of bitumen and bitumen filled with filler. Constant strain rate tests were performed on 50pen bitumen as well as mortars containing different filler types and contents at temperatures ranging from –10°C to +10°C. Typical results of stress plotted against strain for different applied strain rates, and temperatures were shown in Chapter 4. It can be seen that for each applied strain rate, the stress increases with the strain up to a point, beyond which it decreases for many tested specimens. At high strain rates, the material fails by fast fracture before necking can occur. It can be seen that at any given temperature the value of peak stress increases with the applied strain rate until fracture occurs for some specimens.

7.2.1 Effect of Filler Types and Content

Figure 7.1 shows the results of peak mortar stress against peak bitumen stress for mortars containing different filler contents and tested at different temperatures and strain rates. Peak mortar stress increases as the filler concentration increases. This suggests that for the specimens tested, the stress increase can be explained by the solid volume concentration of the filler particles within the mortar, which both increases the area of the fracture surface and increases effective stiffness.

The effect of the filler on bitumen is an increase in the strength of the mortar. The effect of the filler on bitumen doesn't reduce its adhesive power, which can be observed during testing, in that the specimens after failure were still well adhered to the end inserts. The results show that stress at failure and strain at failure are related to the quantity of the mixed-in filler. This raises the importance of proper bitumen-filler mortar in the bituminous mixture, because the mixture in practice should be able to withstand mechanically-induced strain without damage, even at low temperature. Therefore, bitumen-filler mortars with the highest possible strain at failure are desirable. However, the two requirements of high viscosity at high temperature and high strain at failure at low temperature are largely contradictory.

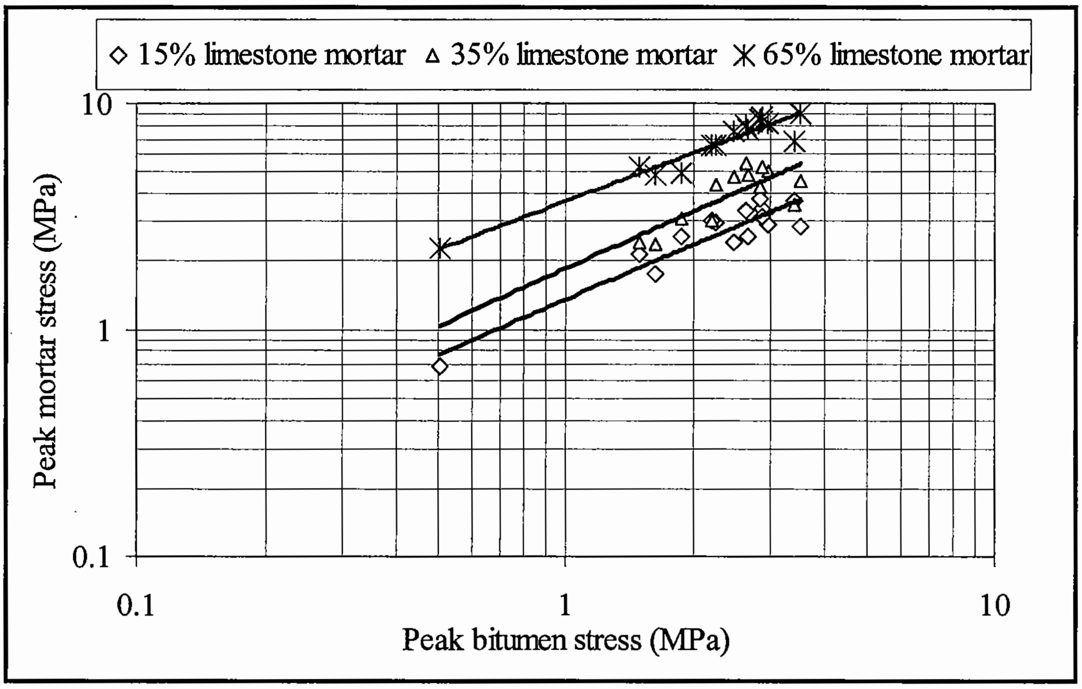


Figure 7.1 Maximum mortar stress against maximum bitumen stress at different temperatures (-10 to +10°C) and strain rates (5.9 to 147.9%/min)

Some researchers [57] have shown that different fillers will stiffen bitumen binder differently. That may be true if the filler shape, surface texture and particle size are considered. The statement is confirmed by the results of the sewage sludge ash when added to bitumen. The stiffening effect of this filler was different from the others, and the main reason for this was assumed to be because of the geometric characteristics of the filler particles and their surface activity. It is noted that some researchers [59] reported that Calcium, Sodium and Chloride are present in the sewage sludge ash as well as Lead, Cadmium, Zinc and Copper. Some or all of these may affect behaviour.

It is assumed that size, shape and texture are the fundamental physical properties of the fillers. The importance of these lies in their effect on the viscosity of the resulting mortar and then its stiffness. The results from the DTT indicate that as the concentration of filler increases, the tensile viscosity of the mortar increases, and that the internal strength of the resulting mortar also increases as shown in Figure 7.2. The tensile viscosity is calculated as peak stress divided by strain rate, the results shown exclude those with brittle fracture behaviour.

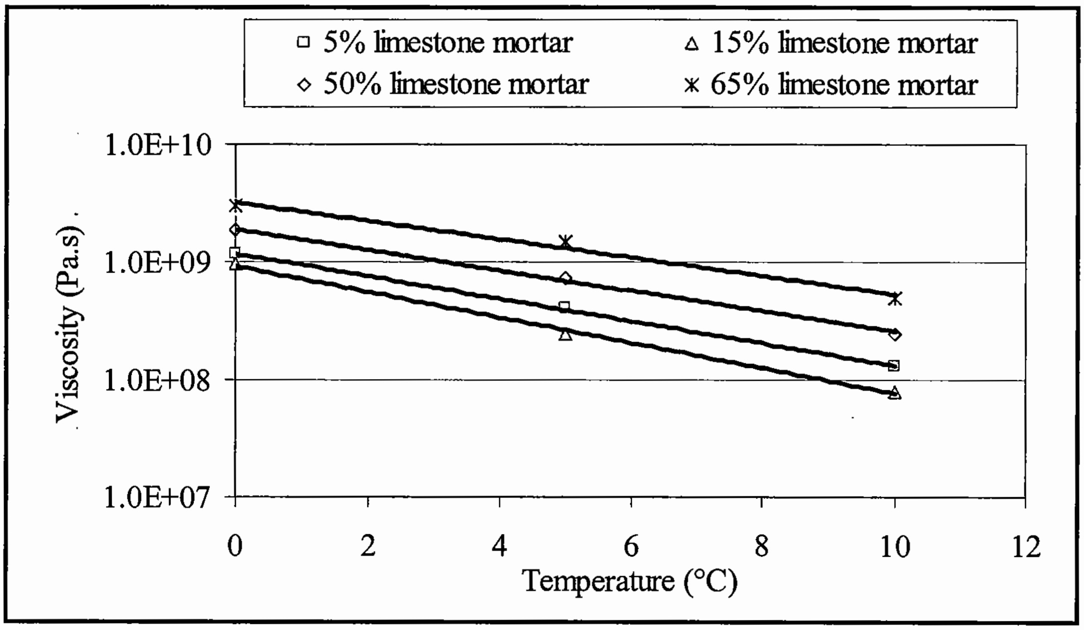


Figure 7.2 Effect of filler content on bitumen viscosity at strain rate 5.9%/min

From these results obtained from the DTT, it is assumed that the stiffening effect of the filler on the bitumen results relatively from the physico-chemical interaction between the bitumen and the surface of the filler mineral particles, which is an issue in need of further investigation. Some researchers [129] state that the physico-chemical stiffening effect can cause apparent changes in the viscosity of the bitumen and therefore produce stiffening of mortars.

From the measurements conducted, it is clear that the fillers have an effect on the stiffness properties. This effect on stiffness has been understood for a considerable period of time. Einstein in 1906 [see ref. 97 pp199], made a hydrodynamic calculation relating to the disturbance of the flow lines when identical, non-interacting, rigid spherical particles are dispersed in a liquid medium, and arrived at the expression:

$$\eta = \eta_0(1+2.5V_f) \qquad (7.1)$$

where

η = viscosity of solution or dispersion

η_0 = viscosity of pure solvent (bitumen in this case)

V_f = volume fraction of filler

The above expression can be applied quite adequately to the case of spherical filler particles dispersed in bitumen. The effect of such particles on the viscosity of the dispersion depends, therefore, only on the total volume they occupy and is independent of their size.

At a constant strain rate, $\dot{\epsilon}$, this relation leads to a flow stress appropriate to low volume fraction expressed as:

$$\sigma = \sigma_0(1+2.5V_f) \qquad (7.2)$$

where

σ = maximum mortar stress

σ_0 = maximum stress of pure bitumen at strain rate $\dot{\epsilon}$,

Figure 7.3 shows the flow stress plotted against volume fraction at different temperatures and a strain rate of 5.9 %/min using equation 7.2. At low volume fractions, it appears to give a reasonable account of the stiffening due to filler inclusion. But it underestimates the flow stress at high volume fraction, when the filler particles increase the bitumen viscosity much more significantly.

Some modifications have been introduced to Einstein's formula, in order to estimate the peak stress in the mortar at high volume fraction. Although different models have been used to estimate the fracture stress and strain with respect to volume fraction [130,131], none of them estimated the stresses well, for the materials tested in the DTT in this work. Therefore a new equation is proposed for the peak stress ratio of filled bitumen as follows:

$$\sigma = \sigma_0(1+aV_f^{\,b}) \qquad (7.3)$$

where

a = material constant (in this case =3)

b = material constant, temperature dependent (0.5-0.85)

The peak mortar stresses are plotted against volume fraction at temperature 0°C and strain rate 5.9%/min in Figure 7.3. The above equation seems to give a better estimate of the peak mortar stress than Einstein's formula.

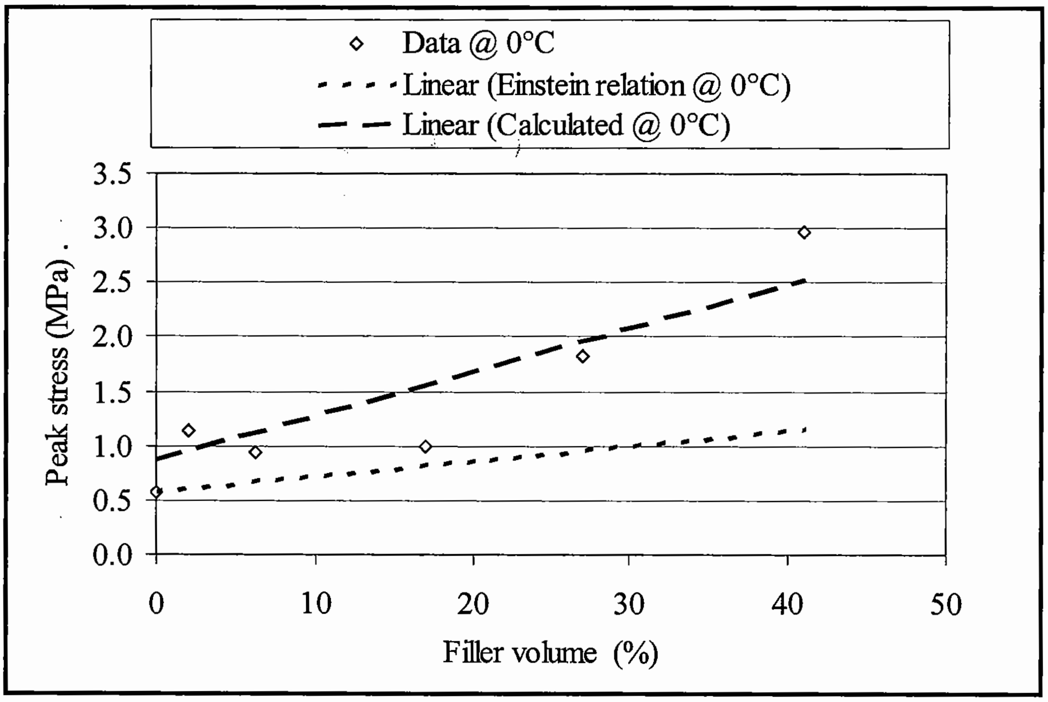


Figure 7.3 Peak stress against filler volume for mortar tested at strain rate 5.9%/min

7.2.2 Power Law Relationship

Other results obtained from the DTT are presented in Figure 7.4. Within the stress, strain rate and temperature range of these tests, it can be seen that the relative viscous properties of the bitumen and bitumen-filler mortar at a particular temperature can be represented in terms of stress and strain rate by the power law equation:

$$\frac{\dot{\epsilon}}{\dot{\epsilon}_0} = \left(\frac{\sigma}{\sigma_0}\right)^n \tag{7.4}$$

where

$\dot{\epsilon}$ = strain rate,

σ = maximum stress

σ_0 = reference stress

$\dot{\epsilon}_0$ = reference strain rate

n = exponent constant

The value of the exponent (n) obtained from the curve fitting was n ≈ 2.0. However the value of (n) is temperature dependent. Similar results have been reported by Cheung [37] for pure bitumen tested at different temperatures.

The results indicate that the maximum stress is increased as the strain rate increases for a particular temperature. A power law appears to fit both pure bitumen and bitumen/filler mortars. Some researchers [37] observed linear behaviour for stress levels at or below 2kPa, which they modelled successfully using theories of linear viscoelasticity. Some of them stated that non-linearity could become more marked at higher stress levels as confirmed by the power law results' presented in Figures 7.4 and 7.5. However for cases where the specimen fractured the power law was not found to be applicable.

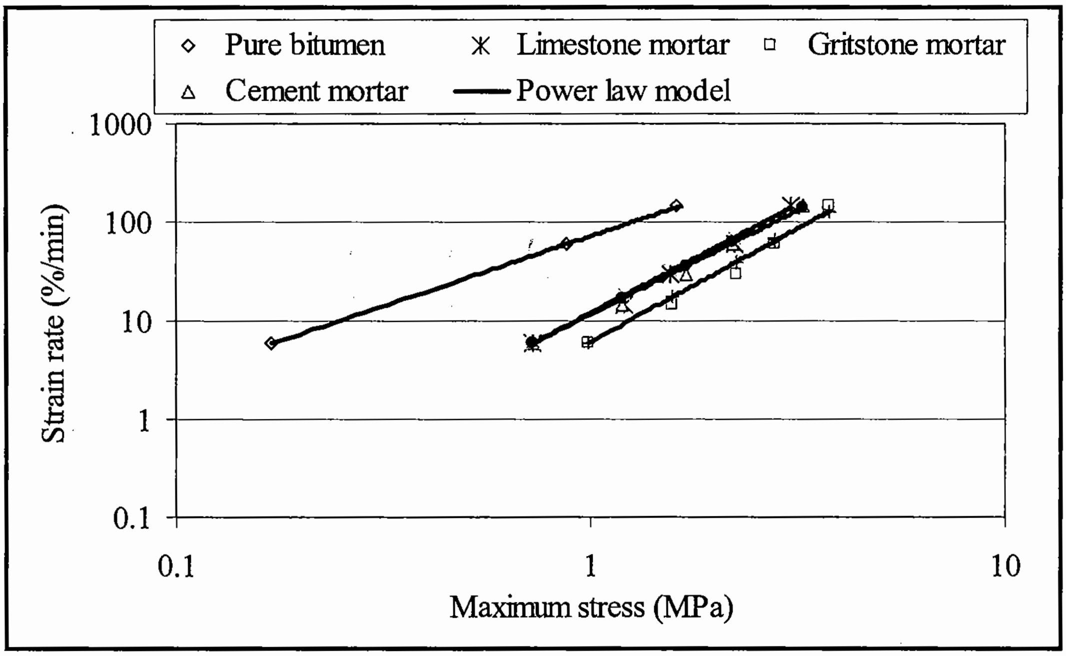


Figure 7.4 Strain rate against stress for bitumen and mortars tested at +5°C

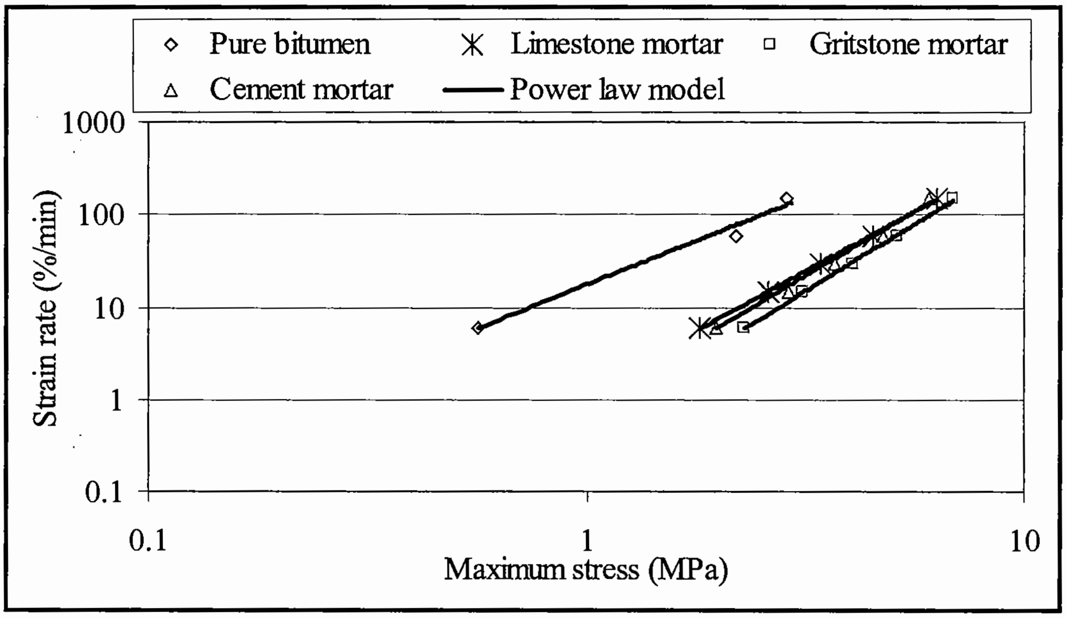


Figure 7.5 Strain rate against stress for bitumen and mortars tested at 0°C

7.2.3 Effect of Filler Particle Size

The results from the DTT, for mortars containing filler particles less than 75 μm and greater than 75 μm, indicate that mortar with filler particles less than 75 μm exhibit slightly higher stress at failure. The reason may be the increase in interfacial area per unit volume of filler as particle size decreases. At a given filler concentration, smaller particles with high surface area influence a relatively high proportion of the binder, thus the strength of the mortar is increased. The chances of transmitting stress to particles are higher for smaller particle filler under the condition of good adhesion, assuming that stress will preferentially be attracted to particles and that the line of force takes the shortest possible distance between the particles.

To determine the effect of a filler particle on the strength of a mortar, the stress distribution around the particle needs to be known. But due to the difference of stiffness modulus and Poisson's ratio between the filler particle and the bitumen, stress concentration occurs at the interface. Researchers [73,132] have shown that the stress concentrations near the particle are independent of particle size under conditions of good adhesion. The larger the particles, the larger the stress concentration area as shown in Figure 7.6.

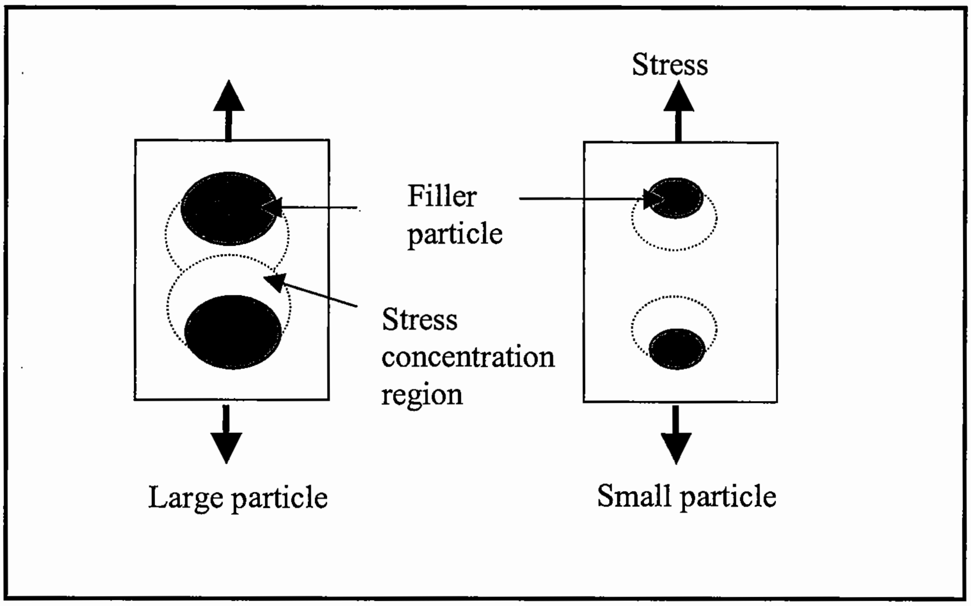


Figure 7.6 Stress concentration around filler particles

7.2.4 Construction of Master Curves

Research has shown that in the case of polymers, failure can be expressed as a universal function of reduced time or strain rate (master curves), using a shift factor, a(T), obtained from rheological data. SHRP [12] has shown through the use of dynamic mechanical analysis data that the temperature dependence of bitumen can be similarly expressed. It is apparent from the DTT failure data that the shift factor determined from rheological tests may also be used to generate a failure master curve.

The results of the DTT for pure bitumen and mortars show the dependence of the stress and strain at failure on strain rate. All the mortars show similar trends as discussed in chapter 4.

Bitumen and mortars were tested in the DTT using a limited strain rate range between 5.9%/min and 147.9%/min. For this reason the rheological properties of the materials tested cannot be presented over a wide range of strain rates. Therefore, a master curves is a logical method to extend the data to cover a wider range of strain rates, not covered under the DTT testing. In order to facilitate the analysis of the rheological data, the data have been combined into a single master curve using time-temperature superposition.

To construct the master curve, a reference temperature was selected first and then the rheological data at all other temperatures was shifted horizontally with respect to strain rate to produce a single smooth curve. The horizontal shifting is done by using the shift factor, a(T), which varies for each test temperature. The values of log a(T) proposed by SHRP [12] can be found from two equations relating the temperature dependence to the defining temperature.

For $T > T_d$ equation 7.5, the Williams-Landers-Ferry (WLF) equation will be used:

$$\log a(T) = \frac{c_1 \times (T - T_d)}{c_2 + T - T_d} \tag{7.5}$$

For temperature $T \leq T_d$ equation 7.6 , the Arrhenius equationwill be used:

$$\log a(T) = 13000 \times \left(\frac{1}{T} - \frac{1}{T_d}\right) \tag{7.6}$$

where,

a(T) = shift factor at temperature ,T, relative to defining temperature, T_d

T = selected temperature

T_d = defining temperature

c_1,c_2 = constants

As presented in chapter 4, different graphs were plotted to present the stress or strain against strain rate at different temperatures; these graphs have then been used to construct the master curves. The shift factor equations were used to determine the shift factor required for horizontal shifting of the data with respect to strain rate. The temperature 0°C (273.15 °K) was selected as the defining temperature for all tested materials. Figure 7.7 shows the shift factors for the data obtained for the materials tested. Although the time dependency of the rheological response is important, the temperature dependency of the response is also important. The shift factors are relatively unchanged by the addition of the filler particles. Thus it can be concluded that temperature dependency of the bitumen and the mortar are essentially the same. Similar findings have been reported by Anderson et al. [133].

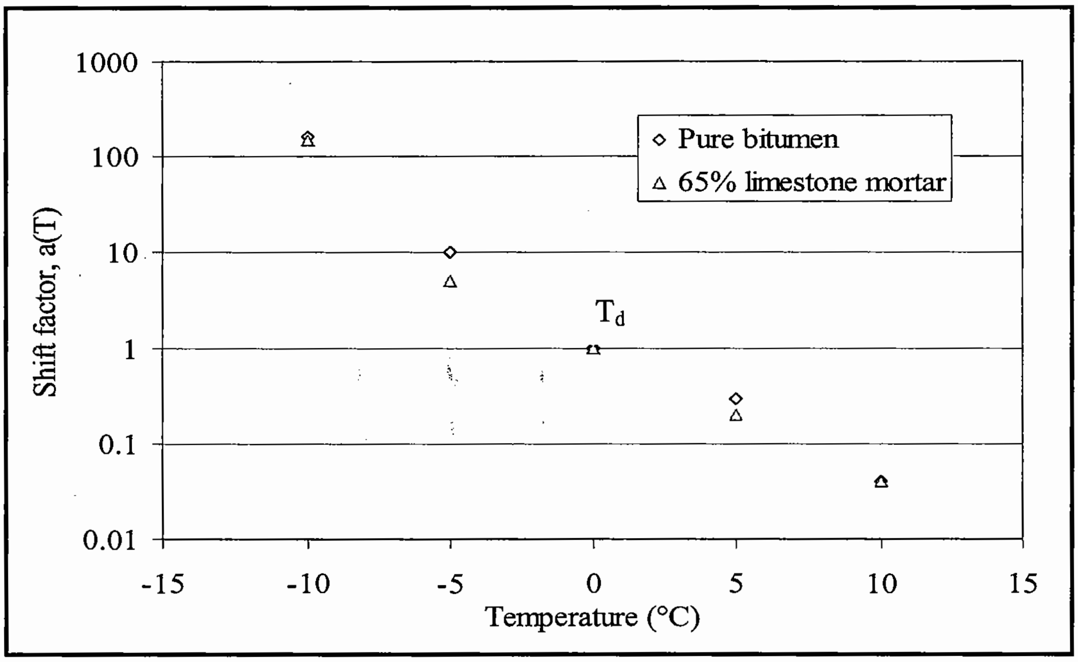


Figure 7.7 Shift factor against temperature for bitumen tested by DTT

Figure 7.8 shows the master curves for bitumen and mortar containing different filler contents, resulting from testing the materials at different strain rates and temperatures using the DTT. The master curves define the strain rate dependency of the stress at failure (or peak stress), and the shift function defines the temperature dependency. The effect of inclusion of the filler particles on the bitumen stress is clear from these master curves; as the filler content increased, so the strength of the material increased.

Figure 7.9 shows the master curve for strain at failure with respect to reduced strain rate for the bitumen and mortar containing different filler contents. In this case the low temperature rheology is changed little with the addition of fillers. The addition of fillers tends to increase the bitumen strength with a much smaller change in the strain at failure. The net result is a significant increase in failure stress as shown on Figure 7.8.

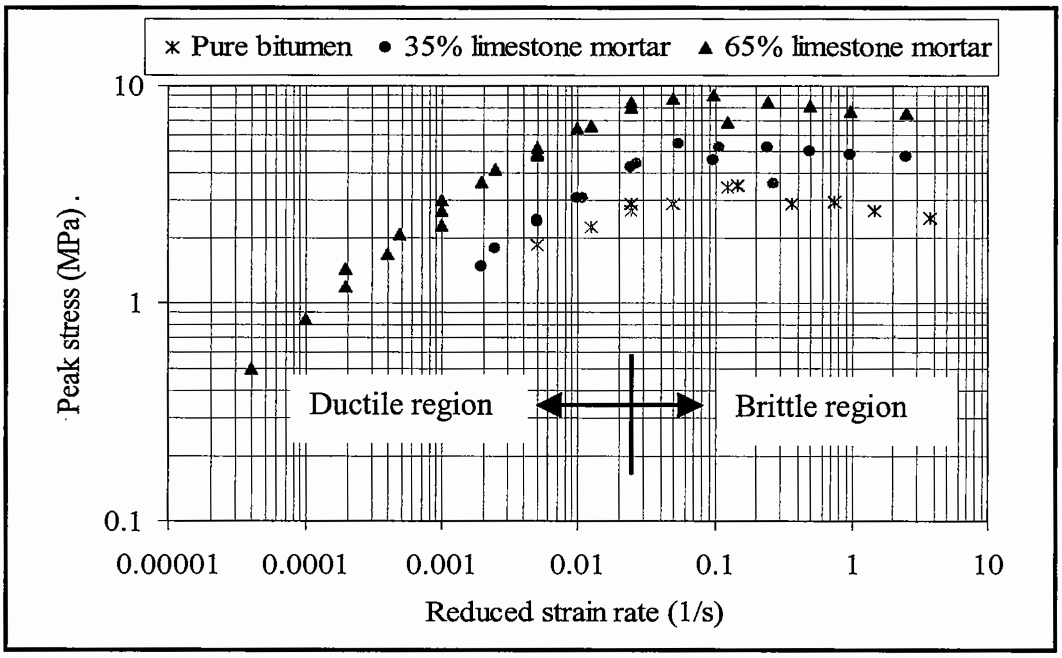


Figure 7.8 Peak stress Vs reduced strain rate for bitumen and mortar (Master curve)

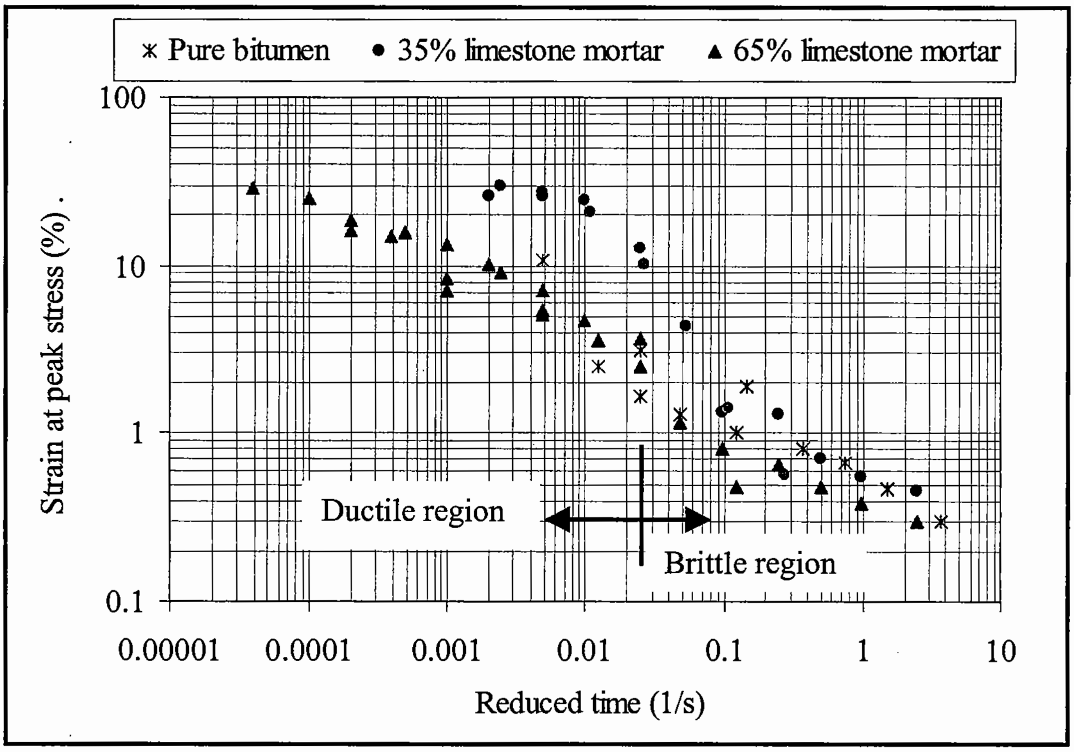


Figure 7.9 Strain Vs reduced strain rate for bitumen and mortar (Master curve)

It can be observed that bitumen and mortars show a viscous behaviour at very low strain rate where the behaviour is Newtonian. At high strain rate, the behaviour can reach an elastic behaviour with a failure strain that is approximately the same for both bitumen and mortars

7.3 DTT and Fatigue Data Relationship

Attempts have been made to develop a relationship between the DTT results and the binder and mortar fatigue results. The relationship is based on the observations of the master curves generated from the DTT results from the bitumen and bitumen-filler mortar as shown in Figure 7.8. Although the strain rate range used for the testing is not capable of making the specimens fail in brittle fracture at temperatures of 10°C, the graph shows that at higher strain rate a specimen would fail in brittle mode. This means that under the special circumstances of high strain rate, bitumen at 10°C can be made to fail in a brittle matter.

All the bitumen and mortar specimens tested in fatigue were tested at a temperature of 10°C and a frequency 10Hz in a stress-controlled mode test (tension-compression). Considering a single loading cycle in a binder fatigue test, where the specimen is stretched by the applied stress and deformation generated, it is possible to approximately determine the strain rate, which will be very high compared with the strain rates used in the DTT. It is practically impossible to apply this kind of strain rate in the DTT machine. The estimated strain rates lie on the brittle fracture regime of the master curve (every time).

The calculated strain rate from the fatigue test can then be used to estimate the stress at failure (under single monotonic loading) from the master curve, which can be compared to the stress theoretically needed to fail a fatigue specimen in the first cycle of load, extrapolated from the fatigue characteristics found. Figure 7.10 shows the relationship between the stress for single cycle failure, derived from the fatigue testing, to that obtained from the DTT for the bitumen and mortars containing different filler contents. The graph shows good agreement between the failure stresses from the fatigue test and DTT.

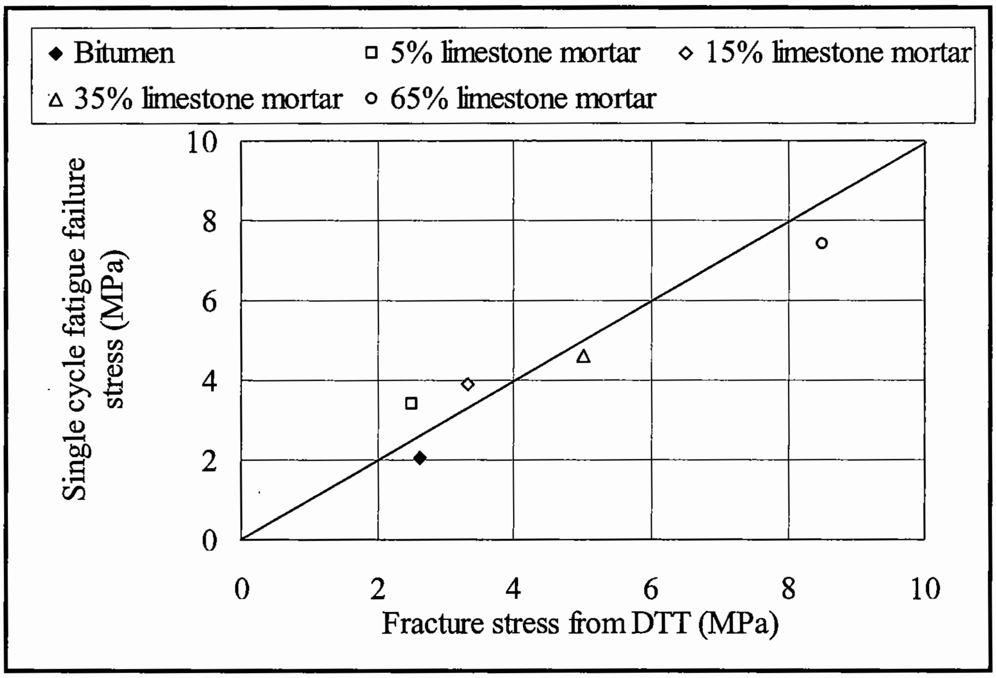


Figure 7.10 Stress comparisons for the DTT data and mortar fatigue tests for different mortars and bitumen

7.4 Fatigue Results of Bitumen, Mortars and Mixtures

Researchers have often looked for relationships between mechanical properties of bituminous mixtures and rheological properties of bitumen when assessing the resistance of mixtures and binder to fatigue.

Bitumen and mortars were tested using the new fatigue equipment developed especially for this project. The materials tested are the same as those tested in the DTT. The results were presented in Chapter 5.

The comparison of the test results obtained from both flat and rounded ends was discussed in Chapter 5. Figure 7.11 shows the strain results for the mortars tested with flat end, rounded end and mixtures in the 4-point bend test containing different filler contents. For the mixtures, the fatigue lines based on the strain are almost similar, compared with those of the mortars where different fatigue lines were obtained depending on filler content.

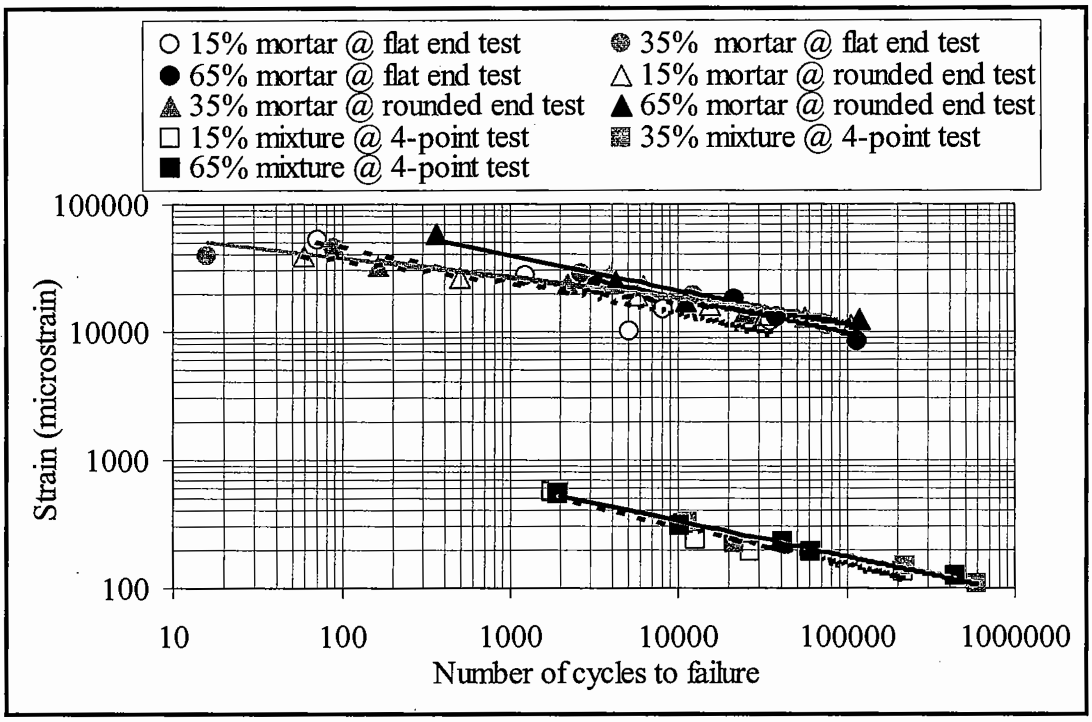


Figure 7.11 Strain against number of cycles to failure for mortars and mixtures containing different filler content tested at 10°C

Figure 7.12 shows the stress against number of cycles to failure for bitumen, mortar and mixture (tested in the 4-point bend test). The mortar and the mixture fatigue lines seem to be relatively parallel with higher fatigue life obtained for the mixture materials.

The effect of filler type on fatigue is illustrated in Figure 7.13, where the mortar and mixtures containing different filler types were tested, with flat ends, and in the 4-point bend test respectively. The results indicate that filler type has little effect on the fatigue life of either the mortar or the mixture, compared to the effect of filler concentration. When plotted against strain the results show clearly that filler type has little effect on fatigue life for the materials tested in this research. The different test methods generate similar parallel fatigue lines. The strain in the mortar is much higher than in the mixture. This observation is illustrated by Figure 7.14, where the results are presented on the basis of strain for mortars and mixtures tested by two different methods. The mixture fatigue results as shown in Figures 7.12 and 7.13 seem to be primarily strain dependent.

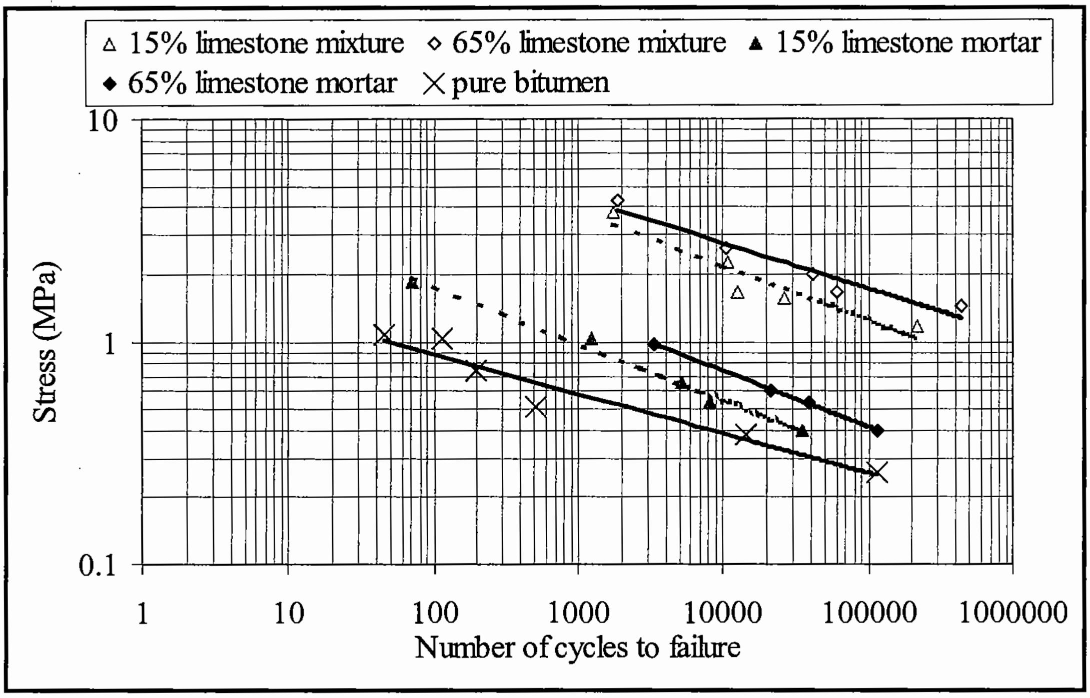


Figure 7.12 Stress against number of cycles to failure for mortars and mixtures tested at 10°C

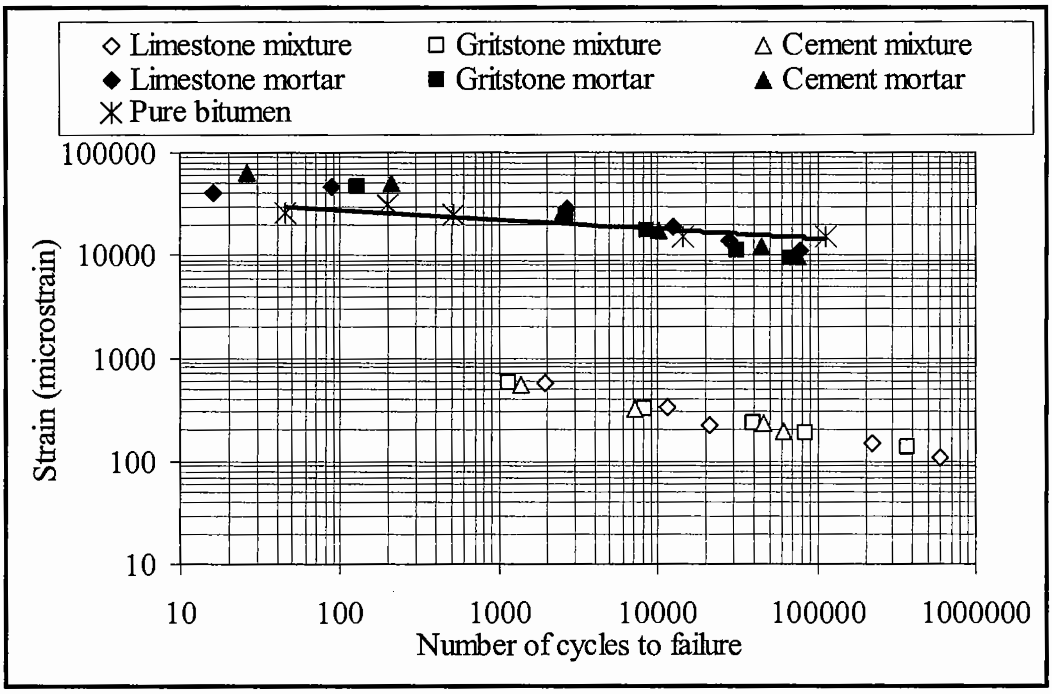


Figure 7.13 Strain against number of cycles to failure for mortars and mixtures containing different filler type tested at 10°C

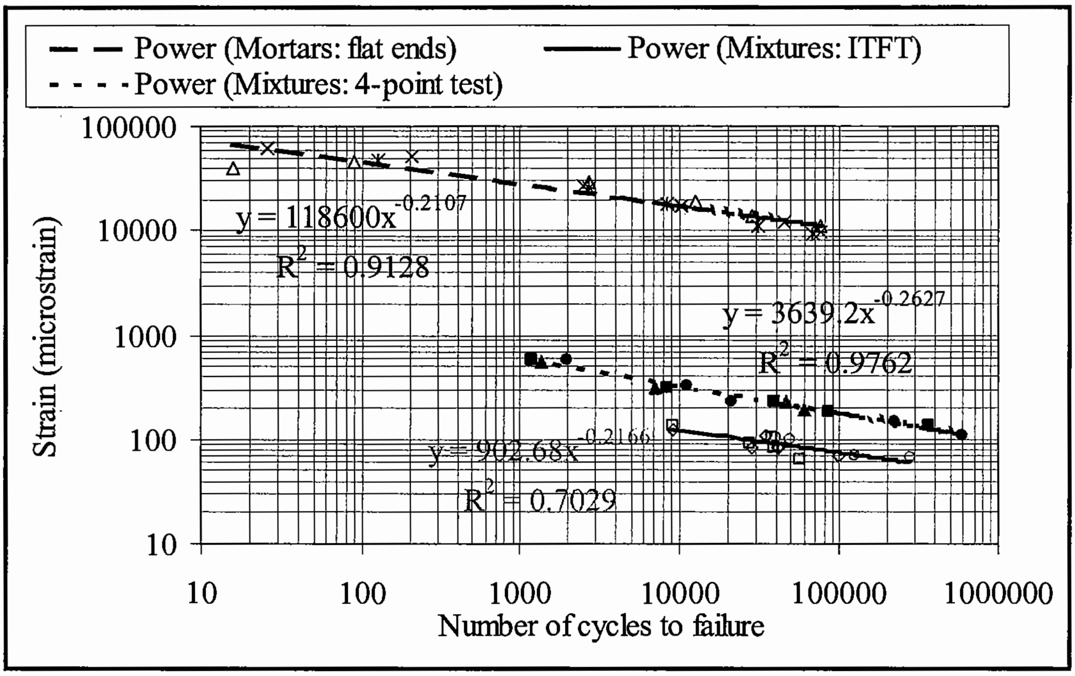


Figure 7.14 Strain against number of cycles to failure for mortars and mixtures tested at 10°C using different tests

When mixtures are subjected to deformation on application of a stress, the aggregate acts as a load bearing material, and the binders deform in response to applied stress. In actual bituminous mixtures, a wide range of mortar film thicknesses will exist and therefore a wide range of strain amplitudes could be present within the mortar domain. Because of the large difference in stiffness between the mortar and the aggregate, most of the bulk strain will be in the mortar domain. Also because of non-uniformity of the mortar domain, it is expected that the distribution of strain is very complex. In order to compare the fatigue behaviour of mixtures and binder, the strains to generate failure after one million cycles for the binder and mixtures were compared.

Figure 7.15 shows the relationship between strain in the mortar and strain in the bituminous mixture giving failure after one million cycles, tested at the same temperature and frequency using the flat ends and the 4-point bend tests. A poor relation between binder and mixture behaviour is obtained. It also shows that the mortar can be subjected to strain levels much higher than the strain to which the mixtures are subjected. Mixture is fatigue relatively insensitive to variables such as filler content compare to mortar fatigue.

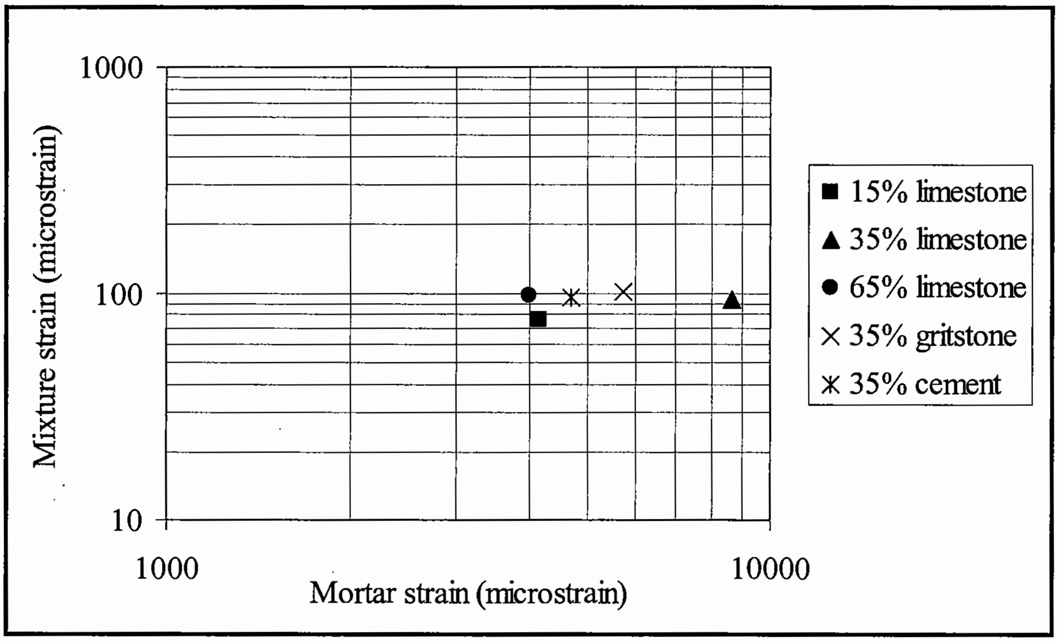


Figure 7.15 Mixture strain against mortar strain for failure after one million cycles for specimens tested by the 4-point and flat ends tests at +10°C and 10Hz

Figure 7.15 suggests that it is realistic to assume that, for the materials tested in this research, strains within the binder film can be as high as 40 to 100 times the bulk strain of the total mixture. Bahia et al. [88] assumed that strain within the binder can be as high as 10 to 100 times the bulk mixture strain.

7.4.1 Stiffness Reduction

Stiffness reduction was observed on fatigue tests for pure bitumen, mortars and mixtures. Nearly similar trends of stiffness reduction behaviour occurred. However, for pure bitumen and mortar, the curve was much flatter over the middle zone followed by sudden fracture at the end, whereas the mixture curves started with reduction followed by a slower rate of reduction over the middle zone until the crack started and propagated in the final stage as shown in Figure 7.16. The two failure points on the figure, explain as follows: for the bituminous mixture the failure point is at 50% reduction of the initial stiffness, whereas for the mortar is the complete failure of the specimen.

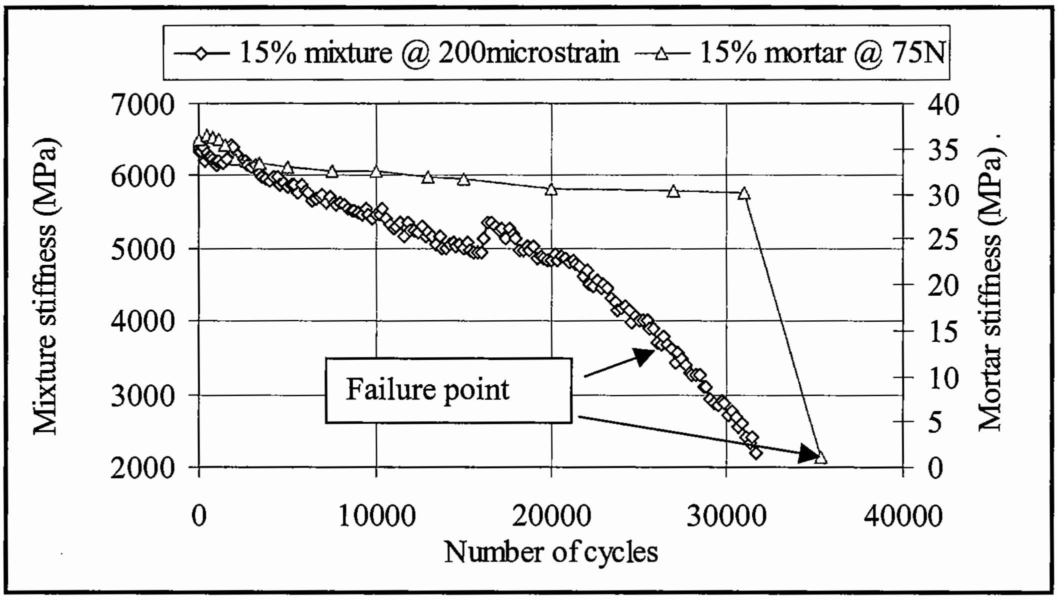


Figure 7.16 Stiffness against number of cycles for a mixture and mortar tested at +10°C and 10hz

The results of the stiffness reduction up to the failure point of the specimen, show that for pure bitumen and mortars it seems possible that the specimen may come to the fracture point without propagation of a crack but as a result of progressive weakening of the molecular bond or cohesion structure of the mortar, as indicated by the sudden fracture at the end. Sudden fracture for pure bitumen has also observed by other researchers [7] investigating the fracture and healing of bitumen. But mixtures have different stiffness reduction behaviour, three phases appearing successively during testing as reported in Chapter 6 Figure 6.14. The possible explanation for this, in phase I the stiffness reduction may partially due to generated heat during cyclic loading, whereas in phase II correspond to cracking (or micro-crack) process in the material, while during phase III cracks (or macro cracks) begin to develop and complete failure is obtained at the end of this phase.

7.4.2 Fracture Concept

The mechanical properties of a material are a manifestation of the way in which the individual bonds in the material respond to an applied stress. From the chemical point of view, bitumen is a very complex material. It consists of a very great number of organic compounds structured in a complex way, making bitumen fracture more complicated to understand. It is deduced that fracture of bitumen tends to take place suddenly; the possible explanation being that as the load is applied the molecules re-arrange themselves to resist the stress. Thus there must be some intermolecular forces between molecules that hold the molecules together, but when the molecule chain breaks then sudden total fracture will occur as shown in Figure 7.17(a). The amount of energy that must be supplied to break a chemical bond in an isolated molecule is called the bond dissociation energy. Every bond in every molecule has its own specific bond dissociation energy.

On the other hand once bitumen is filled with filler, the crack has to propagate around the filler particles which will increase the fracture area and the crack has to take a longer distance as shown in Figure 7.17(b). The result is that more filler particles hold more stress.

In mixtures cohesive failure would take place if the failure is completely within the bitumen (or mortar) phase as shown in Figure 7.17(c). It can be expected that the test measuring binder properties can only be predictive with regard to cohesive failure. The possible explanation of mixture failure is that when a mixture is subjected to stress, micro-cracks tend to form immediately within bitumen/fines mortar at particle contacts where the strains are high, and in the area of high stress concentration. Then the crack cannot propagate far before it reaches a zone of much reduced strain and the cracks propagate around the particles.

The path of a crack in the vicinity of a filler particle depends on the initial approach path and on the strength of the interfacial adhesion. If the interfacial bond is weak the approaching crack tip will cause debonding and crack propagation will be governed by the breakdown of adhesion instead of cohesion. If a crack approaches a hard and well bonded filler particle the stresses at the crack tip decrease by an amount depending on the alignment of the crack plane with the diametral plane of the filler particle. Actually the crack mechanism is very complex with the inclusion of filler particles. However, it is assumed that the failure surface is approximately perpendicular to the load direction, and the crack path runs around the filler particles (low bitumen toughness).

The failure surface in a mixture becomes more complicated still due to the particle gradation and the distribution of the stresses within the bitumen/fines mortar. But referring to Figure 7.12 the strength ratio of mortar to binder is approximately similar to that of mixture to mortar.

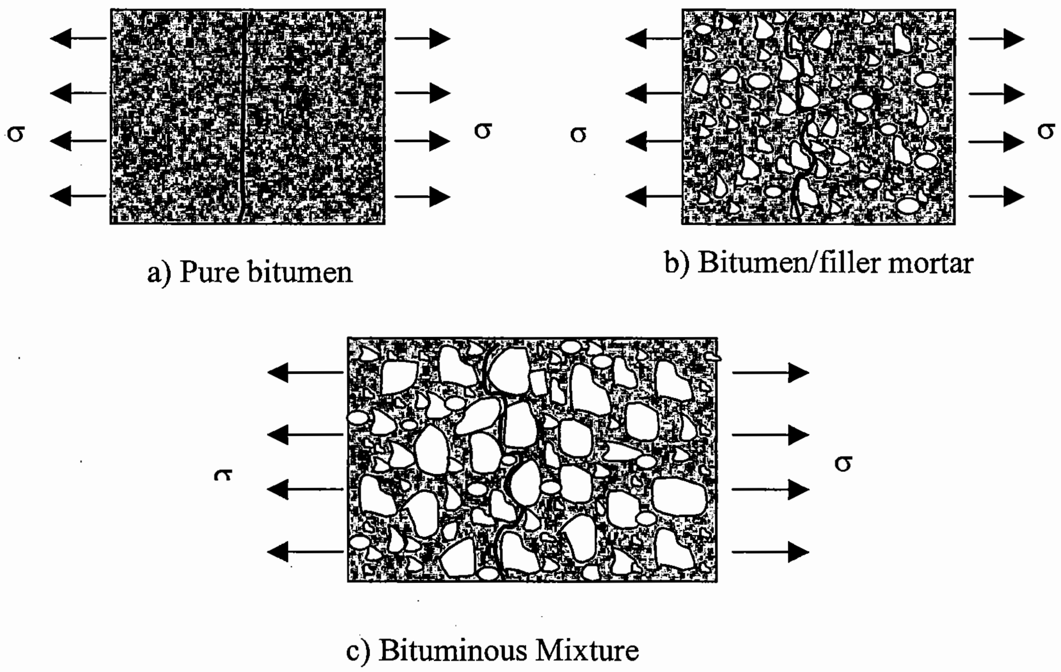


Figure 7.17 Specimen fracture as crack tends to run through the bitumen

7.5 Summary

The key points derived from these analyse and discussions are:

- A difference in viscosity results when different fillers are mixed with bitumen. Fillers induce non-Newtonian flow behaviour provided filler concentrations are sufficiently high. A large increase in mortar viscosity results when concentrations of filler are sufficiently high.
- Since filler induces non-Newtonian flow behaviour in the bitumen-filler mortar, viscoelastic behaviour of the bituminous mixture could be influenced by the type and concentration of the filler.

- A modified relation was used to estimate the peak stress during viscous flow of the mortar as a function of filler volume fraction. It agreed well with the experimental data of the materials tested in this research.
- Within the stress, strain rate and temperature range of the DTT tests, it can be seen that the viscous behaviour of the bitumen and mortars at a particular temperature can be represented by a power law relationship.
- The results also indicate that mortars containing smaller filler particles exhibit higher stress at failure. The reason may be the increase in interfacial area per unit volume of filler as particle size decreases.
- It is possible to construct a master curve for the stress and strain for the materials tested in the DTT using time-temperature superposition. The master curve will extend the strain rate range, which was limited in the experimental data.
- All the results confirm that filler type has little effect on fatigue life for the materials tested in this research. The different test methods generate similar parallel fatigue lines.
- The strain to cause failure at a given number of load applications in the mortar is much higher than in the mixture. The results show that it is realistic to assume that, for the materials tested in this research, strains within the binder film can be as high as 40 to 100 times the bulk strains of the total mixture.
- The master curve in conjunction with the fatigue data generated from the new binder fatigue test can be used to correlate the stresses obtained from both the fatigue and DTT tests. Good correlation was obtained from the limited experimental data in this research, giveing the possibility of correlating to some extent the test methods for the bitumen and bitumen-filler mortar.
- It can be concluded that the new binder and mortar fatigue equipment performs successfully since the results compare well with the mixture data. It appears to be a promising tool to study the fatigue behaviour of the bitumen and mortar together.
- It appears that pure bitumen and mortar specimens may come to the fracture point without propagation of a crack, as a result of progressive weakening of the molecular bond or cohesion structure of the mortar.

- Mixture failure might be explained by supposing that micro-cracks tend to form immediately within the bitumen/fines mortar at particle contacts where the strains are high, and in the area of high stress concentration. However, the crack cannot propagate far before it reaches a zone of much reduced strain, after which the cracks propagate around the particles.

8

Conclusions and Suggestions for Future Work

This research investigates the role of bitumen and bitumen/filler mortar in asphalt mixture fatigue. The conclusions presented in this chapter were drawn from the work done in this project.

8.1 Conclusions

8.1.1 Direct Tension Test

50pen bitumen and four types of fillers have been tested in the DTT using different temperatures and strain rates. Different filler concentrations were used for limestone filler mortar. The following conclusions were drawn from the investigation using the DTT:

- At high temperature bitumen failed in viscous flow mode without any definite failure point, whereas mortar failed in a ductile mode with a maximum stress followed by a decrease in stress. At low temperatures, both bitumen and mortar fractured completely in brittle mode.

- For a particular temperature, bitumen and mortar show that increasing strain rate will increase the stress at failure. Also, increasing testing temperature will decrease the stress at failure. The addition of filler increases the ductile failure stress.

- Limestone mortar and cement mortar had the same failure behaviour, but gritstone mortar gave higher stresses at failure than the others under the same testing conditions. Sewage sludge ash gave slightly higher stress at failure compared with limestone.

- It was assumed that size, shape and texture of the filler particle were the fundamental physical properties governing its effect on the binder.

- Mortars containing smaller filler particles exhibit higher stress at failure.

- The results show that within the range of testing conditions, at a particular temperature, the bitumen and bitumen-mortar behave non-linearly. The stress response can be represented by a power law equation.

- A modified relation was used to estimate the strength of the mortar as a function of filler volume fraction. It agreed well with the experimental data for the materials tested in this research.

- At high testing temperature, differences in tensile viscosity result when different fillers are mixed with bitumen. Filler induces non-Newtonian flow behaviour provided filler concentrations are sufficiently high.

- Strain at brittle failure occurs at a strain less than or equal to 11%. The majority of the mortars tested had a stress at failure of about 3MPa at low filler concentration and up to 9MPa for high filler concentration, where the mortar exhibits non-Newtonian flow.

- Brittle failure stress did not vary much over quite a wide range of strain, i.e. failure appeared to be stress dependent.

- It is possible to construct a master curve for the stress, strain and stiffness for the materials tested in the DTT using time-temperature superposition.

- The master curve, in conjunction with the fatigue data generated from the new binder fatigue test, can be used to correlate the stresses obtained from both testing methods. Good correlation was obtained from the limited experimental data in this research.

8.1.2 Bitumen and Mortar Fatigue

Bitumen and bitumen/filler mortar has been tested using both flat and rounded end platens at a temperature of +10°C and a frequency of 10Hz at different load levels. Four types of filler have been used in the testing programme for the flat end tests with different filler concentrations, but only limestone filler has been used on the rounded end tests with different filler contents. The following conclusions were drawn from the test results:

- Frequency does not have a large effect on the fatigue life for the bitumen tested in this research, when the fatigue life of the binder is considered with respect to the stress and number of cycles to failure.

- The results from the flat end tests show that filler type does not have a great effect on the fatigue life of the binder, as the mortars containing cement, limestone, and gritstone gave almost the same fatigue life with no significant difference.

- Filler content has an effect on the fatigue life, as shown by limestone added at different filler percentage contents.

- Filler particle size is not the main factor affecting fatigue properties, but the shape and the texture of the filler particles play a major role.

- Stiffness reduction was observed during fatigue tests on both bitumen and mortars containing different filler types and percentage filler contents. The initial stiffness reduction is thought to be partially as a result of generated heat during load cycling.

- The master curve, in conjunction with the fatigue data generated from the new binder fatigue test, can be used to correlate the stresses obtained from both testing methods. Good correlation was obtained from the limited experimental data in this research.

8.1.3 Asphalt Mixture Fatigue

Three mixtures containing different limestone filler contents were tested in the 4-point bend test, in addition to three mixtures containing different filler types tested on both the 4-point bend test and ITFT test at a temperature of +10°C. The following conclusions were drawn from the results of testing:

- The results indicate that filler concentration in the mixture has an effect on the fatigue resistance to failure.

- All the results confirm that filler type has no effect on fatigue life for the materials tested in this research. The different test methods generate similar parallel fatigue lines.

- Both the test methods used can generate a single fatigue life for the mixtures containing different filler types for the materials tested in this research.

- The ITFT method gave a shorter fatigue life for mixtures than 4-point bend test.

- Initial stiffness reduction was observed in the 4-point bend test for mixtures containing both different filler types and filler contents.

- The strain in the mortar at failure is much higher than that in the mixture. The results show that it is realistic to assume that, for the materials tested in this research, strains within the binder film can be as high as 40 to 100 times the bulk strains of the total mixture.

8.1.4 Comparison of Binder and Mixture Fatigue

- It can be concluded that the new binder and mortar fatigue equipment works successfully as the results compare well with the mixture data and that it can be a good tool to study the fatigue behaviour of the bitumen and mortar together.

- Scatter in the data is unavoidable, based on the non-homogeneity of the stress and strain field in the specimen tested.

- The stiffness reduction obtained at the beginning of the fatigue tests is independent of the type of test.

- The major reduction in the modulus occurred during the early stage of fatigue life, when there were no cracks in the specimen, this is thought to be due partially to heating caused by the accumulation of dissipated energy.

- Mixture fatigue is primarily strain dependent whereas mortar fatigue is stress dependent.

- It can be deduced that pure bitumen and mortar specimens may possibly reach their fracture point without the propagation of a crack, perhaps as a result of progressive weakening of the molecular bond or cohesion structure of the mortar.

- Mixture failure might be explained by suggesting that micro-cracks tend to form rapidly within the bitumen/fines mortar at particle contacts where the strains are high, in areas of high stress concentration. However, the crack cannot propagate far before it reaches a zone of much reduced strain, after which the cracks propagate around the particles.

- The strength ratio of mortar to binder is approximately similar to that of mixture to mortar, on a stress basis.

8.2 Suggestions for Future Work

- In order to fully understand the mechanical behaviour of bitumen, and the manner in which the mechanical behaviour relates to potential pavement performance, it is necessary to understand how bitumen chemistry affects the viscoelastic behaviour and fracture properties of bitumen.

- Because it is expected that the thickness of bitumen film in pavements is variable, it would be useful to study the mortar fatigue over a wide range of thicknesses.

- Debonding between platen and specimen is one of the main problems of the new fatigue testing equipment. Therefore the most appropriate materials to be used will need to be investigated. If possible using the same surface texture should be used as that of the aggregate as used in-situ.

- The sewage sludge ash waste gave remarkable results as a filler; more fatigue research needs to be done on it.

- The study should be extended to include various bitumens from different sources and bitumen modified by polymers to study their effects on fatigue properties.

- Research [38-43] has shown clear evidence of a healing mechanism occurring in a bituminous mixture. For bitumen and mortar, healing is expected to result in a partial regaining of stiffness, and fatigue life. It is recommended to study the mechanism of healing during rest periods.

- Research work is still needed to specify and quantify the physical mechanisms that accompany stiffness reduction, in particular the phenomena other than heating.

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APPENDIX

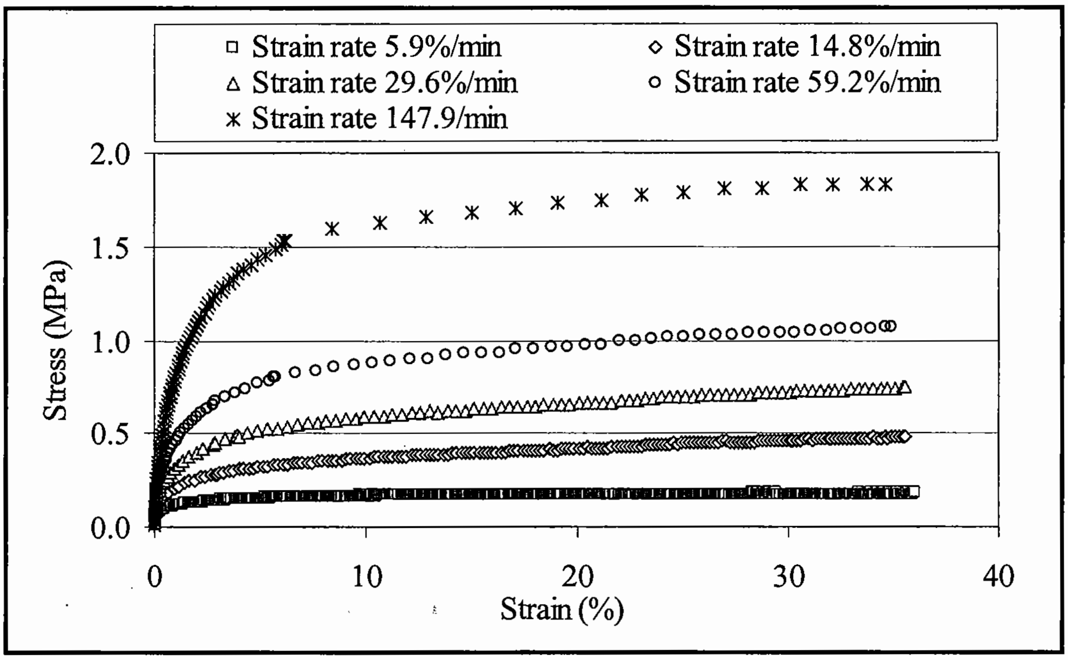

Figure 1 Stress against strain for pure bitumen tested at +5°C

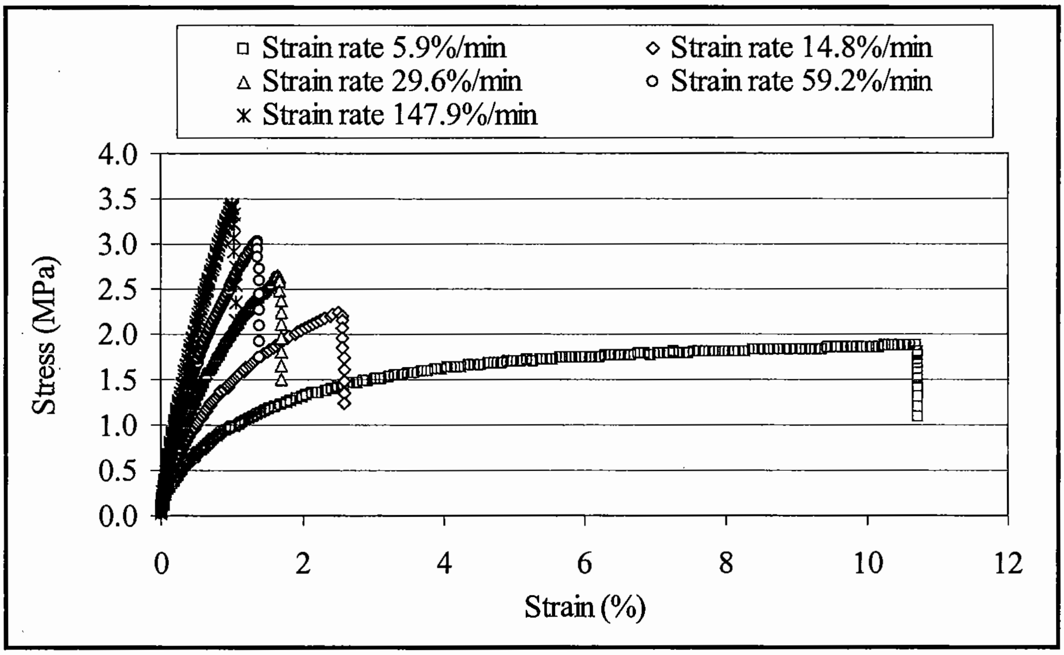

Figure 2 Stress against strain for pure bitumen tested at -5°C

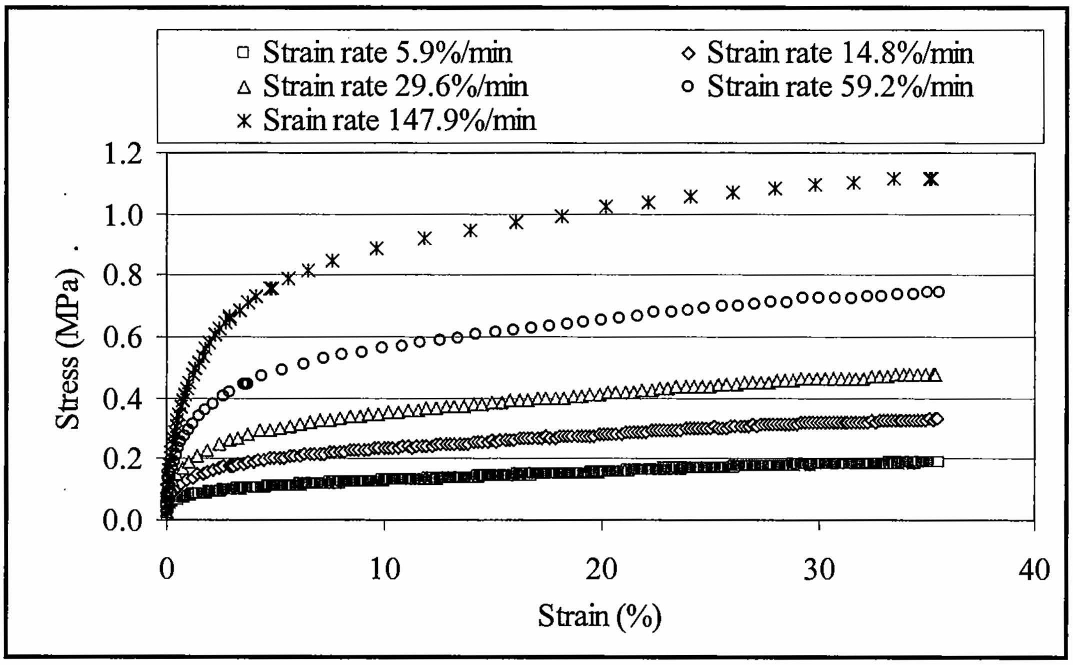

Figure 2 Stress against strain for a mortar containing 5% limestone filler tested at +10°C

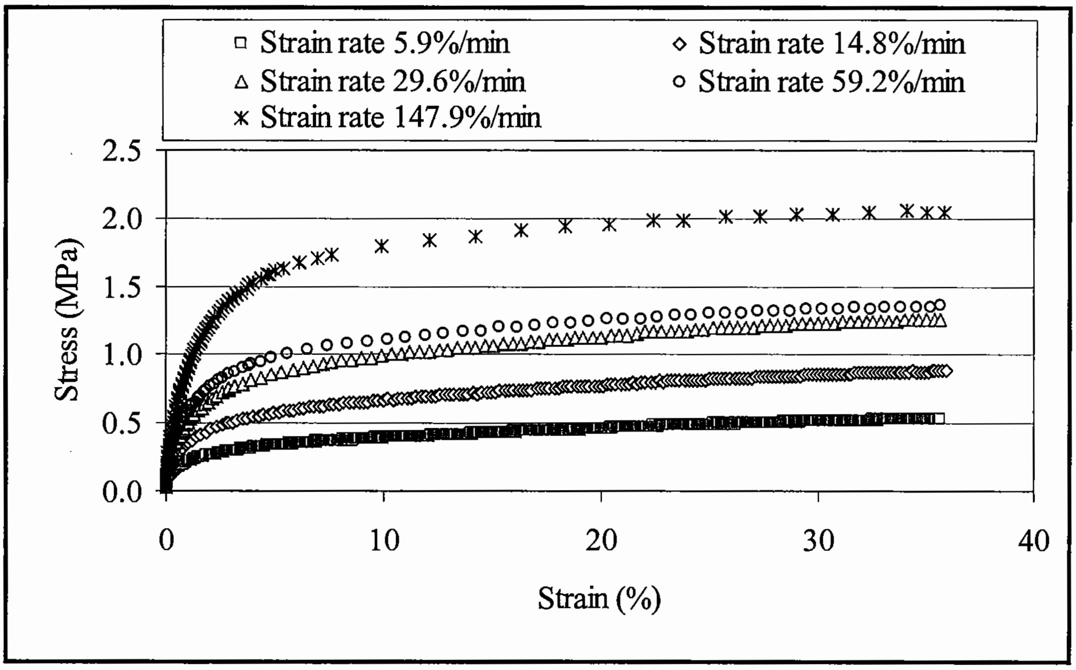

Figure 3 Stress against strain for a mortar containing 5% limestone filler tested at +5°C

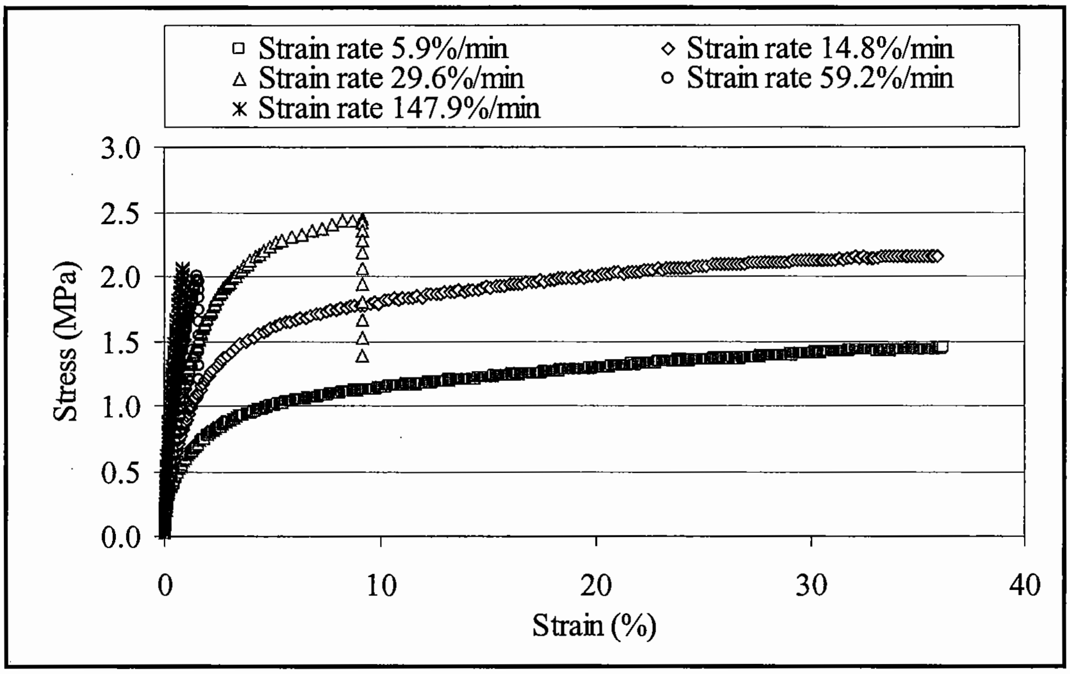

Figure 4 Stress against strain for a mortar containing 5% limestone filler tested at 0°C

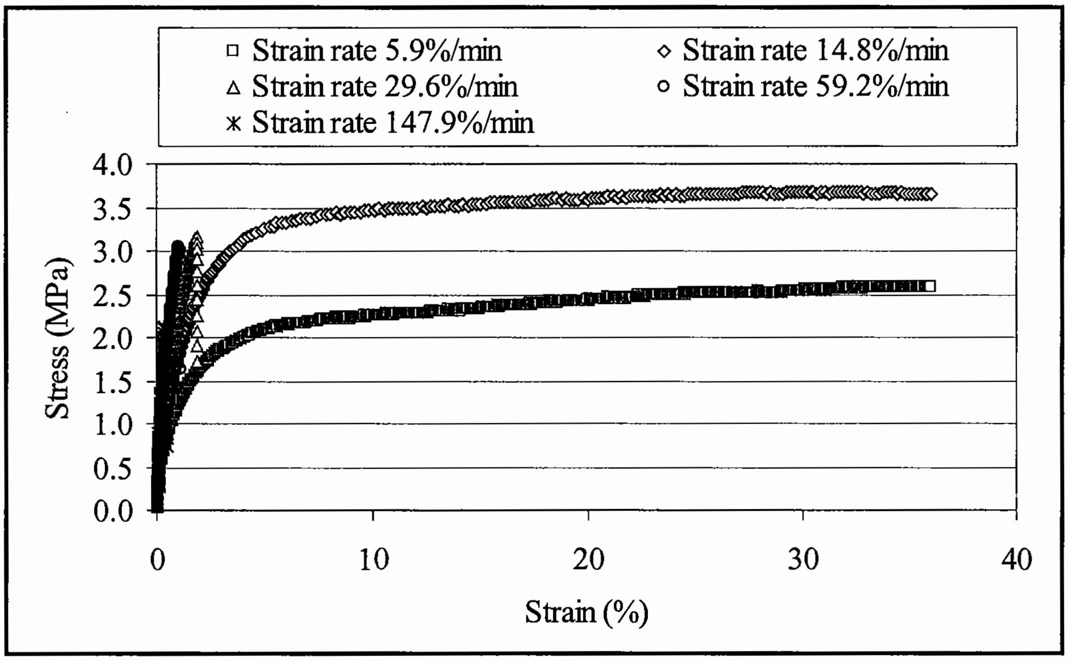

Figure 5 Stress against strain for a mortar containing 5% limestone filler tested at -5°C

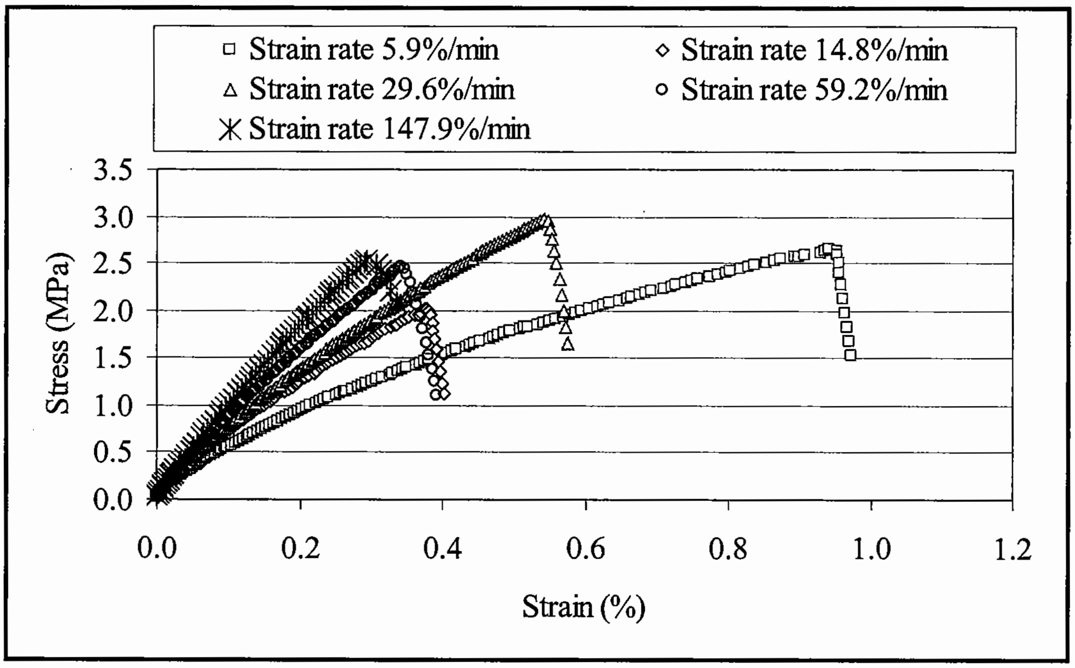

Figure 6 Stress against strain for a mortar containing 5% limestone filler tested at -10°C

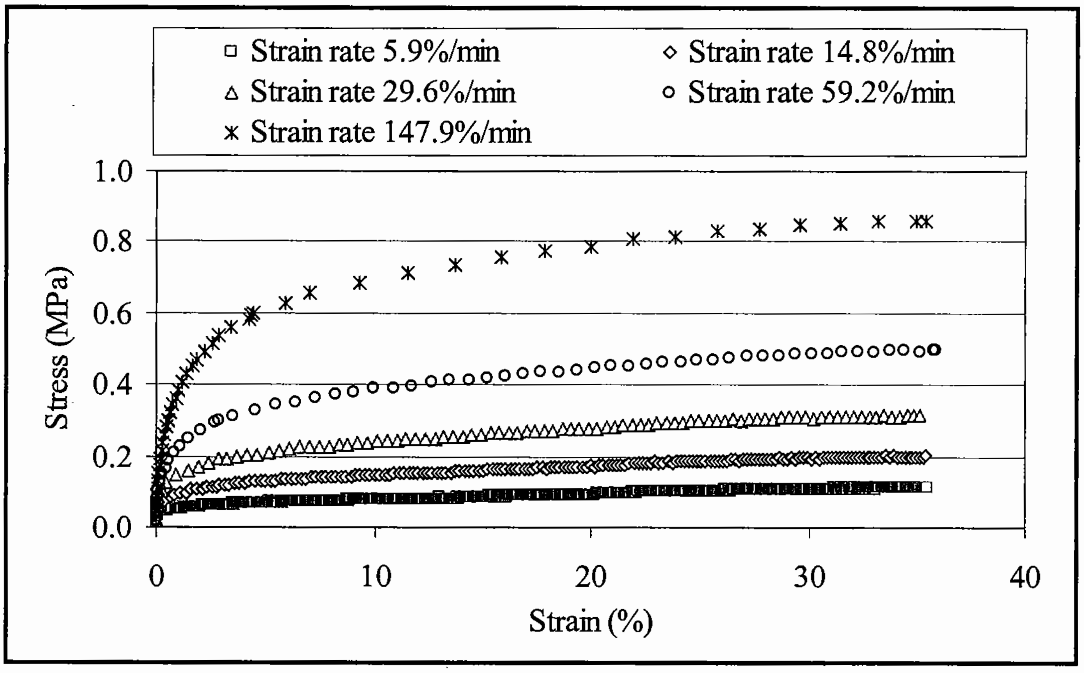

Figure 7 Stress against strain for a mortar containing 15% limestone filler tested at +10°C

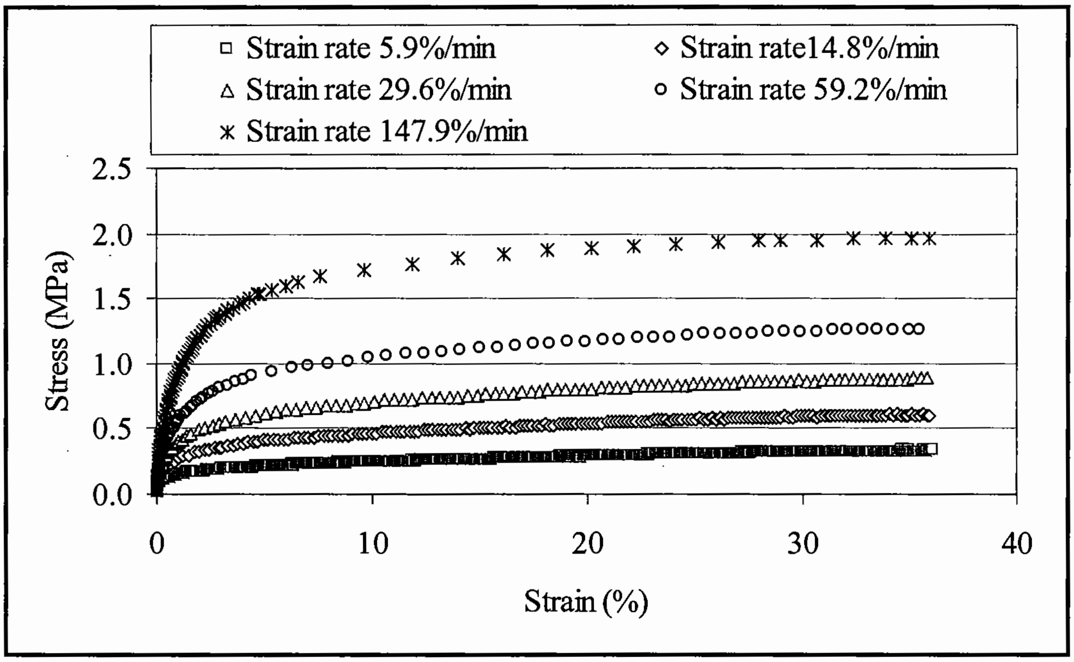

Figure 8 Stress against strain for a mortar containing 15% limestone filler tested at +5°C

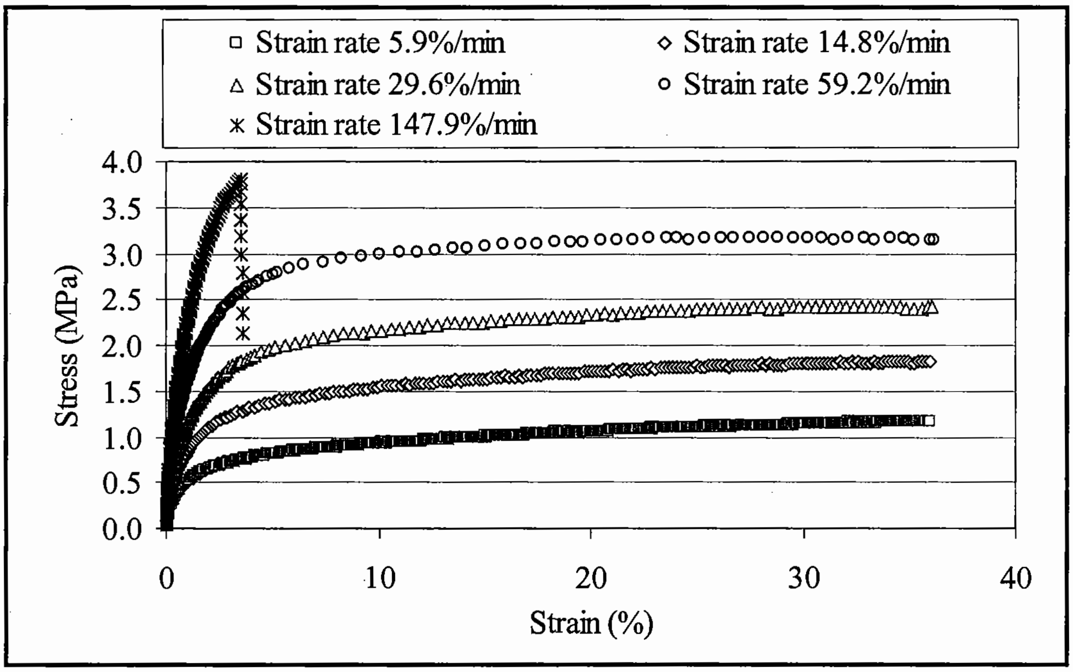

Figure 9 Stress against strain for a mortar containing 15% limestone filler tested at 0°C

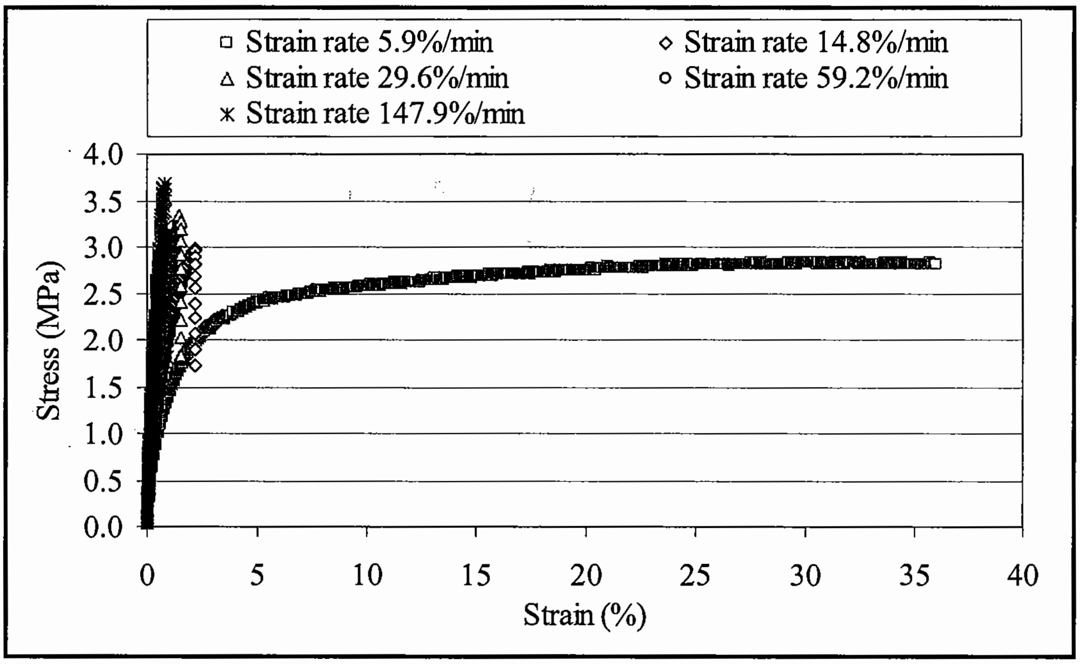

Figure 10 Stress against strain for a mortar containing 15% limestone filler tested at -5°C

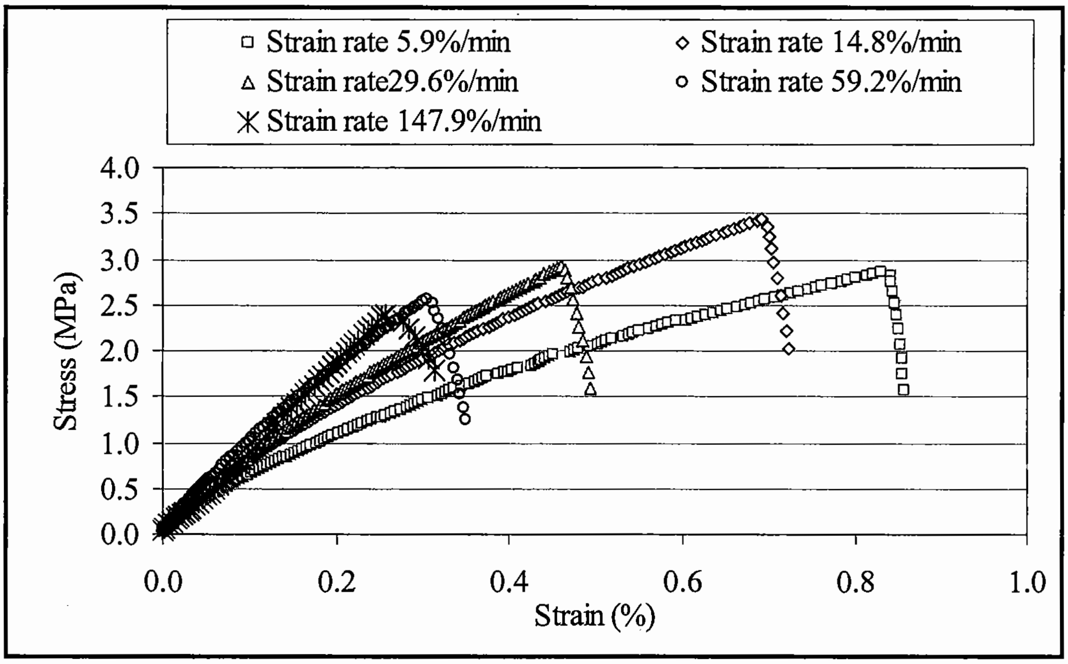

Figure 11 Stress against strain for a mortar containing 15% limestone filler tested at -10°C

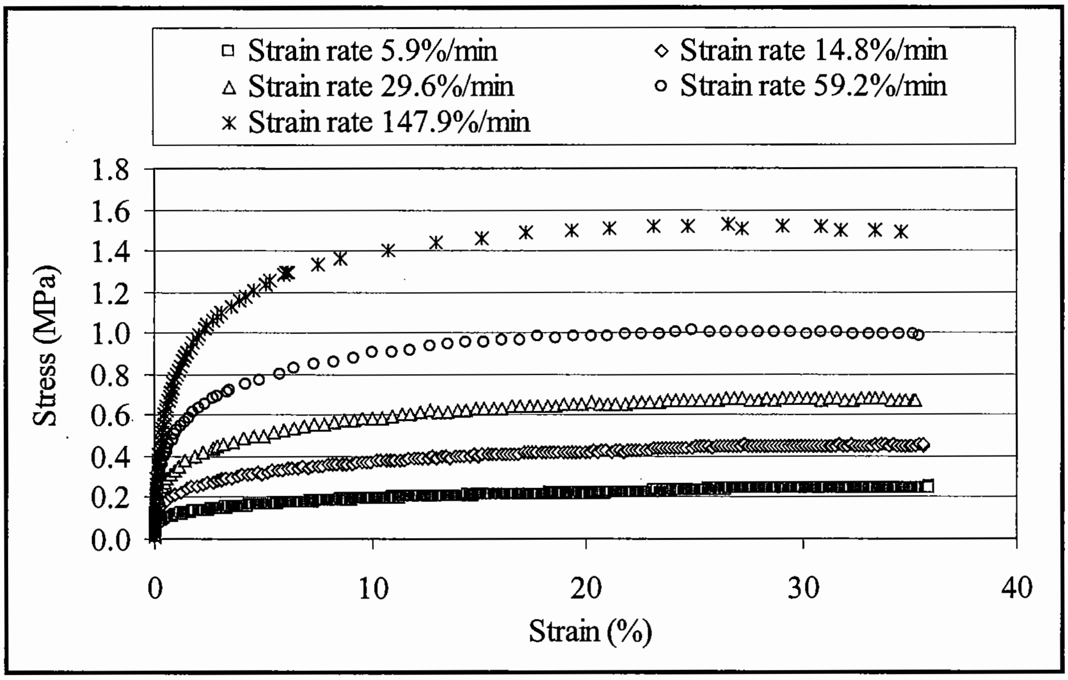

Figure 12 Stress against strain for a mortar containing 50% limestone filler tested at +10°C

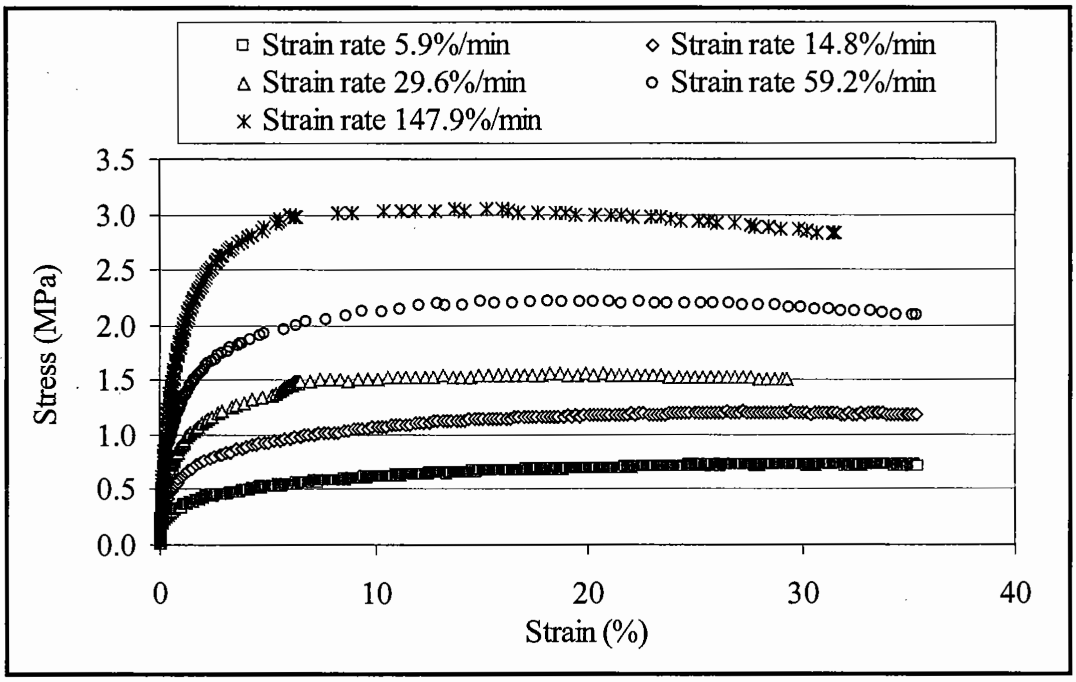

Figure 13 Stress against strain for a mortar containing 50% limestone filler tested at +5°C

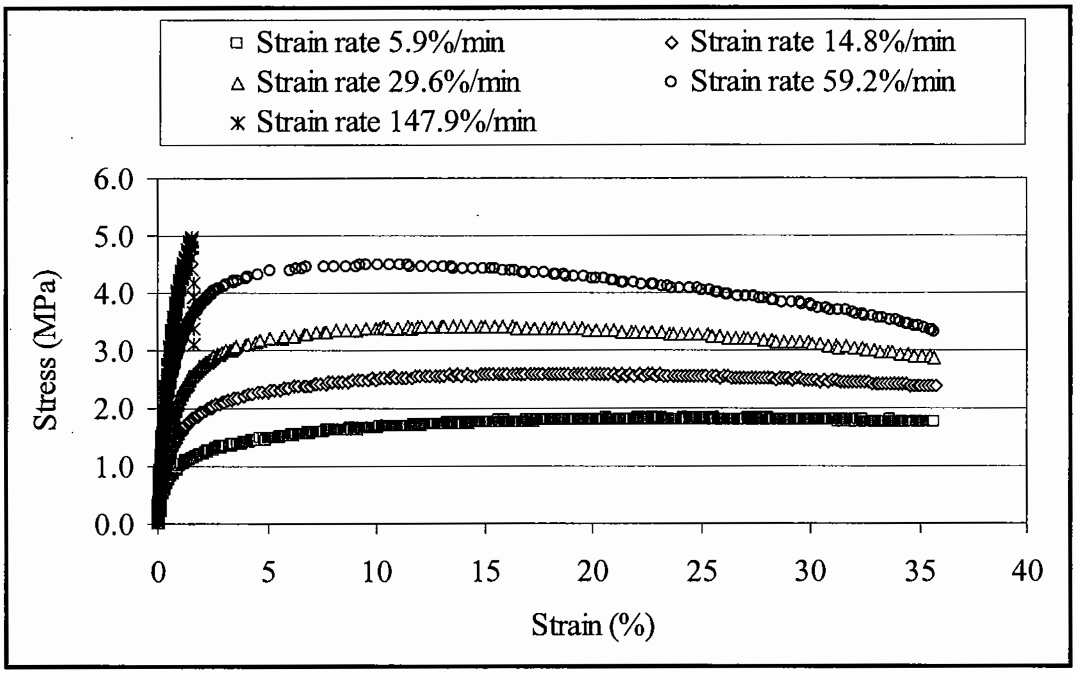

Figure 14 Stress against strain for a mortar containing 50% limestone filler tested at 0°C

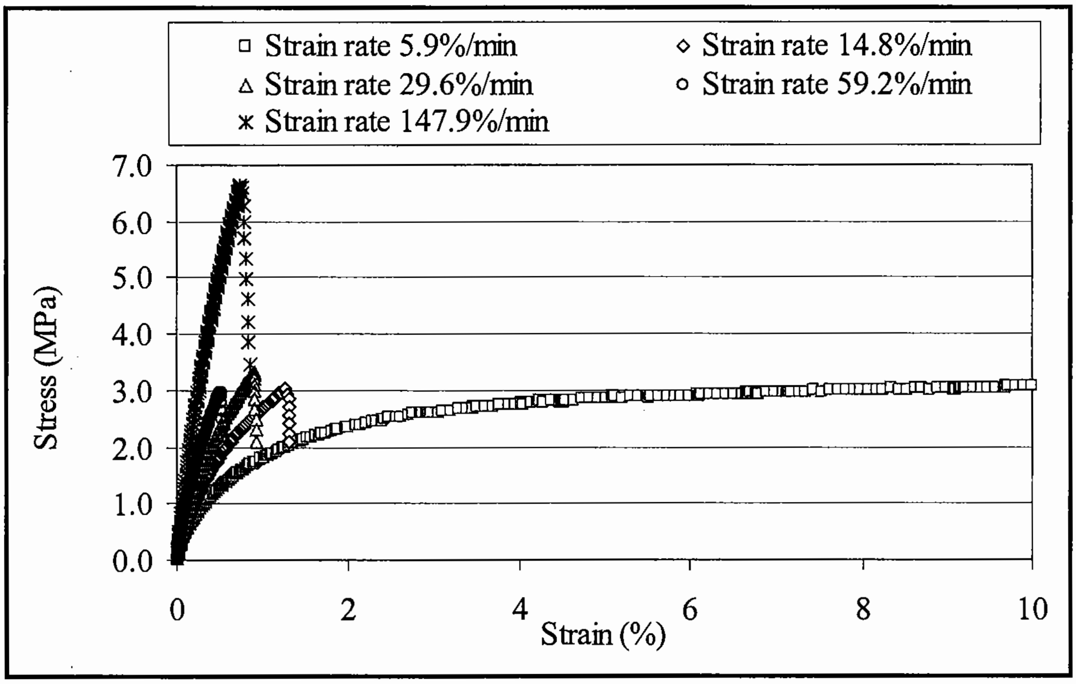

Figure 15 Stress against strain for a mortar containing 50% limestone filler tested at -5°C

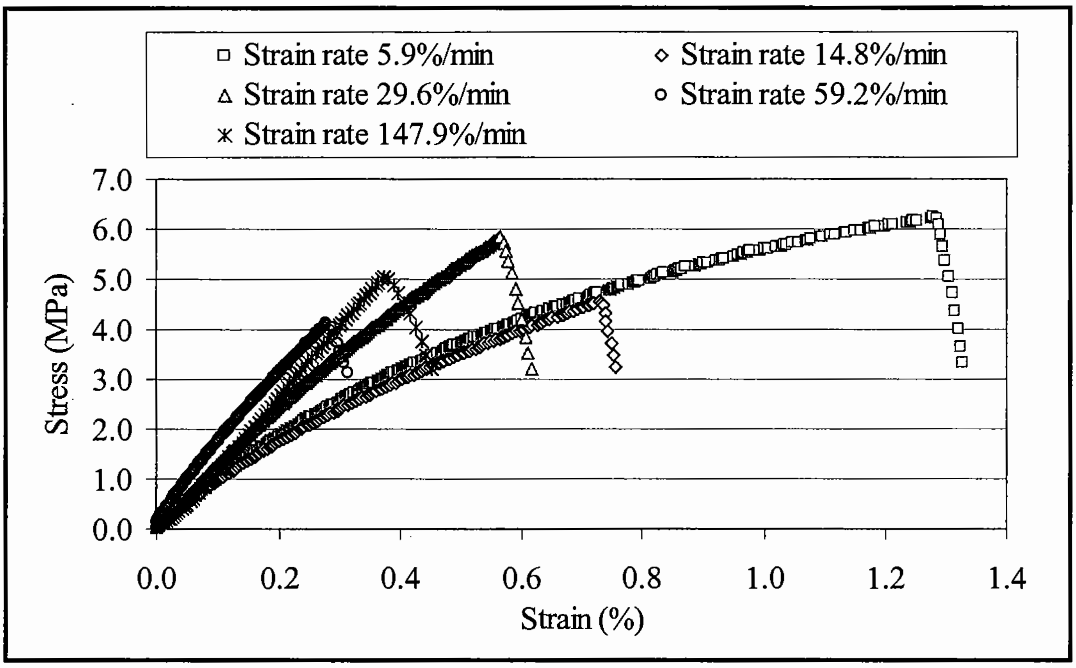

Figure 16 Stress against strain for a mortar containing 50% limestone filler tested at -10°C

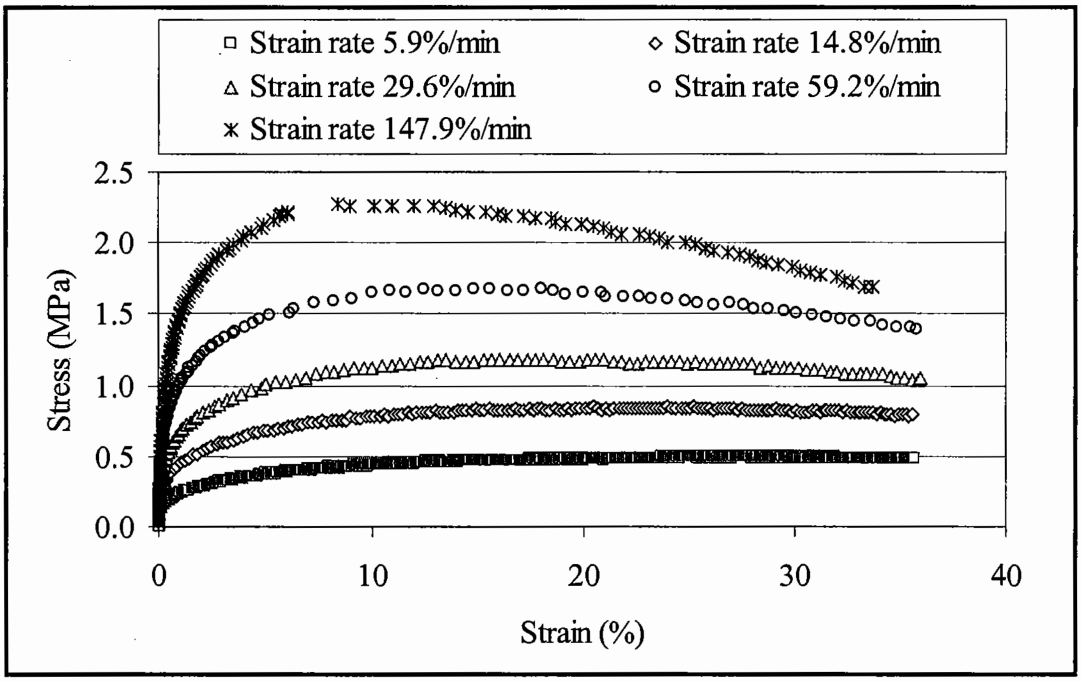

Figure 17 Stress against strain for a mortar containing 65% limestone filler tested at +10°C

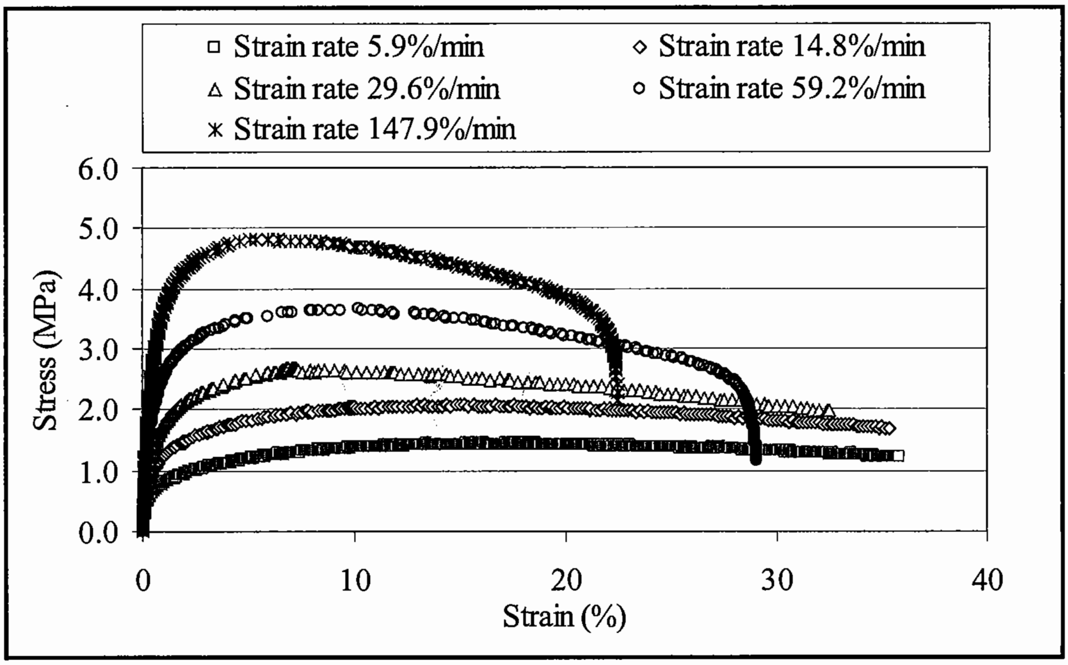

Figure 18 Stress against strain for a mortar containing 65% limestone filler tested at +5°C

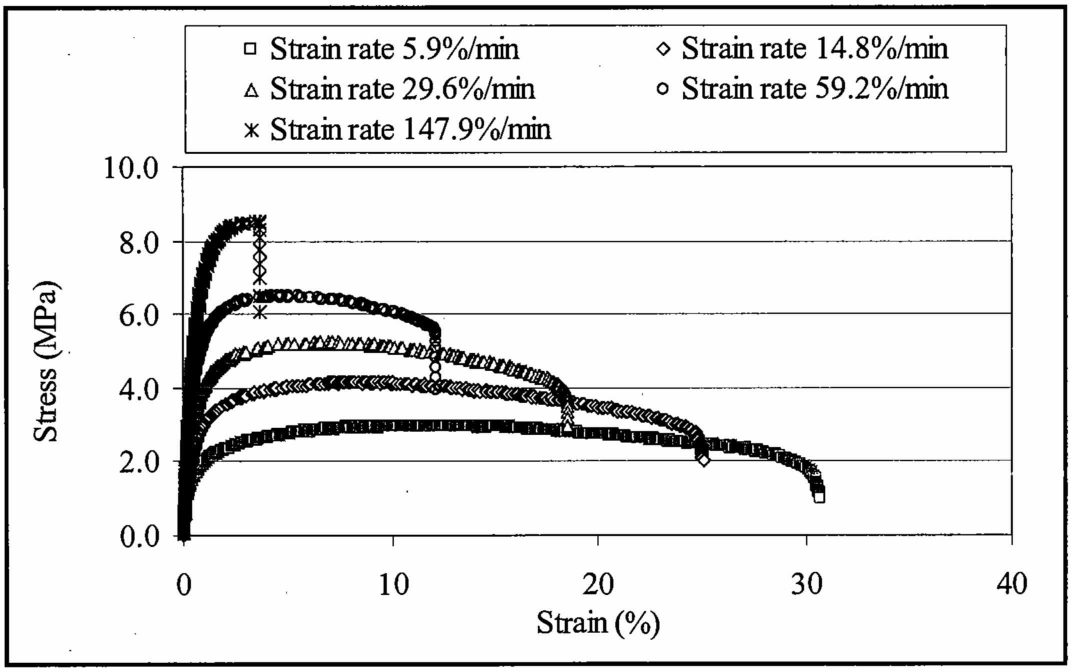

Figure 19 Stress against strain for a mortar containing 65% limestone filler tested at 0°C

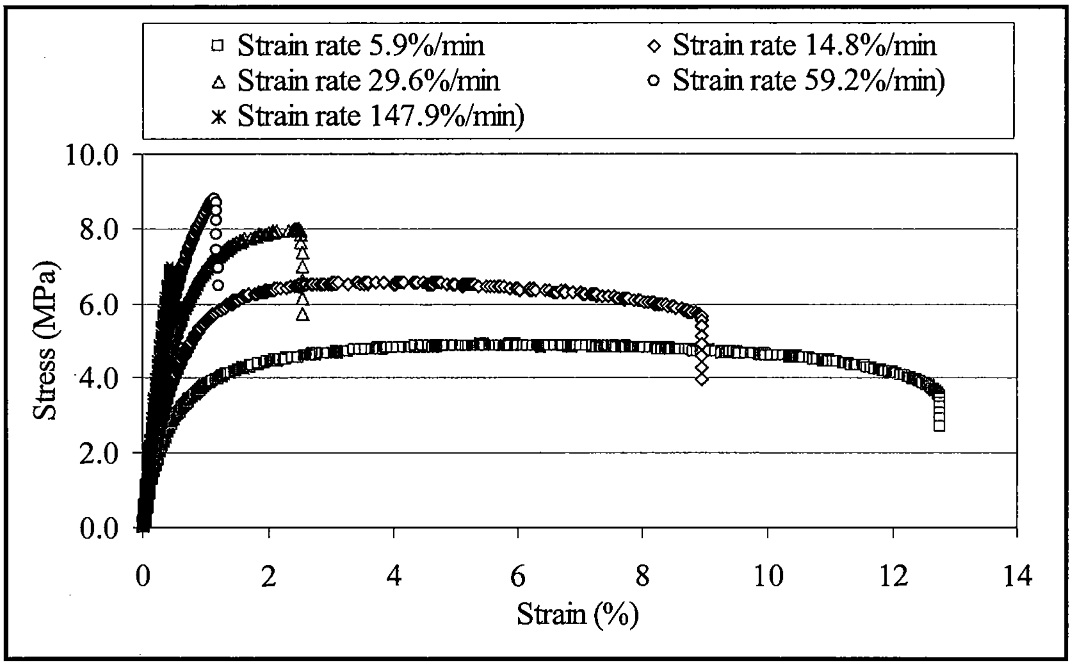

Figure 20 Stress against strain for a mortar containing 65% limestone filler tested at -5°C

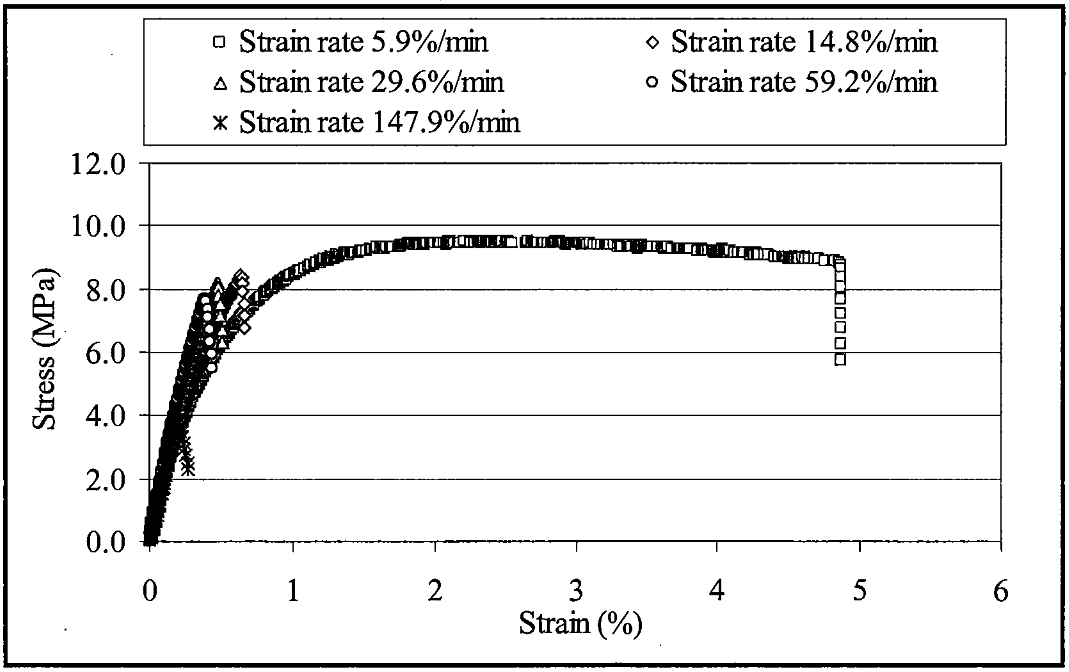

Figure 21 Stress against strain for a mortar containing 65% limestone filler tested at -10°C

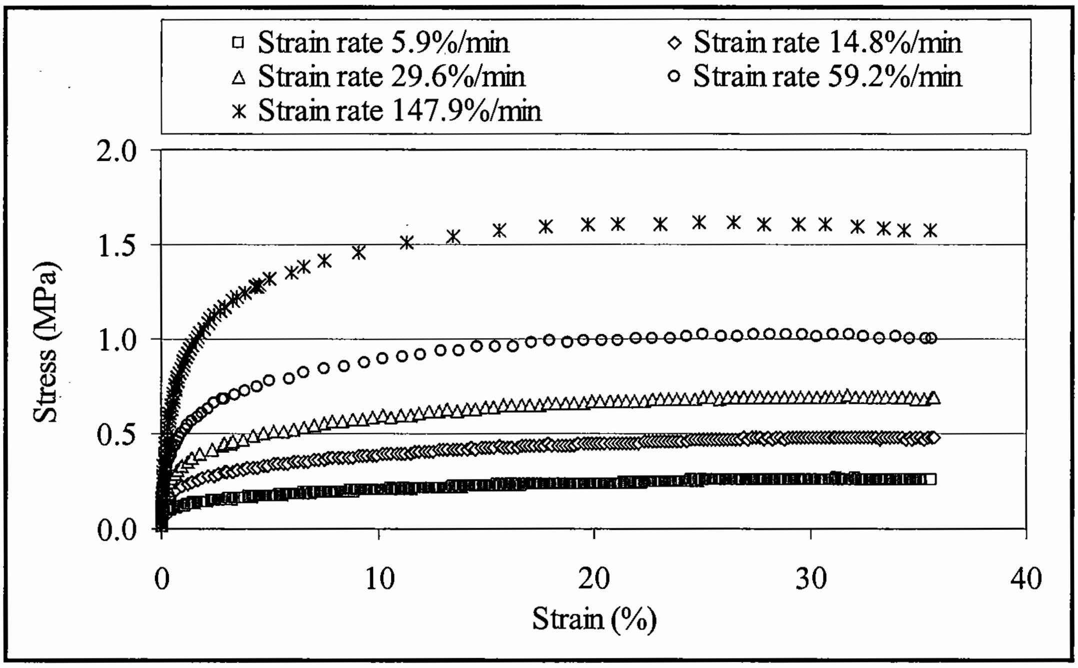

Figure 22 Stress against strain for a mortar containing 50% cement filler tested at +10°C

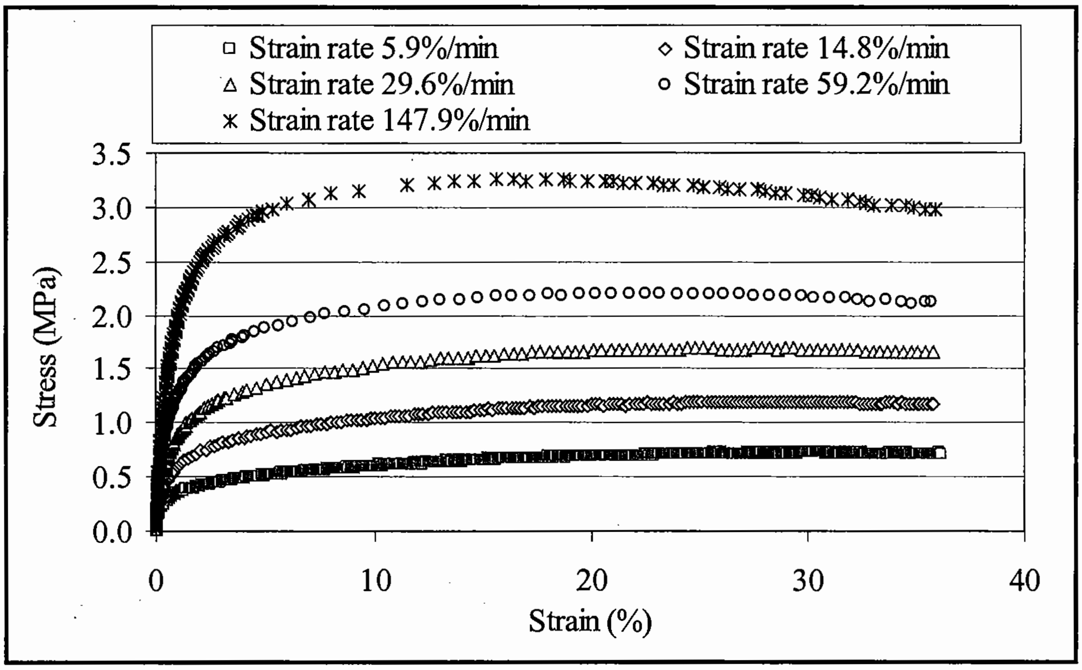

Figure 23 Stress against strain for a mortar containing 50% cement filler tested at +5°C

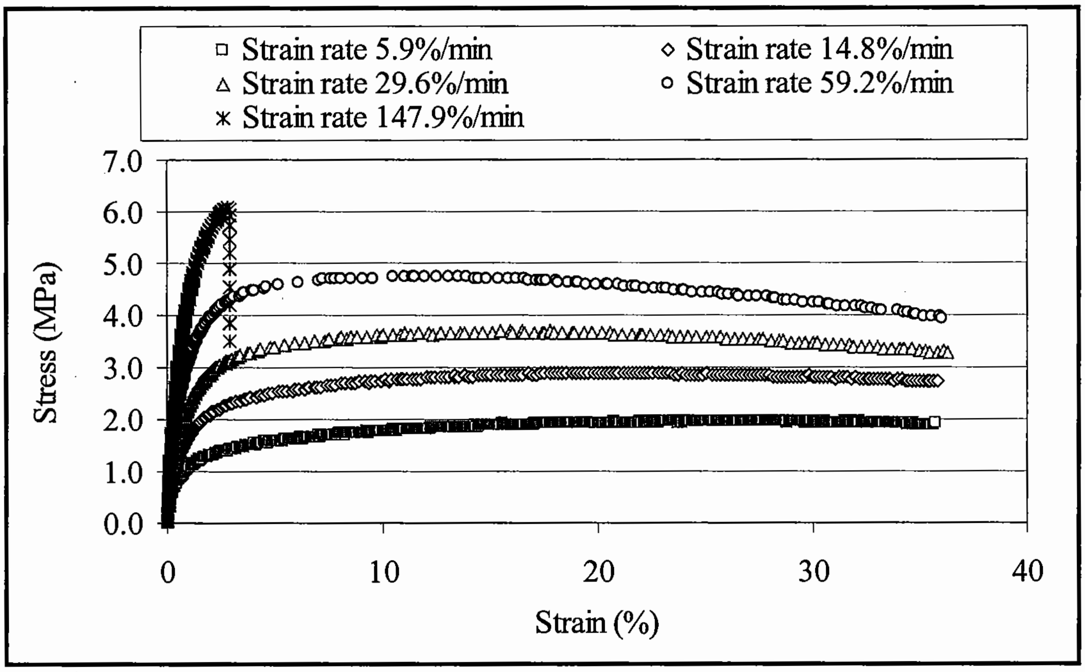

Figure 24 Stress against strain for a mortar containing 50% cement filler tested at 0°C

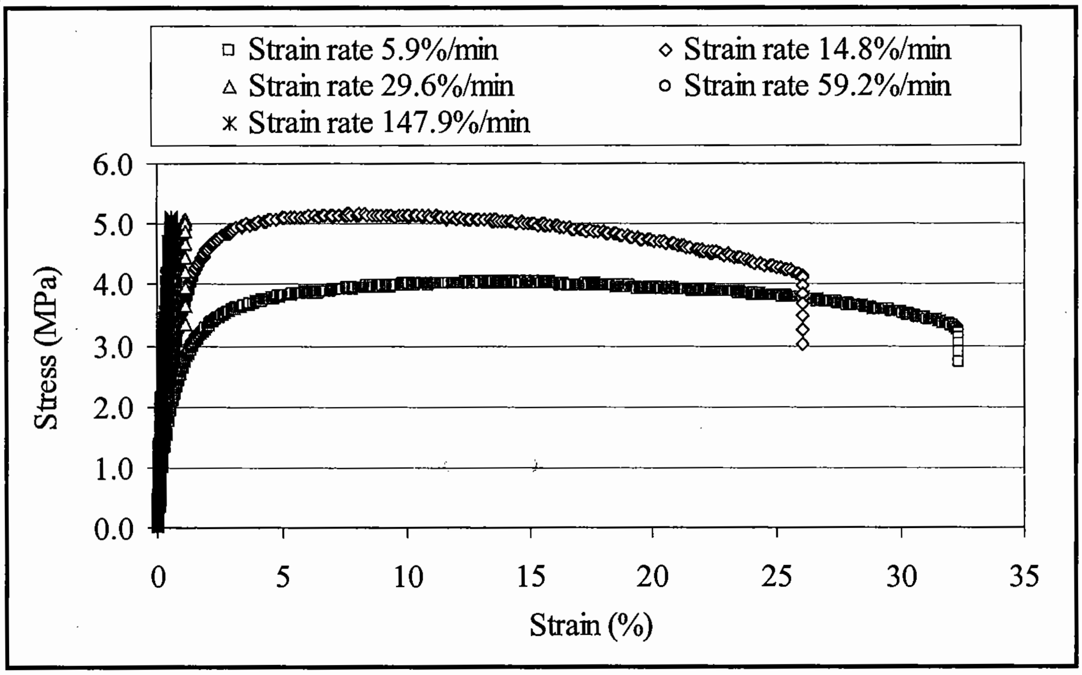

Figure 25 Stress against strain for a mortar containing 50% cement filler tested at -5°C

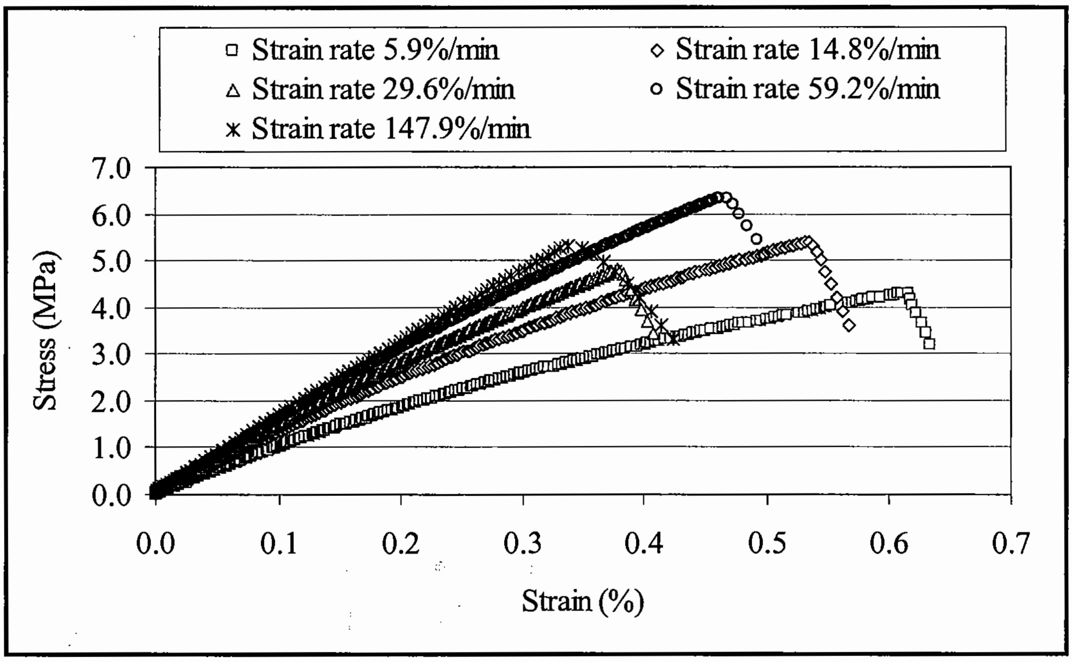

Figure 26 Stress against strain for a mortar containing 50% cement filler tested at -10°C

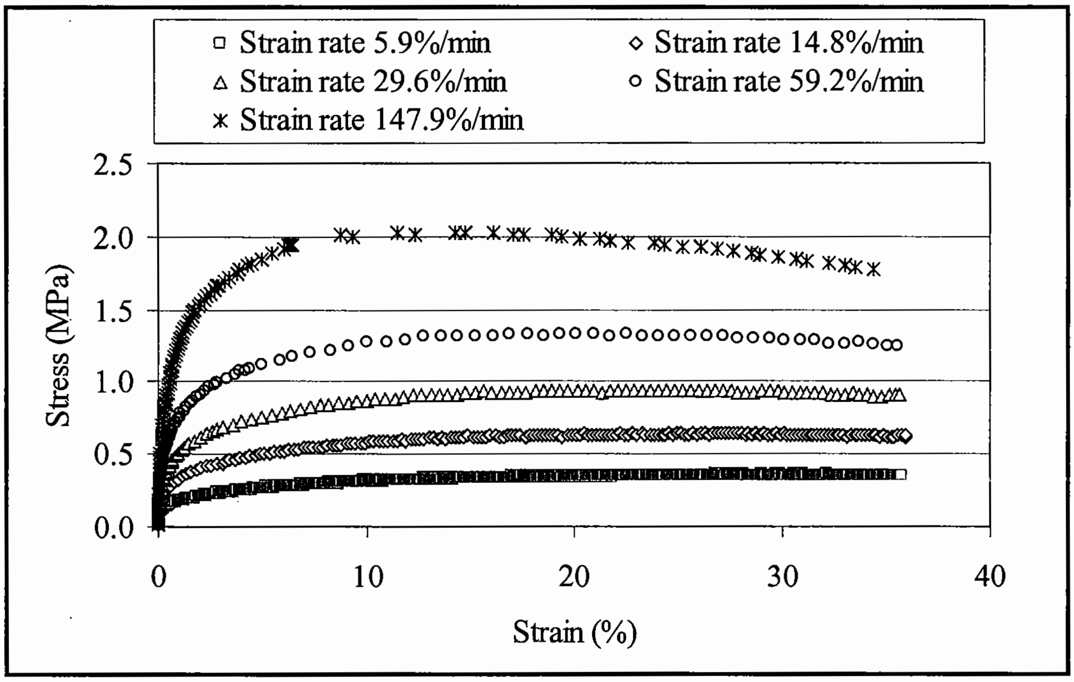

Figure 27 Stress against strain for a mortar containing 50% gritstone filler tested at +10°C

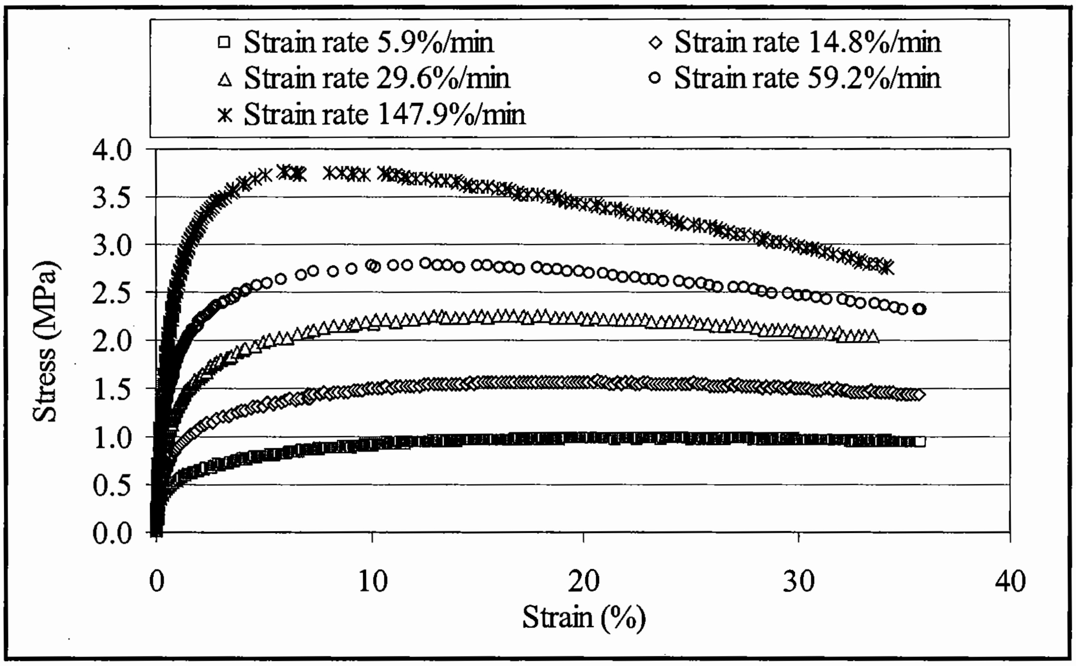

Figure 28 Stress against strain for a mortar containing 50% gritstone filler tested at +5°C

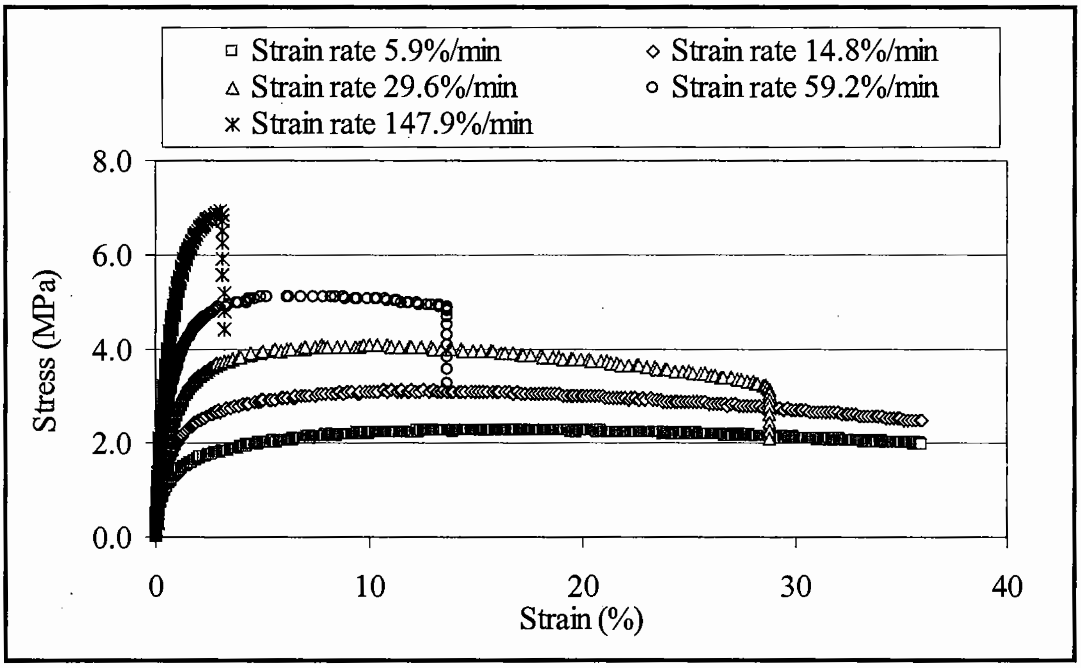

Figure 29 Stress against strain for a mortar containing 50% gritstone filler tested at 0°C

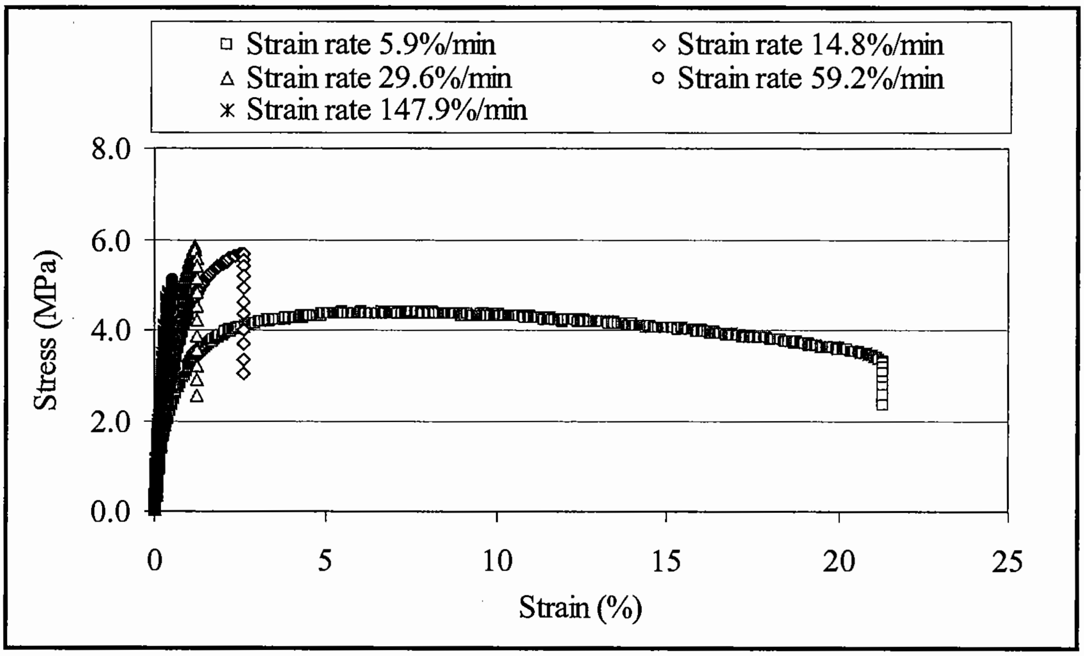

Figure 30 Stress against strain for a mortar containing 50% gritstone filler tested at -5°C

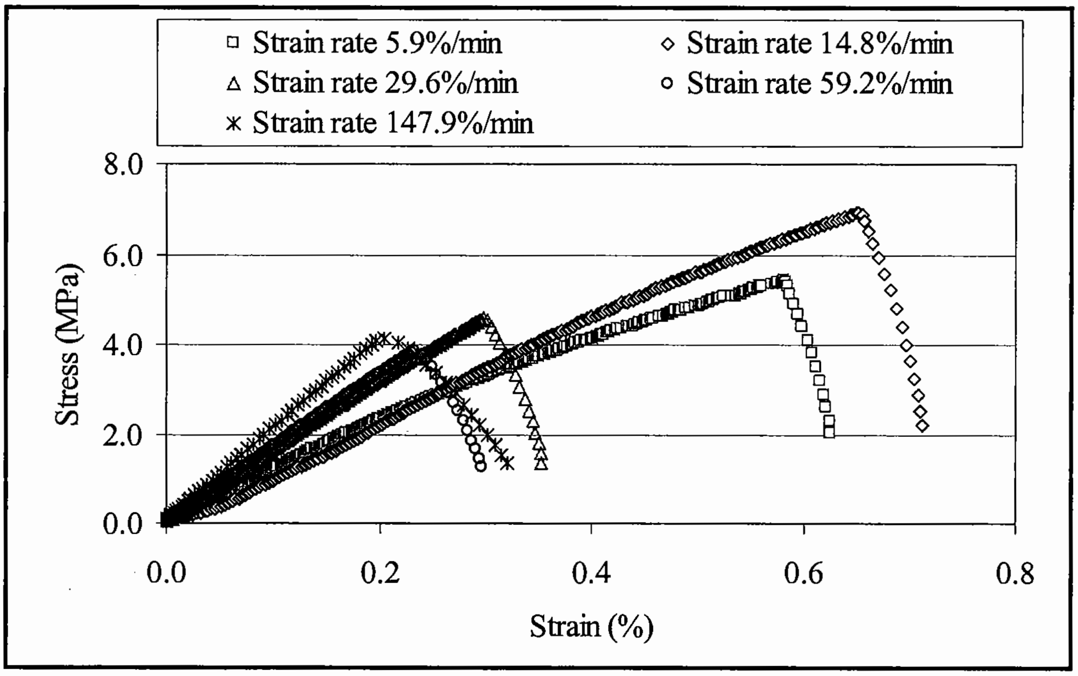

Figure 31 Stress against strain for a mortar containing 50% gritstone filler tested at -10°C

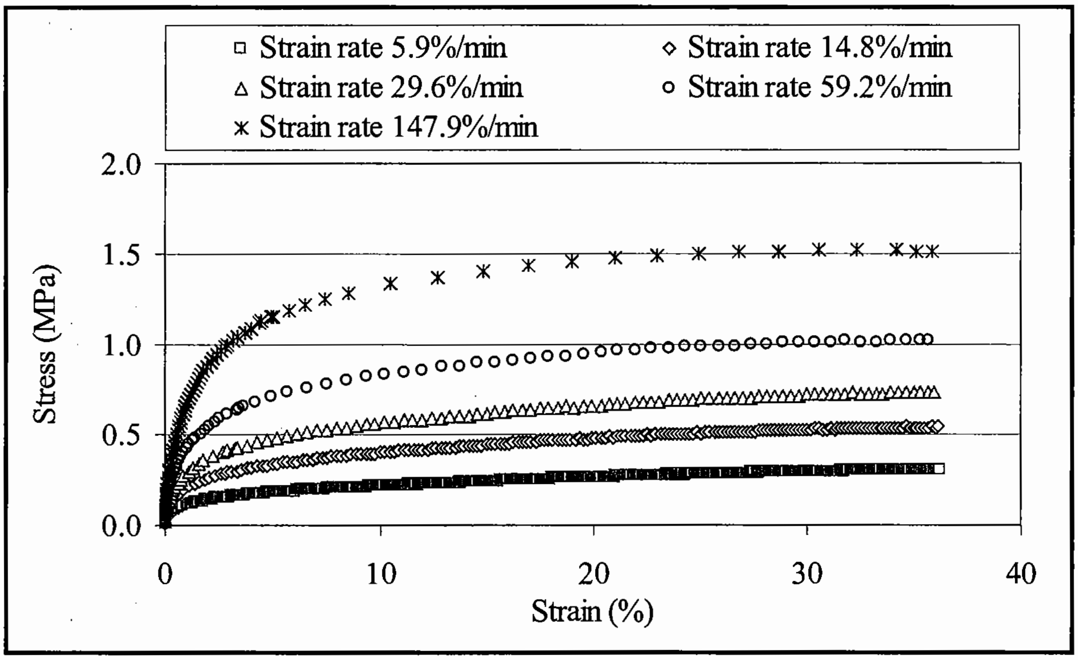

Figure 32 Stress against strain for a mortar containing 15% sewage sludge ash filler tested at +10°C

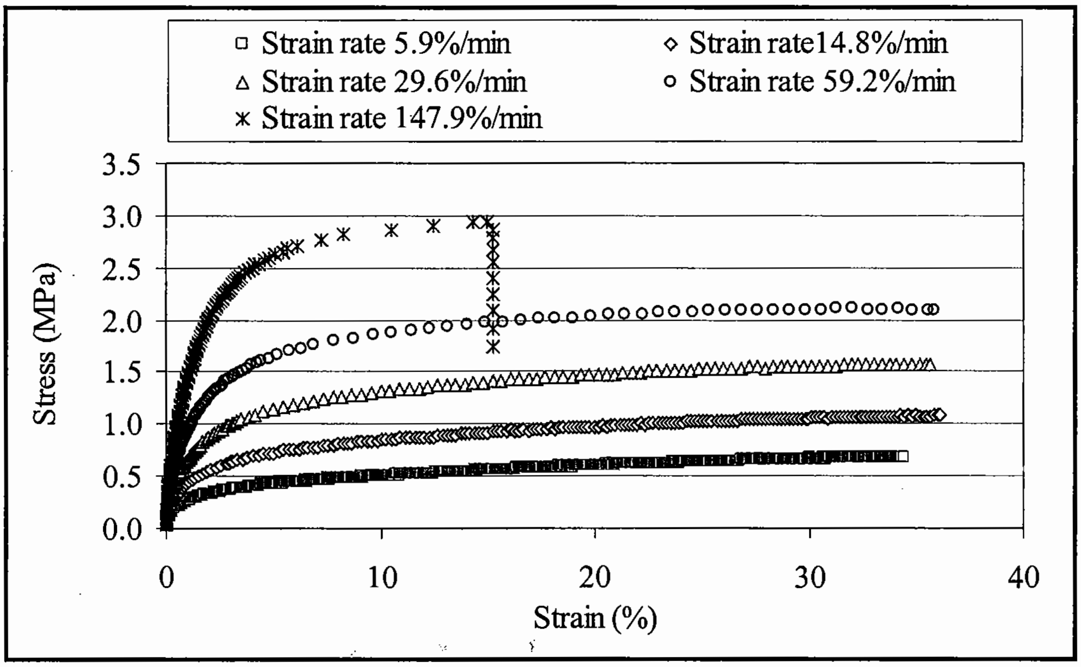

Figure 33 Stress against strain for a mortar containing 15% sewage sludge ash filler tested at +5°C

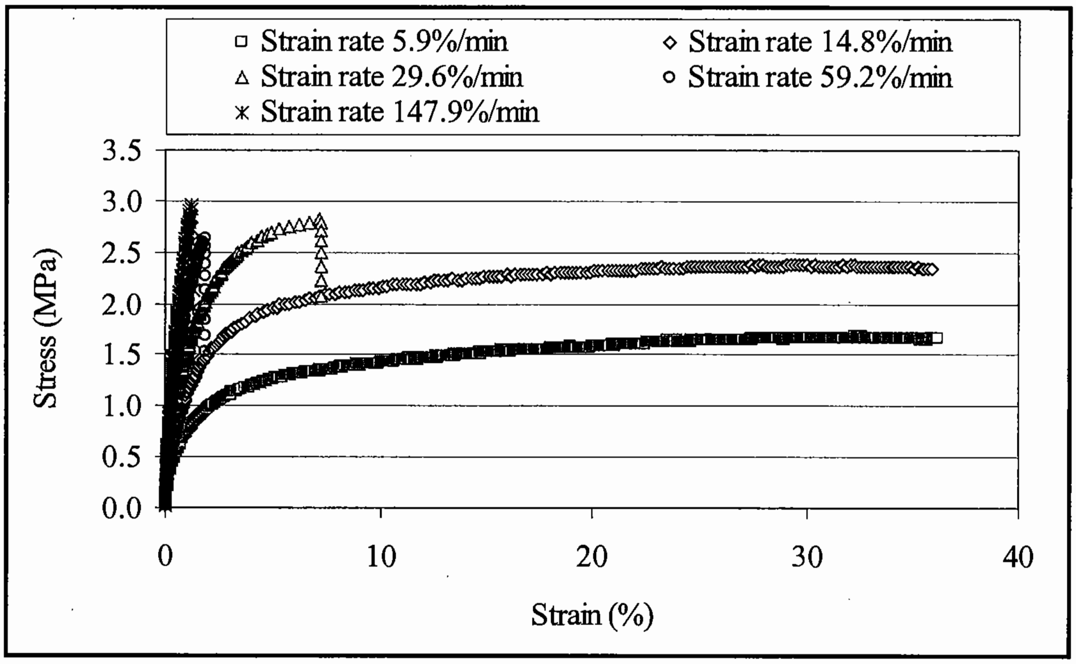

Figure 34 Stress against strain for a mortar containing 15% sewage sludge ash filler tested at 0°C

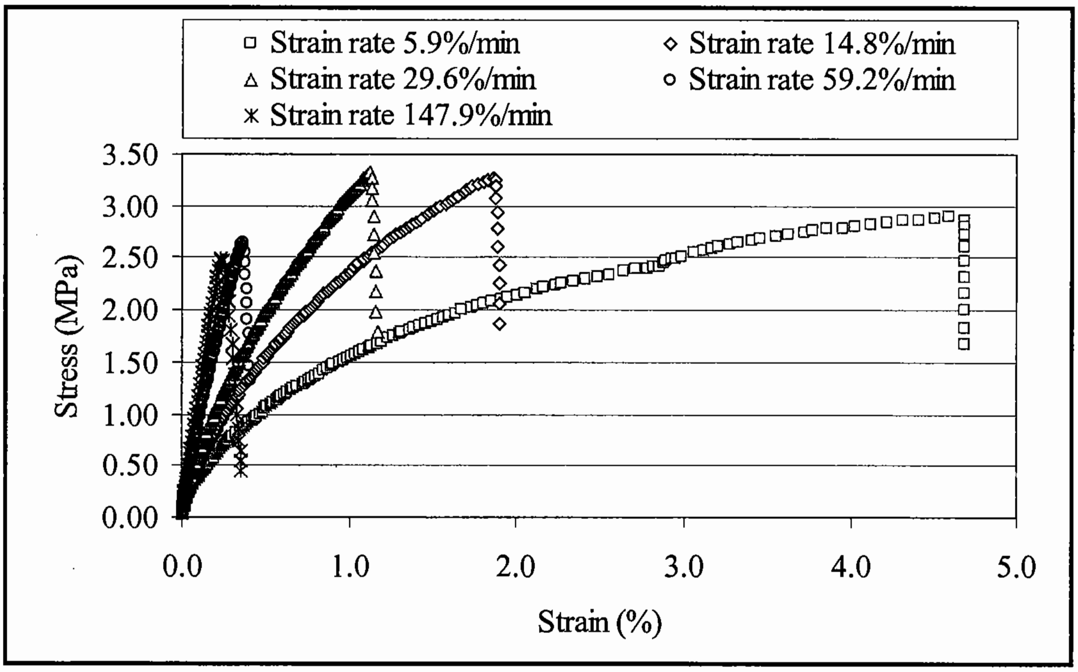

Figure 35 Stress against strain for a mortar containing 15% sewage sludge ash filler tested at -5°C

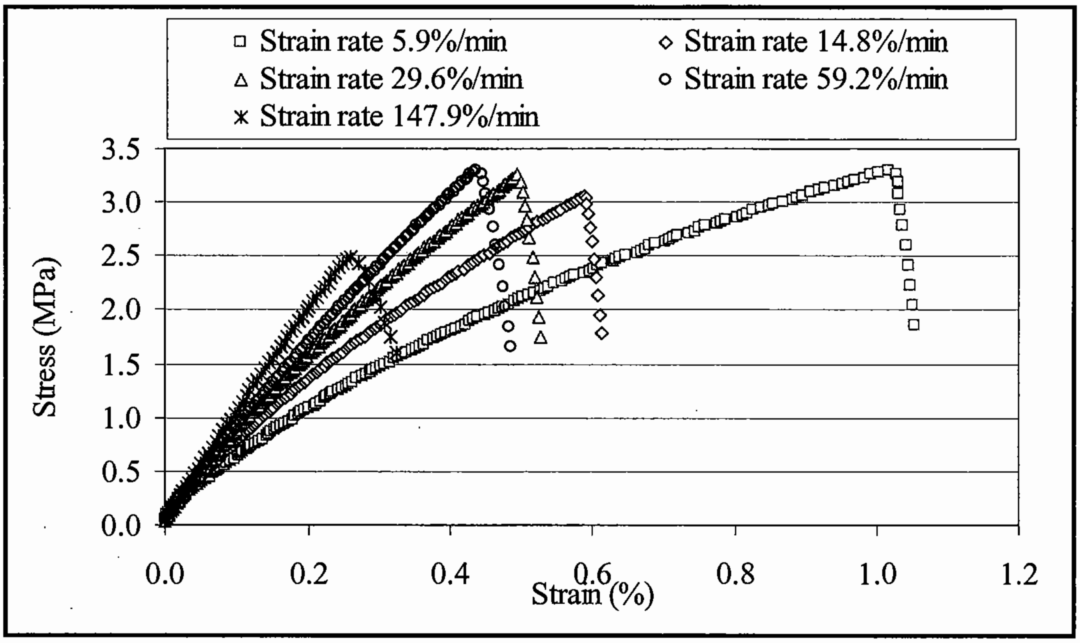

Figure 36 Stress against strain for a mortar containing 15% sewage sludge ash filler tested at -10°C

Table 1 Density of top and bottom section of selected specimen

Sample No.	Density (g/cm^3)	
	Top section	Bottom section
1	1.392	1.410
2	1.229	1.236
3	1.325	1.309
4	1.255	1.251