THE ENGINEERING PROPERTIES AND FATIGUE BEHAVIOR OF ASPHALTIC CONCRETE MADE WITH WASTE THREAD TIRE RUBBER MODIFIED BINDERS

by

Osman Nuri ÇELİK

B.Sc. (Civil), Konya, Turkey, 1984

M.Sc. (Transport), Istanbul, Turkey, 1988

Submitted in accordance with the requirements of the degree of

Doctor of Philosopy

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The candidate confirms that the work submitted is his own and that appropriate credit has been given where reference has been made to the work of others.

to my family

Abstract

Poor performance and premature failure of some bituminous mixtures have led many field engineers to find out new materials. To improve the performance of bitumen and to increase the service life of bituminous mixtures, additives have been incorporated to change the characteristics of the bitumen and bituminous mixtures.

The disposal of scrap tyres, on the other hand, is an increasing environmental problem. In recent years, the most overlooked aspect of rubber modified asphalt is the attention to the ecological problems of disposing of discarded tyres. Moreover the properties of rubbers are in good relevant with the problems in pavement design. To find a usage to the waste rubber in pavement will achieve those problems together.

Many advantageous of incorporating rubber into the pavement have been reported. However, structured studies which deal with the long term performance and durability of asphaltic concrete made with rubber-modified bitumen are not available. Because of the lack of information, a need exists to evaluate the effects of mix ingredients in terms of engineering properties.

This laboratory study examined the effect of shredded vulcanised tyre rubber on the rheology of the rubber bitumen blends and their influence on the engineering properties and fatigue behavior of asphaltic concrete made with crushed limestone, which the gradation of aggregate was selected on the basis of minimum porosity, and ordinary Portland cement as a filler. Rheological measurements were performed using a Carri-Med CSL500 controlled stress rheometer. The Leeds Design Method was used to predict the effect of shredded tyre rubber on the engineering properties of asphaltic concrete. A new porosity measuring apparatus - Leeds Vacuum Porositymeter (LVP)- for the bituminous mixture was introduced. Fatigue behavior was examined using an INSTRON controlled sinusoidal stress fatigue testing machine.

Results of the tests have shown that the addition of rubber imparts the elastic behavior of the binder. This leads an increase on the porosity of the mixture. It is an undesirable result for the bituminous mixtures. An improvement is needed on the compaction method of shredded tyre rubber modified asphaltic concrete.

Acknowledgments

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Finally, his spouse who provided encouragement and support to bring this work to a completion.

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CHAPTER ONE

INTRODUCTION

1.1. General

In most region on the world, road pavements consist of conventional mixtures which perform satisfactorily under relatively low traffic volumes and mild environmental conditions. However, increasing numbers of commercial vehicles with increased axle loads in aggressive climates result in accelerated deterioration and eventual failure of bituminous pavements.

The aim of pavement design is to select the most economic pavement composition, which will provide a satisfactory level of service for the expected traffic. To achieve this, the designer must have sufficient knowledge of the materials, the traffic, the local environment and their interaction to be able to predict the performance of any pavement composition.

Poor performance and premature failure of some bituminous mixtures have led many researchers and engineers to find out new materials. To improve the performance of bitumen and to increase the service life of bituminous mixtures, additives have been incorporated to change the characteristics of the bitumen and bituminous mixtures. The idea of adding natural or synthetic polymers, such as rubber, to traditional binders, to improve the pavement performance dates back to over a century (Thompson, 1964). The earliest references found are British Patents (Austin, 1943; Hancock, 1843) issued in 1843.

Investigations at the Road Research Laboratory (now Transport Research Laboratory) in co-operation with the Natural Rubber Bureau have shown that the addition of small proportion of natural rubber to bitumen had a strong influence on the rheological properties of the binder in a manner likely to result in improved road performance (Thompson and Szatkowski, 1971).

The disposal of scrap tyres is an increasing environmental problem. For example, estimates for the generation of scrap tyres are 1.5×10^6 tones per year in the European Community, 2.5×10^6 tones per year in North America and 0.5×10^6 tones per year in Japan (Roy *et al.*, 1990), and just 5 percent of these are recycled (Dempster, 1979).

Many road authorities are currently evaluating the use of recycled rubber to modify bituminous mixtures. There are also a number of patents some of which are offered commercially, for example a method patented under the trade name Plus Ride™, which is marketed in USA by PaveTech Co.. In this process granulated coarse and fine rubber particles are incorporated into the mix to replace some of the aggregates. The rubber particles are added cold into the mix. The reported advantages of using this paving system are (Tokallou, 1987);

1-high fatigue resistance,

2-high durability,

3-reduced reflective and thermal cracking,

4-improved skid resistance during dry, wet and icy condition,

5-improved ice removal by elastic deformation of the rubber granules under traffic loading,

6-noise reduction,

7-environmental soundness,

8-reduced hydroplaning and water spray and

9-reduced need for salting and sanding.

Another patented method is to modify directly the binder itself before mixing together with aggregates. This process is also offered in USA as Arm-R-Shield (A-R-S) (Tokallou, 1987) by Union Oil Co. (Nielsen and Bagley, 1978) and US Rubber Reclaiming Co.(Huff, 1979).

Rubberized bitumen imparts greater elasticity and a reduced susceptibility to temperature changes to normal bitumen (Szatkowski, 1970). These physical and chemical properties of vulcanised rubber are of far greater technical importance than unvulcanised rubber (Stern, 1967).

However, structured studies which deal with the long term performance and durability of bituminous mixtures made with rubber-modified bitumen are not available. Because of the lack of information, a need exists to evaluate the effects of mix ingredients in terms of engineering properties and long term performance of rubber modified bituminous mixtures.

1.2. Objectives

In this study, shredded tyre tread vulcanised rubber was used to investigate the effect of rubber on the properties of asphaltic concrete. The shredded rubber was prepared by Wellington Rubber Co. Ltd. (Birkett, D.J., 1991, Leeds). Four different rubber gradation were used in the investigation. The gradation of rubber particles are given in Chapter 3 Table 3.1..

The objectives of this investigations were;

1-to study the effect of shredded waste rubber size and amount on the rheological properties of bitumen,

2-to design a continuos particle size distribution of mineral aggregates based on minimum VMA and to study the engineering properties and performance of asphaltic concrete made with this aggregate gradation,

3- to develop a method to measure directly the porosity of bituminous mixture,

4-to investigate the effects of shredded tyre rubber on the engineering properties of asphaltic concrete particularly on permanent deformation during long term loading (creep),

5-to examine the resistance to fatigue cracking of the rubberized asphaltic concrete.

1.3. Study Approach

This research consisted of a laboratory study of the properties of asphaltic concrete as a function of variables such as rubber content, rubber size, mixing temperature and mixing time of rubber-bitumen blend, aggregate gradation and porosity. The influences of these variables were evaluated on measuring the engineering properties and fatigue life of the mixtures. For this purpose it was necessary to undertake the following steps.

Step 1. Evaluation of Use of Rubber in Bituminous Mixtures

Earlier investigations on rubber modified bituminous mixtures were reviewed to identify the potential problems and benefits as well as the usage of material. This step consisted mainly a review of the available literature in order to;

1-identify the effects of rubber particle shape, type and gradation on the properties of rubber modified binder,

2-evaluate the current mix design procedure and guidelines for rubber modified mixtures,

3-evaluate the effect of varying rubber content, rubber gradation and bitumen type,

4-evaluate the rubber-bitumen interaction.

These are presented in Chapter 2.

Step 2. Materials and Procedures

In this step materials and rubber bitumen blend procedure were considered to evaluate the effect of mix variables on the properties of rubber modified asphaltic concrete. It contains in particular, 1-material used and their properties,

2-specimen preparation techniques.

Chapter 3. explain this step.

Step 3. Design of Bituminous Mixture

This step contains the design procedure of the waste shredded rubber modified asphaltic concrete. The Leeds Design Method (LDM) (Cabrera, 1992) was used to determine the Optimum Binder Content (OBC) and evaluate the effect of rubbers on the engineering properties of the shredded rubber modified asphaltic concrete.

Chapter 4. explains this step.

Step 4. Rheology of Waste Shredded Rubber-Bitumen Blends

In this step the rheological properties of rubberized bitumen were determined using current test methods, i.e. specific gravity, penetration, softening point, ductility, dynamic mechanical analysis (oscillatory shear stress) and modified ductility.

The results of this step are given in Chapter 5.

Step 5. Porosity of The Rubberized Asphaltic Concrete

Porosity is one of the most important factor effecting the long term performance and durability of bituminous mixtures. In this step, a method developed to measure directly the porosity of bituminous mixtures is presented. The porosity of the asphaltic concrete is measured by the new method based on vacuum porosimetry. The results were compared against calculated porosities from;

1- porosity calculated using geometry of the samples

2- porosity calculated after weighing the standard test samples in air and in water.

Chapter 6. includes the result of this step.

<u>Step 6. Engineering Properties of Asphaltic Concrete made with Waste Rubber-Bitumen Mixtures</u>

Mix designs were prepared for thirty-four different combination to evaluate the optimum binder content. Standard 101.6 mm ∅ × 63.5 mm height samples were tested for flow, stability, permeability and density. The major mix material variables used in the mix design were as follows,

1-four different rubber gradation; passing 3.35 mm ,2.36 mm, 2.36 mm (fine) and 1.18 mm,

2-four rubber content 2, 5, 10, 20% by weight of bitumen,

3-two different penetration grade bitumen (50 pen. and 100 pen.).

The results of this step are presented in Chapter 7.

<u>Step 7. Creep of Rubberized Asphaltic Concrete</u>

The creep measurement was done on the CANIK UL apparatus (Cabrera and Nikolaides, 1987). to evaluate the creep stiffness of waste shredded rubber modified asphaltic concrete.

The results of this step are given in Chapter 8.

Step 8. Fatigue Behavior of Rubberized Asphaltic Concrete.

The behavior of flexible pavements under repeated loading is of critical concern to highway engineers as well as laboratory researchers. In this investigation, the INSTRON Testing Machine (Model 8033) was used to evaluate the fatigue life of the rubberized asphaltic concrete.

The results of this step are presented in Chapter 9.

Step 9. Summary and Recommendation

This step brings together all the test data and research information to estimate the effects of the mixture variables on mix design and mix properties.

This final chapter summarizes the work done. It presents the advantages, disadvantages, data analysis and recommendation for use of waste shredded rubber modified asphaltic concrete.

The results are given in Chapter 10.

CHAPTER TWO

LITERATURE REVIEW

2.1. Introduction

A number of investigations have been reported for the rubberized bitumen. The essential characteristic of rubber is its property of high elasticity which allows it undergo large deformation from which almost complete, instantaneous recovery is achieved when the load is removed. This property of high elasticity derives from the molecular structure of rubber. Rubber belongs to the class of materials known as polymers and also referred to as an elastomer. The characteristics of an elastomer are (Freakley and Payne, 1978);

1-the molecules are very long and are able to rotate freely about the bonds joining neighboring molecular units,

2-the molecules are joined, either chemically or mechanically, at a number of sites to form a three dimensional network. These joints are termed cross-links,

3-apart from the cross-links, the molecules are able to move freely past one another.

In recent years, the most overlooked aspect of rubber modified bitumen is the attention to the ecological problems of disposing of the large volume of discarded tyres. The properties of rubbers as modifiers in bitumen to impart improved performance to bituminous mixtures used in pavement construction are in of great relevance since the construction of pavements requires large amounts of binder and therefore this is a large source for the useful utilization of discarded tyres. To find an usage to the waste rubber in pavement will achieve double purposes, i.e. safe disposal of rubber tyres and improved pavement performance..

2.2. History of The Rubber Industry

Rubber was one of the first substances to impress the early European explorers of the New World. They had never encountered anything like the resilient balls that were used by the natives of Central and South America for playing games. The balls were made from a dried milky liquid which could be obtained by cutting the bark of certain trees. Samples of this curious gun were taken back to Europe by the Spaniards and Portuguese. However, their discovery had no impact on civilization at that time (Lagrane, 1980).

In 1770, Joseph Priestly discovered that the material could be used to rub out pencil marks and called the name "rubber". Rubber was not widely used on a commercial basis until Charles Goodyear, in 1839, discovered how to "vulcanize" it with sulphur. Vulcanization with sulphur reduced the temperature susceptibility of the rubber. With further compound development, vulcanization made possible the production of items, such as the pneumatic tyre, which consumes more than half of all rubber used world-wide. About 80% of all rubber is used in the motor industry, mainly for tyres (Barron, 1949). The rubber tyre accelerated the development of the automobile and this, in turn, created the necessity for an improved highway system (Lagrane, 1980).

2.3. Properties of Rubber

Several detailed references about the properties of rubber have already been published (Stern, 1967; Treloar, 1975) so that only a brief summary will be given here.

The physical and chemical properties of vulcanised rubber are of far greater technical important than unvulcanised rubber. Probably the most important single property of rubber whether vulcanised or not, is its elasticity.

A typical stress-strain curve for vulcanised rubber is shown in Figure 2.1.. The value of Young Modulus for small extension is 981 kPa compared with about 9.81×10^6 kPa for a hard solid. For rubber, elongation at break is about 500 to 1000 %. This value is seldom for a hard solid over 1 % (Stern, 1967).

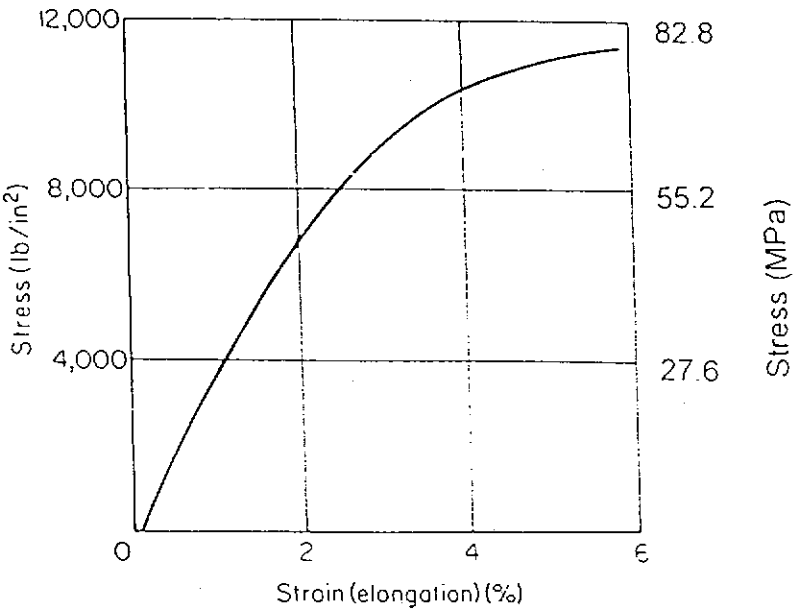


Figure 2.1. A typical stress-strain curve for vulcanised rubber (Stern, 1967).

Another important property of rubber is its long chain molecules with freely rotating links. On deforming rubber, the chain of molecules, which are curled, extend These chains will try to return to the usual position. The maximum possible elongation will therefore depend on the actual length of the chain. Thus, the greater the chain length, the greater should be the elongation before the rubber breaks (Stern, 1967).

Ordinarily rubber begins to soften at about 120 °C and starts to decompose above 170 °C, and as the temperature rises it changes into a thick brown oil with a characteristic odour which is no longer rubbery. Above 300 °C cracking taken place, and breakdown occurs with the formation of numerous decomposition products (Barron, 1949).

2.4. Tyre Composition

Tyre contains vulcanised rubber in addition to the rubberized fabric reinforcing textile cords, steel or fabric belts and steel-wire reinforcing beads. The most commonly used tyre rubber is SBR (Styrene-Butadiene-Copolymer) containing about 25% styrene by weight. Other rubbers used in tyre, manufacture include natural rubber (cis-polyisoprene), synthetic cis-polyisoprene, and cis-polybutadiene. A typical composition for tyre rubber is shown in Table 2.1. (Williams *et al.,* 1990).

The carbon black is used to strengthen the rubber and aids abrasion resistance, and the extender oil is a mixture of aromatic hydrocarbons, which serves to soften the rubber and improve workability. Sulphur is used to cross-link the polymer chains within the rubber and also to harden and prevent excessive deformation at elevated temperatures. The accelerator is typically an organo-sulphur compound, which acts as a catalyst in the vulcanization process. The zinc oxide and stearic acid also act to control the vulcanization process and in addition, enhance the physical properties of the rubber (Williams, *et al.*, 1990).

Table 2.1. Rubber Compounding Composition.

Component	Weight(%)
SBR	62,0
Carbon Block	31,0
Extender Oil	1,9
Zinc Oxide	1,9
Stearic Acid	1,2
Sulphur	1,1
Accelerator	0,7

2.5. Bitumen as a Road Material

Bitumen is a dark, highly viscous to almost solid substance, consisting of hydrocarbons and their derivatives and is completely soluble in carbondisulphide. It is the residue of certain kind of petroleum after the evaporation of their most volatile components (Pfeiffer, 1950).

If the residue has been formed from petroleum under geological influences, the bitumen is found as such in the earth's crust in the form of "pitch lakes" (e.g. Trinidad), the bitumen in this form is called natural bitumen (Pfeiffer, 1950).

Bitumen was imported into UK on a small scale early in this century. In the twenties bitumen consumption was about 200000 tones per annum. The demand increased rapidly, reaching 1 million tones per annum by 1960 and with the start of the motorway building program the demand for bitumen soared to 2 million tones per annum by 1970, reaching a peak of 2.4 million tones in 1973 (Shell, 1990).

2.5.1. Bitumen specifications and quality

In the UK, bitumens are manufactured to British Standard, BS 3690. Part 1 (1989) covers bitumen for road purposes, Part 2 (1989) covers bitumens for industrial uses and Part 3 (1989) covers blends of bitumen with coal tar and with lake bitumen or pitch.

There are five types of bitumen, which are of common use in industry. They are characterised by a combination of penetration, softening point and viscosity, these are namely;

1-penetration grades,

2-oxidised grades,

3-hard grades,

4-cutback grades,

5-bitumen emulsions.

Penetration grade bitumens are specified by the penetration and softening point tests, but are designated by penetration only, e.g. 100 pen. bitumen has a penetration of 100 ± 20 and a softening point of 46 ± 5 °C. The grades range from 15 pen. through to 450 pen., with corresponding softening point ranges for each grade. The majority of penetration grade bitumens are used in road construction.

Oxidized bitumens are used almost entirely for industrial applications, and are specified and designated by both softening point and penetration test, e.g. 85/40 is an oxidised grade bitumen with a softening point 85 ± 5 °C and a penetration 40 ± 5 (1/10)mm..

Hard bitumens are also used entirely for industrial applications, they are also specified by both softening point and penetration test but are designated by a softening point range only and a prefix H, H80/90 is a hard grade bitumen with a softening point between 80 °C and 90 °C.

Cutback bitumens are manufactured by blending either 100 pen. or 200 pen. bitumen with kerosene to comply with a viscosity specification. In the UK cutback bitumens are specified and designated by the flow time trough a standard tar viscometer (STV). Three grades are available, 50 seconds, 100 seconds, and 200 seconds. The majority of cutback bitumen is used in surface dressing but a significant amount is also used for the manufacture of both standard and deferred set macadams (Shell, 1990).

Bitumen emulsions are classified in BS 434: Part 1 (1984) by a three part code and in BS 434: Part 2 (1884) these codes are explained. The first part of the code, either A or K, indicates anionic or cationic emulsion. The second, 1 to 4, indicates the breaking rate or stability, the higher the number the greater the stability. The third part of the code, 40 to 70, indicates the bitumen content of the emulsion. For example; K1-70 is a kationic emulsion, rapid setting with a bitumen content of 70%. These specifications (BS 434, 1984) principally specify the bitumen emulsions in terms of viscosity and bitumen content.

2.5.2. Rheology

Rheology is the science of the deformation and flow of matter (Van Wazer, 1963) and bitumen is the most unique material for the rheologist. Bitumen rheology is the study of stress-strain patterns of the material in relation to time. The following patterns are considered (Barth, 1962):

1-degree of elastic recovery after a series of deformations,

2-stress or strain relaxation after deformations,

3-internal strain relaxation with time, after a deformation followed by a partial recovery,

4-change of strain with time under constant stress

5-recovery of strain after release of stress.

Bitumens are distinguished as regard their rheological properties in two respect, namely their character and their hardness (Pfeiffer, 1950). Differences in character mainly consist in deviations from Newtonian flow, and differences in hardness can be indicated by the value of the viscosity. The character may vary when bitumens are produced from the same petroleum by different methods, or by the same process from crudes of different origins.

2.5.2.1. Flow

An ideal viscous body can not sustain stress for long since this is relieved by flow, with the rate of flow being a function of stress (Shell, 1990). Flow, brittleness and viscosity are interrelated on bitumen and temperature greatly affects these properties.

2.5.2.1.1. Newtonian Flow

The Newtonian model of flow is shown in Figure 2.2.. It consists of two parallel planes A and B, the space between them being filled with a liquid. A tangential shearing stress F is applied at *a*.

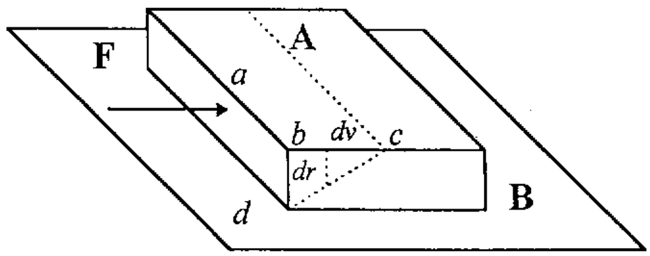


Figure 2.2. The Newtonian model of flow (Barth, 1962; Van Wazer, 1963).

The A plane then moves with respect to B, carrying with it, the innumerable parallel planes of liquid existing between A and B. The top plane A moves the furthest with respect to the plane B, which remains stationary. When point *b* reaches to point *c*, the straight line *bc* represents the distance travelled by plane A in one second, and its length gives the velocity of plane A. Then *bc* divided by *bd* becomes the rate of shear. This is usually written *dv/dr*, where v and r are the velocity and distance between the two planes respectively. This equation is often written as;

$$\eta = \frac{F}{S} \tag{2.1.}$$

in poises, where, η the viscosity, is the ratio of shearing stress F to the rate of shear, S; F is the shearing stress in dynes/cm, and S is the rate of shear in reciprocal seconds.

2.5.2.1.2. Non-Newtonian Flow

For a Newtonian fluid in laminar flow, the shear stress is directly proportional to the shear rate. Viscosity η depends only on temperature and pressure and is independent of shear rate. However for several classes of fluids this coefficient of dynamic viscosity is found to vary with changing shear rates. Such materials are called non-Newtonian fluids (Van Wazer, 1963).

Typical flow curves for both Newtonian and non-Newtonian fluids are shown in Figure 2.3.. Line A represents a Newtonian liquid, the coefficient of viscosity η is constant. When the rate of shear increases at a greater rate than the shear stress, curve B, the fluid is called pseudo-plastic liquid. A material exhibiting a flow curve shaped like curve C, where the rate of shear increases at a lower rate than the shear stress, is called a dilatant fluid.

Another important phenomenon connected with flow is the existence of a yield value. Certain materials which flow readily under sufficient shear stress will not flow at all if the shear stress is reduced below a certain amount called the yield value. A fluid exhibiting this type of behavior is called Bingham (Van Wazer, 1963) body, see curves D, E and F in Figure 2.3..

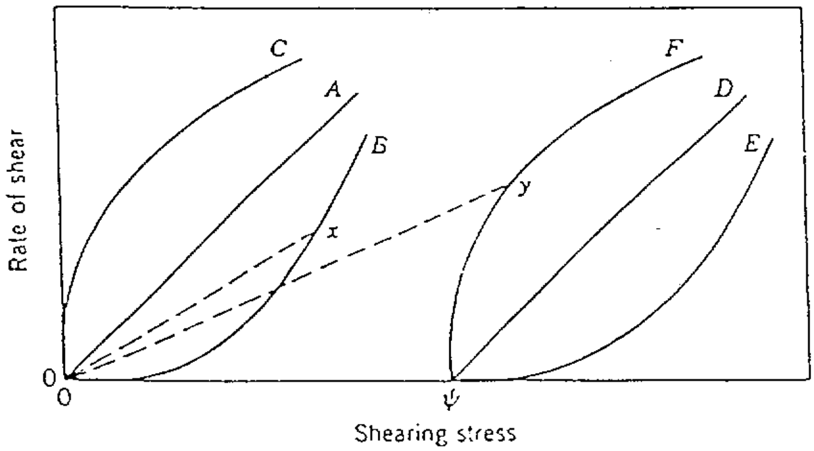


Figure 2.3. Flow curves for a number of fluids with differing rheological properties (Van Wazer, 1963).

2.5.2.2. Viscosity

The apparent viscosity η which is the ratio of total shearing stress to total rate of shear at a given value of, say, rate of shear, is exemplified by the two dotted lines in Figure 2.3.. The concept of a Newtonian fluid has a very important bearing on the measurement of viscosity. Because in a Newtonian fluid, viscosity is independent of the shear rate, instruments employing only one rate of shear can be used, i.e. the so called single point viscometers. However in the case of non-Newtonian fluids, the use of single point viscometer can lead to misleading results.

2.5.2.3. Penetration

The penetration is the normal criterion of consistency and has a wide measuring range. The test method was published in 1889 (Pfeiffer, 1950), improved afterwards by Dow

(1898) and has been standardized by the British Standard Institution (BS 2000 Part 49, 1983).

In this method, a standardized needle is allowed to penetrate under standardized conditions of time (5 seconds), temperature (usually 25 °C) and load (100 gram) in a thin filled with bitumen, and the depth of penetration is measured in 1/10 mm. The determination of the penetration is therefore substantially a determination of the resistance to deformation and is closely related to the viscosity.

Carre and Laurent (1963) have given the relationship between penetration and viscosity for the bitumen used in road construction at given temperature. These equations are;

for the bitumen above 60 penetration;

$$\log P = -0.465 \times \log \eta + 4.725 \qquad (2.2.)$$

or,

$$P = 5.305 \times 10^{4} \times \eta^{-0.4651} \qquad (2.3.)$$

for the bitumen below 60 penetration;

$$\log P = -0.385 \times \log \eta + 4.228 \qquad (2.4.)$$

or,

$$P = 1.69 \times 10^{4} \times \eta^{-0.385} \qquad (2.5.)$$

in viscosity as the function of penetration;

for the bitumen above 60 penetration;

$$\eta = 1.45 \times 10^{10} \times P^{-2.15} \qquad (2.6.)$$

for the bitumen below 60 penetration;

$$\eta = 9.50\times10^{10}\times P^{-2.60} \qquad (2.7.)$$

where;

P, penetration of binder at given temperature,

η, viscosity of binder at given temperature,

2.5.2.4. Softening point

Bitumen has no real melting point. When heated it gradually become softer (Pfeiffer, 1950). To determine the softening point of bitumen the ring and ball test (BS 2000 Part 58, 1983) is an important indicator and is used in UK and elsewhere.

In this test a steel ball (3.5 gram) is placed on a sample of bitumen containing a brass ring; this is suspended in a water or glycerol bath. Water is used for bitumens with a softening point of 80 °C or below, and glycerol is used for softening points greater than 80 °C.

At the beginning of test the bath temperature is 5 °C. The bath temperature is raised 5 °C per minute, the bitumen softens and eventually deforms slowly with the ball through the ring. At the moment the bitumen and steel ball touch a base plate, which is 25 mm. below the ring, the temperature of the water is recorded. The test is performed twice and mean of the two measured temperatures is reported to the nearest 0.2 °C for a penetration grade bitumen and 0.5 °C for an oxidized bitumen. If the difference between two results exceeds 1.0 °C the test must be repeated (Shell, 1990).

2.6. Rubberized Bitumen

The use of bitumen-rubber materials has been increasing as they have proven useful functional applications in various pavement maintenance including binder for hot rolled asphalt (Stephens and Mokrzewski, 1974; Oliver and Dickinson, 1977; Oliver, 1976), low module Stress Absorbing Membranes (SAM's) (Olsen, 1973), Stress Absorbing Membrane Interlayers (SAMI's) (Morris and McDonald, 1976; Huffman, 1980; Schnormeier, 1980; Morris, 1989; Doyt, 1989) and in airport pavements (Hoyt *et al.*, 1988). A wide range of advantages of rubber-modified bitumen have been suggested (Beaty, 1992). These include;

1- to improve low temperature ductility and hence the resistance of bituminous mixtures to brittle fracture,

2- to improve the stiffness of the bituminous mixture at high temperatures so as to reduce permanent deformation and rutting under traffic loading,

3- to improve the adhesion of the binder to the aggregate and thus improve resistance to stripping,

4- to reduce the temperature susceptibility of the binder i.e. to decrease the loss of viscosity at high temperatures and hence to reduce bleeding,

5- to reduce or prevent reflection cracking when the material is laid over a cracked pavement structure,

6- to improve the fatigue resistance of bituminous mixtures, 7- to increase the strength of the bituminous mixtures,

8- to improve the impact resistance of bituminous mixtures,

9- to improve resilience and elastic recovery,

10- to find a usage for scrap rubber from vehicle tyres.

However, structured studies which deal with the long term performance and durability of bituminous mixtures made with rubber-modified bitumen are not available. Because of the lack of information, a need exist to evaluate the effects of mix ingredients in terms of engineering properties.

In spite of the impressive number of benefits to be gained by the use of rubberized bitumen, its acceptance by the road construction industry was rather slow for a long time. This can be attributed to the following (Beaty, 1992);

1-in the majority of road applications traditional bituminous mixtures have given satisfactory service,

2-rubberized bitumen is more costly and its use often required additional processing and modification to traditional handling and laying techniques.

2.6.1. Rubber Bitumen Blend Preparation

The form of rubber and the final use of the blend influences the method of addition and dispersion . The following methods have been used in the past.(Patrick, 1983).

1-Addition into the bitumen: The agitation during dispersion of rubber powder into hot bitumen is the main problem. However the addition of latex into hot bitumen is more difficult, due to the danger of foaming as water evaporates. It is normally recommended (Road Note 36, 1967) that the bitumen be heated to at least 150 °C.

2-Master Batch: This system is traditionally recommended for cutbacks, but it is currently used in the marketing of thermoplastic rubber.

In the case of cutbacks using latex, the total kerosene amount required for the blend is mixed with the same quantity of bitumen. The temperature of this blend is controlled so that the viscosity is such that a vortex will form when mixed with a propeller stirrer. Then latex is added slowly into the binder. The temperature is then slowly raised and the water starts to evaporate at about 95 °C. The mixture will remain at about 100 °C until no water remains, at which point the temperature starts to rise rapidly. The final mix, The "master batch", is therefore a concentrate of rubber, bitumen and kerosene.

This master batch then can be blended with the rest of bitumen required for the job, taking care that there is sufficient mixing to ensure adequate dispersion of the rubber through the blend.

3-Direct into the Pugmill: Addition of scrap rubber directly into the pugmill is being used in United States (Allen, 1990). In Canada, road trials (George, 1978) have been performed with scrap rubber which has been pre-blended with bitumen for 30 minutes at 200 °C, an added directly into the pugmill. The result has shown that, when constructed properly, the rubber modified material can perform as well as a conventional mixture (Allen, 1990).

When using latex in hot mixed asphalt the rubber bitumen can be mixed and sprayed directly into the pugmill. Alternatively, a separate spray bar for the latex can be installed (Patrick, 1983). Tests performed in Italy (Rampichini, 1971) indicated that when spraying the latex separately from the bitumen best results were obtained when the latex spray started 3 to 4 seconds after bitumen. This practice of initially coating the stones by bitumen before addition of the latex is also recommended by Thompson (1969).

4-In Place: A different approach to the problem of incorporating rubber into the mixture is reported by Rostler *et al.*(1970), where a special rubber emulsion is sprayed on to the completed pavement. This emulsion carries rubber into the voids of the mix, where it is deposited on, and absorbed by, the bitumen.

Laboratory trials carried out in New Zealand, (Williamson and Patrick, 1971) indicated that in order for the emulsion to penetrate the bitumen, the void content of the asphalt needed to be greater than 8%. Most of the asphaltic concrete used in road construction would be below this figure, thus adequate penetration would not be obtained.

5-Emulsion: As rubber latex is an emulsion of rubber particles in water, it can be blended with an appropriate bitumen emulsion. Natural rubber latex has been used in New Zealand up to a concentration of 20% of an anionic emulsion. Although phase separation occurs during storage, the components are easily remixed (Patrick, 1983).

2.6.1.1. Rubber by Chemical Type

There is a wide range of materials available that can be classified as rubber (Patrick, 1983). Those that have been commonly used as an additive to bitumen are listed below;

1-Natural rubber: This is the extract from rubber trees. It is chemically a polyisoprene, and is processed into various forms.

2-Neoprene: This was the first synthetic substitute for natural rubber and chemically a polyisoprene, and is processed into various forms.

3-Styrene-Butadiene (SBR): This is a random copolymer of styrene and butadiene. It comes in many types, depending on the method of polymerization, ratio of styrene to butadiene units, and presence of additives.

4-Styrene-Butadiene-Styrene (SBS): This is a block copolymer which behaves as a liner polymer at high temperatures but reverts to a vulcanised rubber form at ambient temperatures. Rubbers of this type are known as thermo-plastic rubbers.

5-Vulcanised rubber: For use in bitumen this is normally vulcanised natural rubber. It is formed by heating a mixture of natural rubber with sulphur, sulphur compounds or other chemicals, to produce cross-linking of the rubber molecules. The degree of cross-linking "vulcanization" can be controlled.

6-Reclaimed rubber: This is previously used rubber which has been ground and then undergone some processing. The processing depolymerises the rubber.

7-Scrap rubber: This is often called waste rubber or crumb rubber. It is ground or pulverized rubber that has been screened into specified size fractions. The main source is normally vehicle tyres. Passenger car tyre treads are usually SBR, while more heavily stressed tyres are made from natural rubber.

2.6.1.2. Rubber by Physical Form

Rubber has been dispersed into bitumen in various physical forms (Patrick, 1983);

1-Powders: Fine and coarse rubber powders are available. The majority of work used vulcanised or lightly vulcanised rubbers. Unvulcanised rubber is sticky and can only be easily handled when mixed with a mineral carrier, such as barite (Winter, 1956).

Scrap rubber is comparatively coarser powder. The most commonly used material in retained between 1.18 mm and 0.60 mm screens.

2-Latex: This is, in effect, an emulsion of the rubber in water. Natural rubber obtained from the trees is in a latex form, which is then concentrated to between 60 and 70% rubber.

3 -Liquid: Natural rubber and some synthetic rubbers can be taken up by a solvent and added to bitumen in this form.

2.6.1.3. Rubber Concentration

As the cost of rubber is so much greater than that of bitumen the minimum quantity to affect an "improvement" should be used. Consideration of the compatibility of the rubber-bitumen blends also limits the quantity of rubber that can be dispersed.

In laboratory trials, rubber concentration of from 0.5 to 10% (by weight) of the bitumen have been used. For field applications the content of rubber has been from 0.5 to 6% of bitumen. Low (0.5 to 1.0%) rubber contents have normally been used for chip seals, although 3 to 4% has been used in Australia. The higher rubber contents (2 to 6%) have normally been used in asphaltic concrete (Patrick, 1983).

Chehovits *et al.* (1982) described the bitumen-rubber blends as a concentration of rubber ratio in two forms;

1-bitumen-rubber containing a higher amount of granulated rubber (15 to 25% by weight), which was originally developed by McDonald (1965),

2-rubberized bitumen (1 to 3 % by weight).

A special case is the scrap rubber of relatively large particle size (4.2 mm to 6.35 mm) in bituminous mixture, with rubber concentration of 3 to 4% by weight which were originally developed in late 1960 in Sweden (Tokallou and Hicks, 1988).

2.6.2. Effect of Temperature

Temperature has two effects on swelling of the rubber particles in bitumen (Green and Tolonen, 1977).

a-as the temperature increases, the rate of swelling greatly increases,

b- as the temperature increases, the extent of swelling decreases.

2.6.3. Effect of Rubber in Bitumen

As mentioned earlier rubber can be incorporated into the bituminous mixture in many different ways. The most commonly used way is the addition of rubber into the bitumen before mixing with aggregate.

2.6.3.1. Rubber as the powder form in bitumen

Piggot *et al.* (1977) used reclaimed rubber from scrap tyres for their tests this was fine vulcanised rubber passing 0.6 mm and much coarser unvulcanised material. Vulcanised rubber showed a large effect on the viscosity of the bitumen under condition chosen for minimal interaction. Figure 2.4. shows the results obtained when mixing was carried out at 110 °C for 10 minutes. The viscosity, measured at 95° C, was increased by a factor of more than 20% when 30% vulcanised rubber was added to the mixture. In contrast, the unvulcanised rubber showed only a very small effect. Very little interaction between the rubber and bitumen was observed at 200 °C. Figure 2.5. shows the effect of heating bitumen-rubber mixtures for periods of up to three days on the viscosity. The vulcanised rubber had the larger effect; probably due to its smaller particle size.

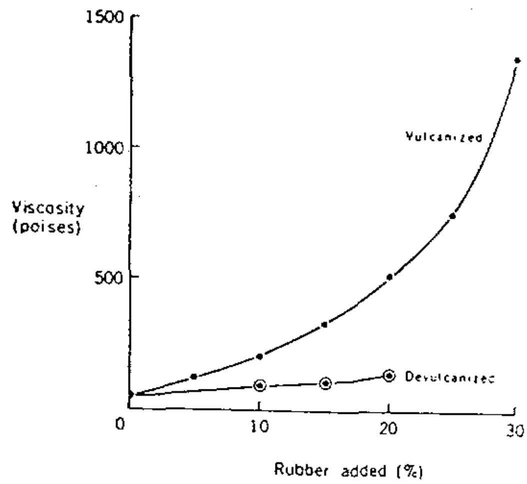


Figure 2.4. Effect of adding ground vulcanised rubber and screened unvulcanised reclaimed rubber on the viscosity of bitumen (Piggot *et al.*, 1977).

In the same investigation, results for void content, voids in mineral aggregate, Marshall flow and Marshall stability were obtained for different aggregates. The void content was greatly increased by the rubber and to keep the void content within the acceptable range (2-4 %) required a higher bitumen content. When the rubber was preheated with bitumen for 30 minutes at 200 °C the void content was even higher.

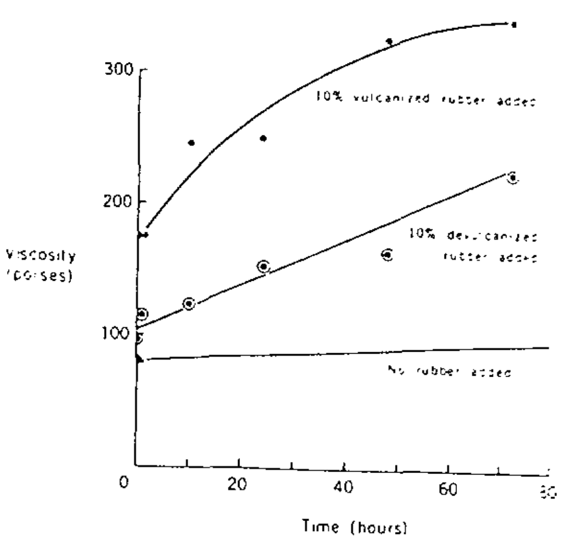


Figure 2.5. Variation on viscosity with varying mixing time of rubber-bitumen mixtures (Piggot *et al.*, 1977).

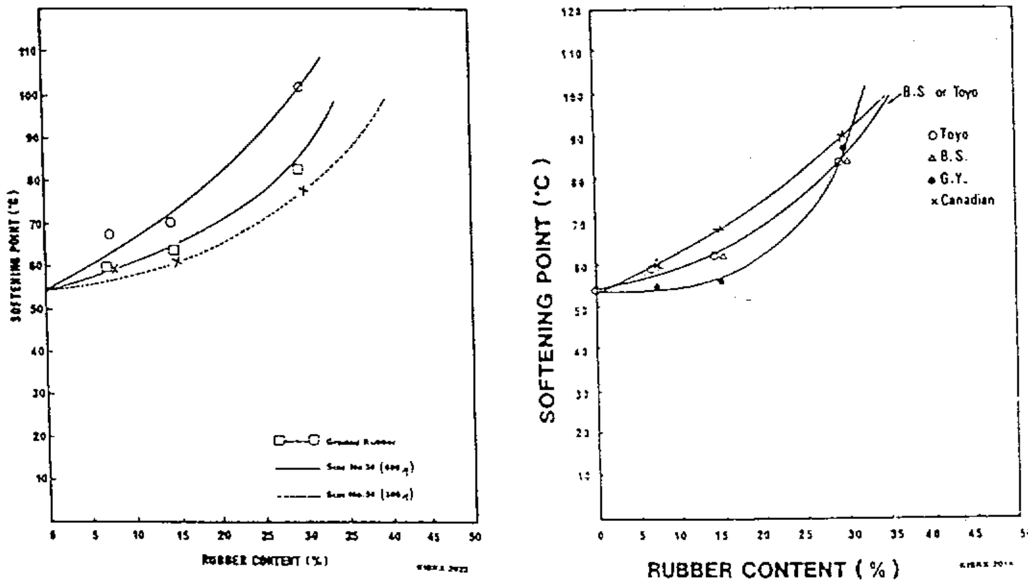


Figure 2.6. The effect of the rubber concentration and rubber particle size on the softening point of rubber- bitumen blends (Lalwani *et al.*, 1982).

Lalwani *et al.* (1982) used truck tyres from three different manufacturers and automobile tyres in bitumen with concentration of 7.5, 15 and 30% .They investigated the rheological properties of bitumen-rubber blends and Marshall stabilities of rubberized asphalt.

Figure 2.6. shows the effect of the rubber concentration (Figure 2.6.a.) and the effect of the rubber particle size (Figure 2.6.b.) on the softening point of the rubber-bitumen blends.

In the same study ground rubber was mixed with bitumen in a closed one litre capacity metal container. Heat was applied via a hot plate controlled by an electronic temperature control device. The bitumen was stirred continuously to prevent local overheating. The ground rubber was added slowly, keeping the temperature of the contents within 5 percent of the desired setting. One hour heating after the addition of rubber was allowed in order to obtain good dispersion.

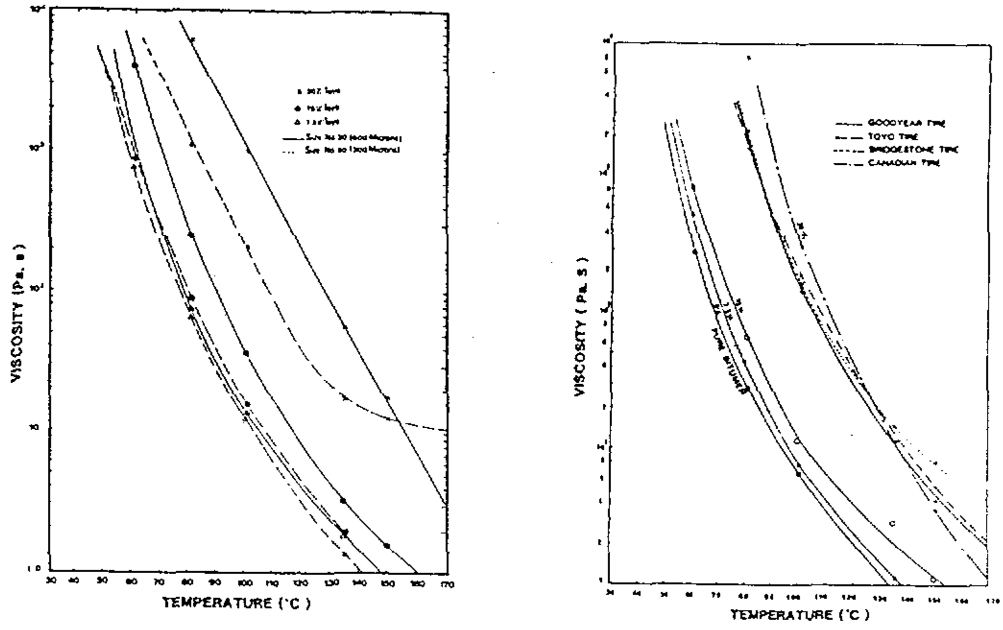


Figure. 2.7. The effect of the rubber concentration and particle size on the viscosity of rubber-bitumen blend (Lalwani *et al.*, 1982).

Figure 2.7.a and b show the effect of the rubber concentration and rubber particle size on the viscosity of the rubber-bitumen blends.

Marshall stabilities in excess of 17.7 kN. at 7% binder content and 4.5% porosity were obtained by using a 30% reclaimed rubber-bitumen blend and a dense aggregate gradation. In comparison, the conventional bituminous mixtures lacking rubber shown stabilities in the order of only 13.2 kN. at approximately 4.9% binder content and 3.4% porosity.

Chehovits *et al.* (1982) studied rubber from two different production process, ambient grind thread peel crumb and ambient grind high natural rubber contents unvulcanised crumb. Each type of rubber was studied at three different particle size distributions. These particles were variable between 2.36 mm-0.075 mm.

To prepare the rubber-bitumen blends they heated the bitumen up to 191 °C in the mixer with the stirring mechanism in operation at 200 to 300 rpm.. The mixer speed was increased to 500 rpm. after the bitumen has reached 191 °C. The required amount of rubber was introduced into the bitumen as rapidly as possible while maintaining the mixing speed at 500 rpm.. The temperature of the mixture dropped slightly due to rubber addition, thus activating the heating mechanism which was restored the 191 °C temperature. After the temperature has returned to 191 °C mixing at 500 rpm. was continued for one hour while maintaining 191 °C. After completion of mixing the rubber-bitumen blend was separated into about 250 cm^3 sealed tins and stored at -18 °C until tested. Material used for testing was reheated only once after mixing.

The creep-strain recovery test procedure was used on the sliding plate viscometer to evaluate the viscoelastic and viscous properties of the rubber-bitumen blends. The model used to interpret test data is a combination of purely viscous response which results in permanent deformation and fully recoverable viscoelastic response and is similar to that proposed by Shweyer and Burns (1978). A conceptional mechanical model (Figure 2.8.) is a dashpot element in series with a second element consisting of a spring and dashpot in parallel.

The creep-strain recovery test consist of four segments each of which was 1/2 hour in duration. The first test segment to measure creep was started by placing a 500 gram weight (resulting in a 9807 Pa. shear stress) on the sliding plate microviscometer and recorded resultant displacement at various time intervals. Displacement was recorded at ten seconds intervals during the first minute, 20 seconds intervals during the second minute, 30 seconds intervals during the third minute, one minute interval during the 7 minutes at two minutes intervals until 28 minutes had elapsed, and then at 29 and 30 minutes.

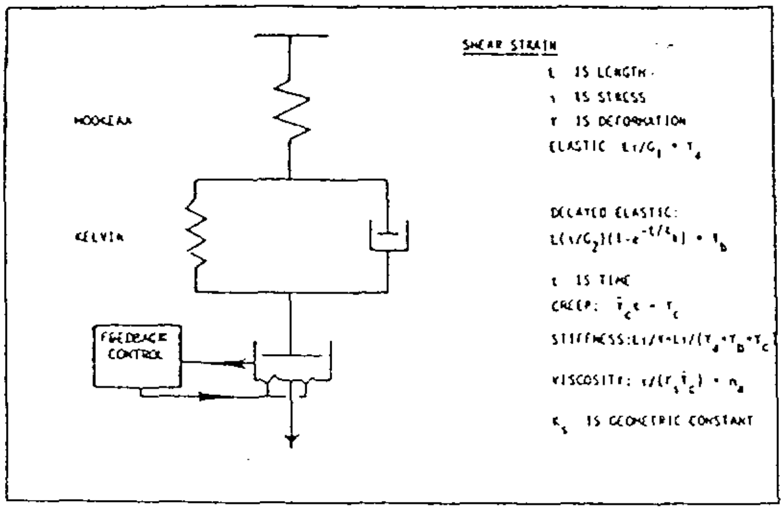


Figure 2.8. Burns-Schweyer model (Scheweyer *et al.*, 1978).

The second segment of the test procedure to measure strain recovery was started immediately after completion of the 30 minutes creep period by removing the 500 gram load from the apparatus. Displacement data were recorded at the same intervals as for the creep cycle of the test.

The third and fourth segments of the test procedure were the repeats of the first and second. After completion of the fourth segment, the sample was allowed for full recovery of elastic strain for 20 hours. A schematic diagram of the strain vs. time curve for the entire test is given in Figure 2.9..

The response of the samples to shear stress was separated into a non-recoverable apparent viscosity component and a recoverable viscoelastic component. Apparent viscosity was calculated from the non-recoverable strain (after conditioning for 20 hours following completion of the test) and the 9807 Pa. stress which was applied to the sample for 60 minutes as follows;

$$\eta = \frac{\tau}{\gamma} \qquad (2.8.)$$

where:

η, apparent viscosity,

τ, applied shear stress,

γ, shear strain rate calculated based on the measured non-recoverable flow, specimen thickness, and 3600 seconds loading time.

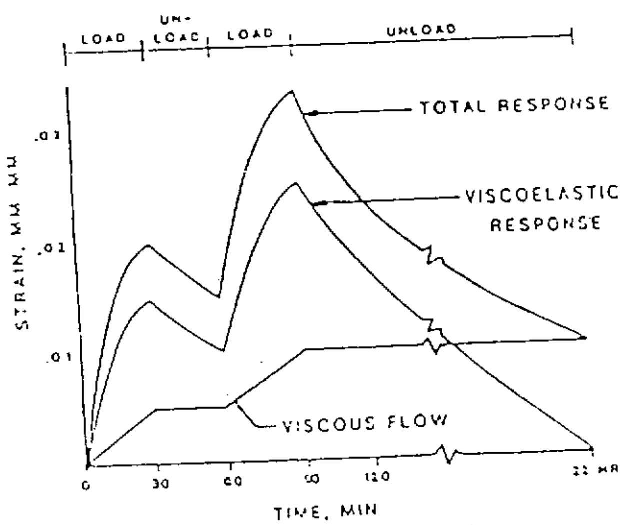


Figure 2.9. Strain as a function of time during the creep-strain recovery test (Chehovits *et al.*, 1982).

The viscoelastic creep component of the data was determined by subtracting the strain resulting from viscous flow from the total strain at each time interval. Viscous effects are not present in the strain recovery portion of the test, thus the data, as measured, is representative of the viscoelastic strain recovery response.

As a result of this research Chehovits *et al.*, (1982) described that the viscous and viscoelastic properties of bitumen-rubber materials may vary and may depend on the bitumen grade, rubber gradation, concentration of granulated rubber, and several interactions between bitumen gradation and rubber concentration.

Another research has been processed by Oliver (1982), who used the scrap rubber derived from the tread area of used passenger car tyres and was therefore mainly synthetic. He also used 85/100 pen. bitumen.

The main experimental program was to determine the effect of the digestion time on the bitumen-rubber blends. For this purpose, samples had been removed for testing after digestion time of 0.5, 1.0 and 2.0 hours. Basic deformation testing of bitumen-rubber digestions using specialized sinusoidal loading equipment indicated that a simple test based on elastic recovery of strain has been used. The apparatus developed was based on the Shell sliding plate rheometer. In the modified instrument a cast prism of the material 10 mm. thick was sheared at 60 °C between two parallel aluminum plates to a strain of 1.0 mm/mm. The shearing stress of 330 Pa was then automatically removed and the recovery of strain was measured after a period of ten times the period of straining has elapsed. The movement of the free (loaded) plate was followed by a transducer and automatically recorded. A typical strain time curve for a bitumen-rubber digestion is shown in Figure 2.10.. Two parameters were used to characterize the material: the time to reach a strain of 1.0 mm/mm (referred to as "time under stress") and percentage of this strain which is recovered when the stress is removed (percent elastic recovery).

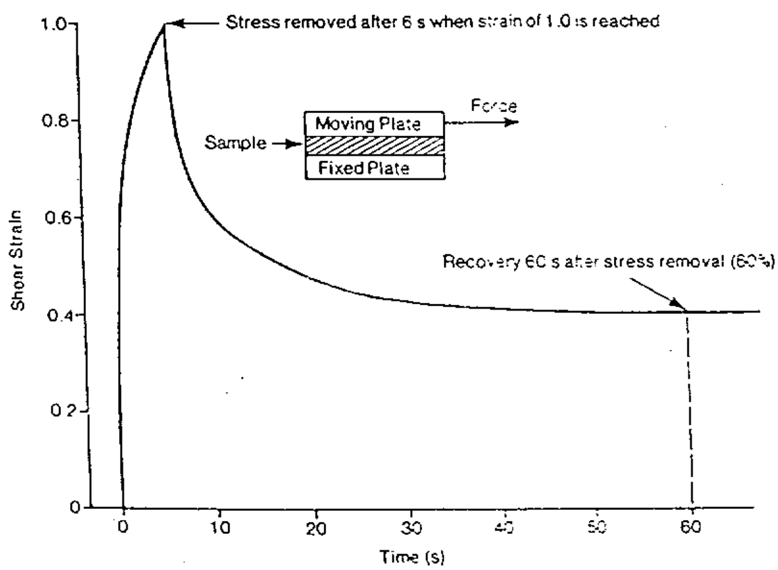


Figure 2.10. Sliding Plate Rheometer Test of a rubber-bitumen mix (Oliver, 1982).

The desirable condition, from the point of view of service on the road, is that the material exhibits a high value of elastic recovery indicating behavior approaching that of soft elastic solid.

The main variables effecting the digestion behavior and final properties of bitumen-synthetic rubber digestions are the morphology of the original rubber particles and the concentration of rubber in bitumen (Oliver, 1982).

Morphology has been characterized by measuring the bulk density of the rubber particles in water (Oliver, 1981). Porous particles have a low bulk density while smooth angular particles pack more closely to give a high bulk density. Figure 2.11. shows the relationship observed between bulk density of (undigested) synthetic and natural rubber particles, manufactured by a number of different processes, and the elastic strain recovery of 15% concentration of the particles digested in bitumen at 200 °C for 0.5 hour. For the desirable high elastic recovery condition, low bulk density (porous) rubber is required.

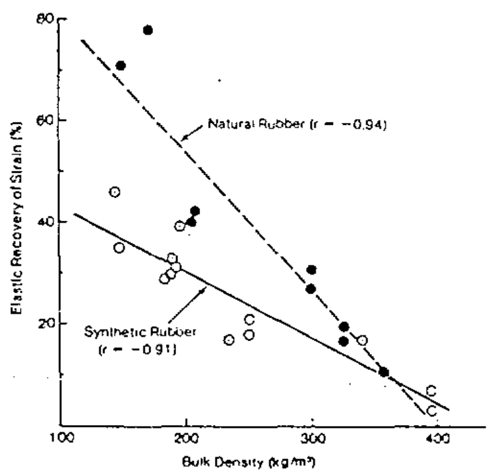


Figure 2.11. Relationship between elastic recovery and bulk density (Oliver, 1982).

A linear relationship between rubber concentration and elastic recovery was seen for synthetic rubber from car tyres digested in bitumen at 200 °C for various digestion times (Figure 2.12.) (Oliver, 1982). This figure shows that the effect of rubber particles morphology on the elastic strain recovery of digestion is greater for natural rubber digestion than for synthetic rubber ones. The effects of concentration on elastic recovery was the same for natural and synthetic rubbers, a linear relationship with a slope of about 1.0 was obtained in both cases. In Figure 2.13. it can be seen that there is a gradual improvement in elastic recovery as either time or temperature of digestion, both are increased.

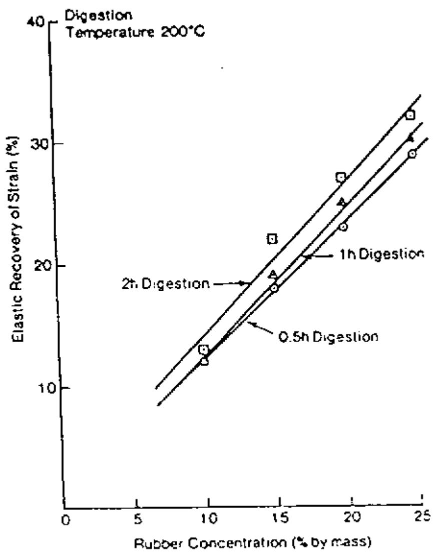


Figure 2.12. Effect of rubber concentration on elastic recovery (Oliver, 1982).

For synthetic rubber there is a systematic increase in elastic strain recovery as particle size decreases. Natural rubber digestions has been found more sensitive to the effects of time and temperature of digestion.

The effect of time and temperature of digestion on natural rubber is shown in Figure 2.14.. Natural rubber digestion tends to have somewhat superior elastic properties at low temperatures and short digestion times, when compared with equivalent synthetic rubber digestions and also to improve more rapidly than the latter with increasing time or temperature of digestion. Elastic recovery then has reached a maximum value and dropped off rapidly with further heating unlike synthetic rubber digestions where a maximum was not reached under normal operating conditions.

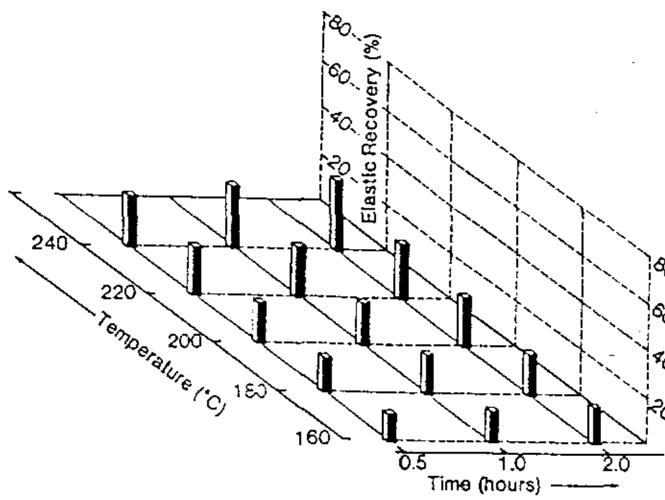


Figure 2.13. Effect of mixing time and temperature of digestion on the elastic recovery of synthetic rubber (Oliver, 1982).

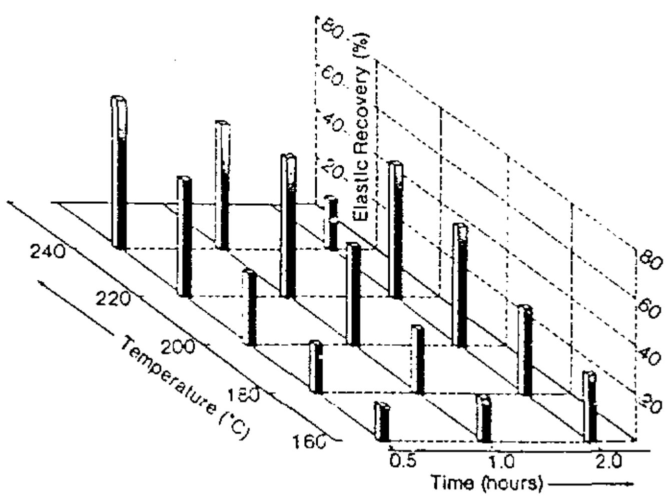


Figure 2.14. Effect of mixing time and temperature of digestion on the elastic recovery of natural rubber (Oliver, 1982).

Oliver (1982) recommended the development of an industrial manufacturing process to produce low bulk density rubber and the segregation of rubber scrap, and where possible, the use of only comminuted natural rubber and investigation of the possibility of manufacturing rubberized bitumen by adding of rubber with the aggregate fractions.

A special case is the scrap rubber of relatively large particles (4.2 mm to 6.3 mm) as a powder in bituminous mixture with rubber concentration of 3 to 4 % by weight which was originally developed in the late 1960 in Sweden (Tokallou *et al.*, 1988).

The benefits of adding rubber to the mix, beside elimination of rubber tyre waste, are increased flexibility, resistance to studded tyres, increased fatigue life, reduced noise and crack reflection control (Tokallou, 1987). This system was developed to increase skid resistance and durability and was found to provide a new form of winter ice control as well as reduced noise level (Esch, 1982). The most important difference between modified and normal mixes was indicated by the shape of the aggregate gradation curve (Figure 2.15.). To provide a space for the rubber particles, it was necessary to create a gap in the gradation curve. The rubber particles then replaced some aggregate particles.

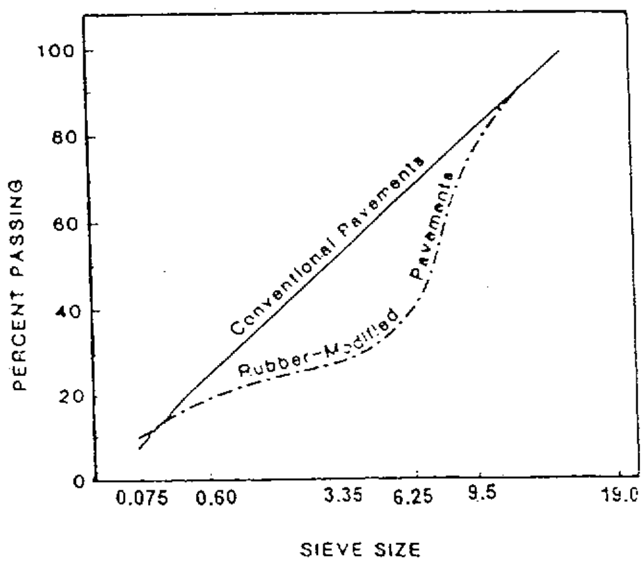


Figure 2.15. Comparative aggregate gradation curves for normal and rubber modified bitumen (rubber in mix) (Esch, 1982; Tokallou *et al.*, 1988).

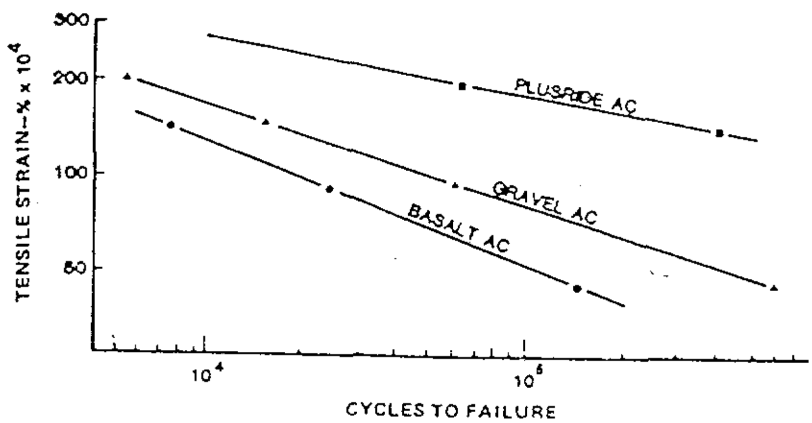


Figure 2.16. Comparisons of fatigue life for bituminous mix samples with and without rubber (Esch, 1982).

The fatigue life of rubberized asphaltic mixes has been found to be more than ten times greater than normal mixes (Figure 2.16.). They exhibit also lower creep resistance than the mixes with no rubber (Tokallou *et al.*, 1988). The mixtures without rubber show greater increase in stability with time than rubber modified mixture. Although rubber modified mixes stability was unacceptable low, there is no evidence of rutting. This would indicate that the stability test is not valid indicator of field performance for rubber bitumen mixtures of this type (Lundy, 1987).

2.6.3.2. Latex in Bitumen

Beaty (1992) studied a rubber in the form of latex which had a dry rubber content of 68.1% in 85/100 penetration grade bitumen. Tests on a rubber-bitumen blend were carried out at latex content of 2, 3, 4 and 5% by mass of bitumen. The test performed following the relevant standards were Penetration and ring and ball Softening Point. Figure 2.17. shows the relationship between temperature and penetration as a function of latex content for the range of temperature from 5 to 30 °C. From this Figure it can be seen that at low temperature penetration increases with increasing latex content, whereas at high temperatures penetration decreases with increasing latex content.

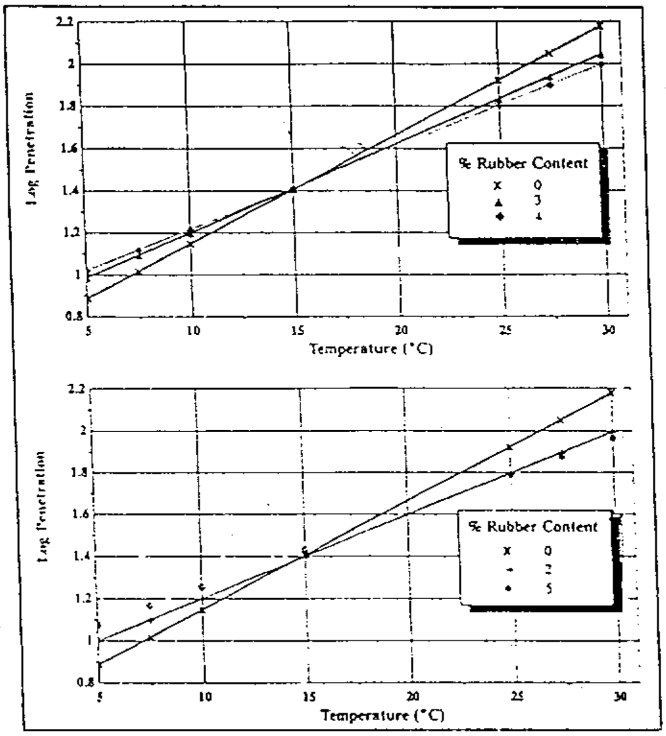


Figure 2.17. Log penetration versus temperature (Beaty, 1992).

The addition of latex into the bitumen also increases the Softening Point of the binder. Figure 2.18. shows the effect of the latex content on the softening point of the mixture.

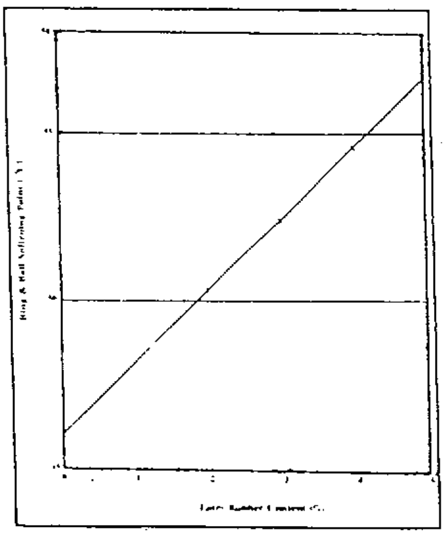


Figure 2.18. T (R&B) vs. latex rubber content (Beaty, 1992).

2.7. Summary

The literature review presented here indicates that the use of shredded rubber seems to improve the properties of bitumen and bituminous mixtures. Some of the researchers say that rubber is increases the stability of a bituminous mixture and others says that of them says rubber increases the porosity of a mixtures. McDaniels' (1994) conclusions reflect the actual consensus with relation to the research findings on the effect of waste rubber in bituminous mixtures. He says 'performance of mixtures containing shredded rubber has been mixed; some agencies have had very good success, others have had dismal and premature failures' when taken road trials into account.

However, the use of waste tyre rubber as a modifier in bituminous mixtures for pavement construction is on the increase because the large availability of used rubber tyres and the environmental problems caused when disposing them.

This review has brought into light the current problems associated with the use of rubberized bituminous mixtures and highlighted the areas of research required to make this technology available for practical applications.

CHAPTER THREE

MATERIALS AND PROCEDURES

3.1. Introduction

Asphaltic concrete consist of binder and well-graded aggregate. The binder may be a normal penetration grade bitumen or a modified bitumen. In this investigation the modifier used consist of waste shredded tyre rubber particles.

This chapter includes the materials and experimental design procedure used to evaluate the effect of mix variables on the properties of rubber modified asphaltic concrete. It contains in particular;

1-material used and their properties,

2-specimen preparation techniques,

3-type of tests,

4-test procedures.

3.2. Rubber Used in This Investigation

After reviewing the work of earlier investigators on the evaluation of the effect of rubber on bitumen and bituminous mixtures, the rubber concentration selected were 2, 5, 10, 20% by weight of the bitumen. In this investigation, vulcanised tyre tread rubber was used. The rubber, which was produced by Wellington Rubber Co. Ltd., was supplied in different size ranges. These were designated as 3,s 12,s 12/16's and 40's mesh. Table 3.1. and Table 3.2. show the gradations and specific gravities of the rubbers used in this study.

Table 3.1. Rubber Gradation Used in This Investigation

Sieve size (mm)	Passing(%)			
	3's	12's	12'/16's	40's
3.35	100.00	100.00	100.00	100.00
2.36	96.22	100.00	100.00	100.00
1.18	14.62	78.74	99.87	100.00
0.6	0.74	2.52	48.86	99.83
0.075	0.00	0.00	0.00	0.00

Table 3.2. Rubber Specific Gravities Used in This Investigation

Rubber Type	Rubber Specific Gravity(gr/cc) (by Helium Picnometer)
3's mesh	1.153
12/16's mesh	1.156
12's mesh	1.168
40's mesh	1.198

3.3. Bitumen Used in This Investigation

The base bitumens used in this study were 50 penetration and 100 penetration grade base bitumens. The two base bitumen were blended with the different sized vulcanised waste shredded tyre rubber in varying concentrations. The effect of mixing time and the mixing temperature of the rubber-bitumen blend were the other important aspects investigated

3.5.2. Method to Obtain Aggregate Gradation with Minimum Porosity

Multiple-size dry compaction of aggregates can give an advantage of increasing the packing density i.e. decreasing the porosity. Furnas (1928), Lees (1970), Tons and Goetz (1968) Cabrera and Hoque (1984) and Sardaneh (1988) studied the relationship between porosity and size compositions of mixed aggregate sizes. The packing volume concept developed by Tons and Goetz (1968) is used to introduce the ideal gradation for used specific aggregate. The major uses of the packing volume theory concept is to study the inter particle voids outside the aggregates when particles compacted in bulk.

In order to determine the minimum packing porosity in multiple-size aggregate mixes, a vibratory Dry Compaction Test (DCT) method was used which was developed by Sardaneh (1988). The DCT equipment given in Plate 3.3. consisted of a vibrating table. The selected time of vibrating was 60 seconds at 25 mm. amplitude and 100 rpm., because most of the compaction takes place in 30-60 seconds (Sardaneh, 1988) and to avoid the aggregate degradation.

A standard mould (101.6 mm. ∅) with a surcharge weight of 1200 gr. steel cylinder was used in order to assure uniformity and an even surface. Crushed lime stone were sieved in the sizes of 14.0, 10.0, 6.3, 5.0, 2.36, 1.18, 0.6 mm. and stored in different containers. In first step, the 14.0 mm. size aggregate was mixed with the 10.0 mm. size aggregate in a varied percentage (by weight) and compacted in the dry compaction procedure as mentioned earlier.

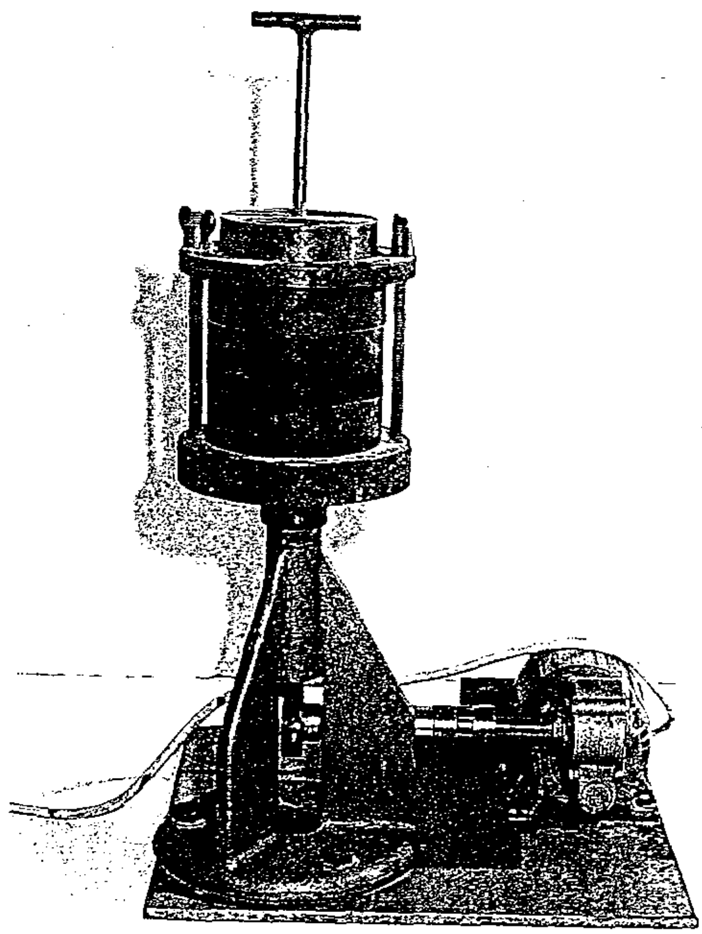


Plate 3.3. The Apparatus used for Dry Compaction Test (DCT)

The inter particle packing voids can be calculated by using the following equation:

$$Vc = \left(1 - \frac{D}{SG_{agg}}\right) \times 100 \quad (3.2.)$$

where;

Vc, packing void content (%),

D, density of the total mix (gr/cm^3),which is calculated from dividing the total weight of mix by the volume of the mix $V=(h\times(\pi\times d^2/4))$, h is measured, d=101.6 mm.,

SG_{agg}, specific gravity of the mixed aggregate (gr/cm^3).

The specific gravity of the mixed aggregate is obtained also from the following relationship:

$$SG_{agg} = \frac{100}{\sum \frac{Pw_i}{SG_i}} \quad (3.3.)$$

where;

Pwi, Percentage weight of aggregate fractions involved into the mix (%) (by weight),

SGi, specific gravity of aggregate fractions involved into the mix (gr/cm^3).

The second step is to mix the blend, which satisfy the minimum packing porosity from 14.0 mm. and 10.0 mm. aggregate size, with 6.3 mm. size with varied percentage (by weight). By using the two previous equation (3.2. and 3.3.) the packing void content at each percentage was obtained. The later steps were the repetition of the this two steps. The results of the Dry Compaction Test procedure are given in Figure 3.1.. The indications of A to G shows the aggregate sizes from bigger size to smaller size respectively as shown in Table 3.6..

The optimal aggregate gradations used in this study, which is obtained from the dry compaction test, are given in Table 3.6. column 2 and in Figure 3.1.. For comparison the gradations, according to the equation (3.1.) using different values of n are also shown in Table 3.6.. As it was seen from the same table the proposed gradation gives the lowest value of VMA. However the VMA given by the aggregate gradation obtained from the equation of Cooper *et al.*(1985) for the n = 0.5 is very close to the result obtained by DCT. The aggregate gradation with n value of 0.5 was also used during this investigation in order to investigate quantitatively the effect of small variation of particle size distribution.

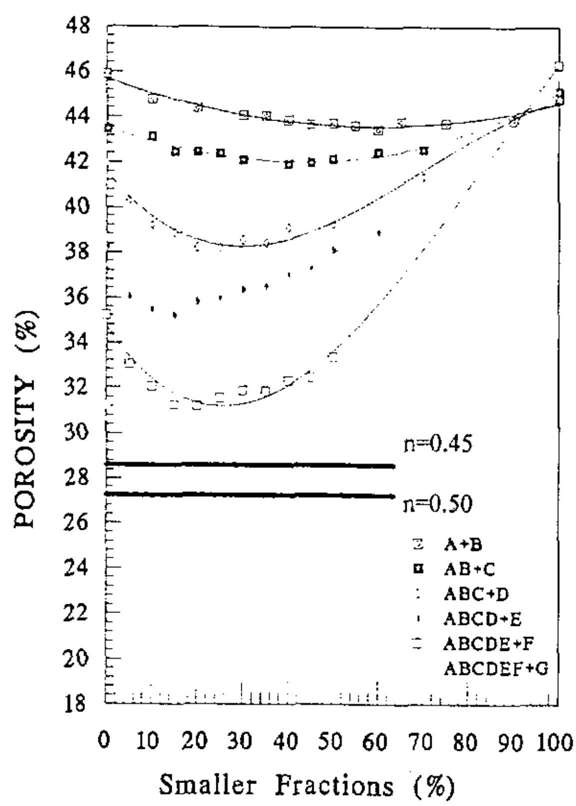


Figure 3.1. Dry compaction test results.

Table 3.6. Aggregate Gradations Their Bulk Specific Gravities and Compacted Mixture Porosities Investigated for Dry Compaction Test.

Sieve size (mm)	Weight Passing (%)				
	DCT [1]	According The Equation (3.1.)			
		n=0.45	n=0.50	n=0.60	n=0.70
14.0 **(A)**	100.00	100.00	100.00	100.00	100.00
10.0 **(B)**	90.97	86.02	84.96	82.80	80.62
6.3 **(C)**	77.42	69.98	68.03	64.19	60.45
5.0 **(D)**	62.37	63.12	60.93	56.64	52.56
2.36 **(E)**	49.82	45.18	42.76	38.25	34.19
1.18 **(F)**	40.86	33.22	31.08	27.25	23.98
0.6 **(G)**	26.20	24.65	23.00	20.13	17.81
0.075	10.00	10.00	10.00	10.00	10.00
Va[2] (%)	26.00	28.30	27.56	29.00	30.00
SG[3] (gr/cc)	2.755	2.754	2.753	2.752	2.751

1 The proposed aggregate gradation after Dry Compaction Test (DCT).
2 Air Voids in the dry compacted aggregate
3 The Bulk Specific Gravity of Mixed Aggregate (SG).

CHAPTER FOUR

DESIGN OF BITUMINOUS MIXTURES

4.1. Introduction

Modern design of bituminous mixtures involves not only compliance with structural requirements but also compliance with performance requirements during service (Cabrera, 1992). Researchers in the field of bituminous materials agree that performance can be assessed by one or more of the following properties, porosity, permeability, adhesion of binder to mineral aggregate, long term deformation (creep), and fatigue life.

The statutory methods of design for Hot Rolled Asphalt (BS 594, 1985) and Asphaltic Concrete (Asphalt Institute, 1988) provide a recipe specification or an alternative design using the Marshall test and measuring the density, porosity and voids in the mineral aggregates (VMA). In these methods the measuring parameters i.e. Marshall Stability, Marshall Flow, Density, Porosity and VMA do not give any indication of the workability of the mix or its permeability or finally of its long term resistance to deformation.

In this investigation the design method used to evaluate the effect of rubber in bituminous mixture was the Leeds Design Method, which is based on measuring not only the Marshall Stability, Flow, Density, Porosity, VMA but also measuring, air permeability, static creep stiffness and assessing the workability of bituminous mixture (Cabrera, 1992).

4.2. Design Procedure

The steps of the design procedure for the investigation are outlined below:

1- selection of aggregate grading,

2- determination of the mixing and compaction temperatures,

3- preparation of compacted specimens with varying binder type and contents,

4- prediction of the workability of mixture,

5- determination of density of compacted specimens,

6- calculation of porosity and voids in the mineral aggregate (VMA),

7- testing of specimens for permeability,

8- testing of specimens for stability and flow,

9- testing of specimens for static creep,

10- selection of optimum bitumen content.

4.2.1. Selection of Aggregate Grading

The aggregate gradations selected were the optimal aggregate gradation obtained by experimental DCT (see Table 3.6.) and also the gradation resulting from the Cooper *et al.*(1985) equation using a value of n=0.5. The gradation curve for these two aggregates are given in Figure 4.1..

4.2.2. Mixing and Compaction Temperatures

The following conditions are chosen to prepare the standard test specimens:

1- the aggregate, filler and bitumen are heated at 150 °C (±5), 2- the aggregate and filler are left into the oven for 4 hours prior to mixing and the bitumen for 2 hours to avoid from hardening,

3- material is mixed in an heat controlled mixer and mixing is completed in 1 minute,

4- the compaction temperature should not be under 130 °C, if so, the mixture is not used.

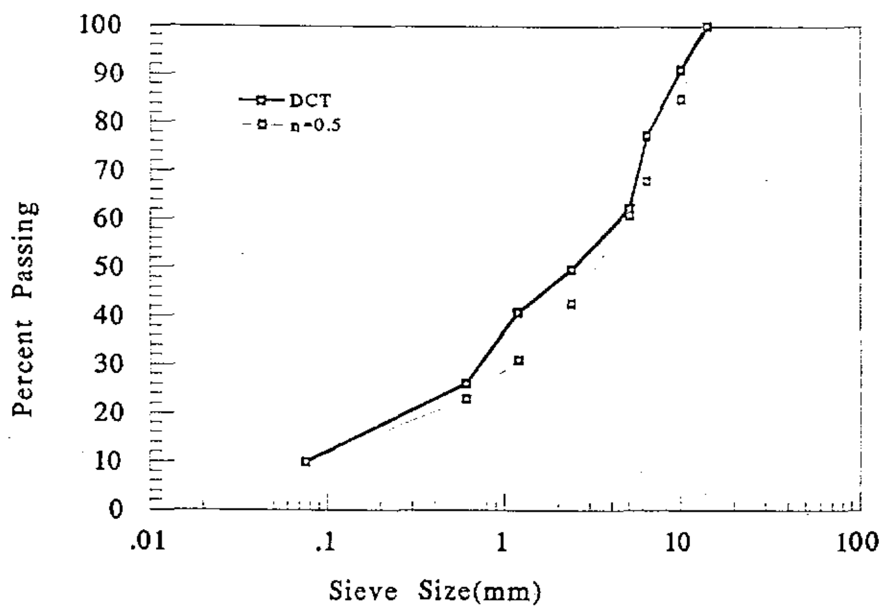


Figure 4.1. Aggregate gradations investigated.

4.2.3. Preparation of Specimens

The Leeds Design Method was used to prepare the standard 63.5 mm × 101.6 mm Ø specimens. These were compacted using the Gyratory Testing Machine for rubberized and normal bitumen. Plate 4.1. shows the Gyratory Testing Machine (GTM) used in the study. The GTM parameters were:

Angle of gyration : 1°

Axial load : 0.7 MPa

Number of revolution under load : 30

These conditions are equivalent to the energy used in the normal standard compaction using the Marshall hammer (50 blows) (Cabrera, 1991).

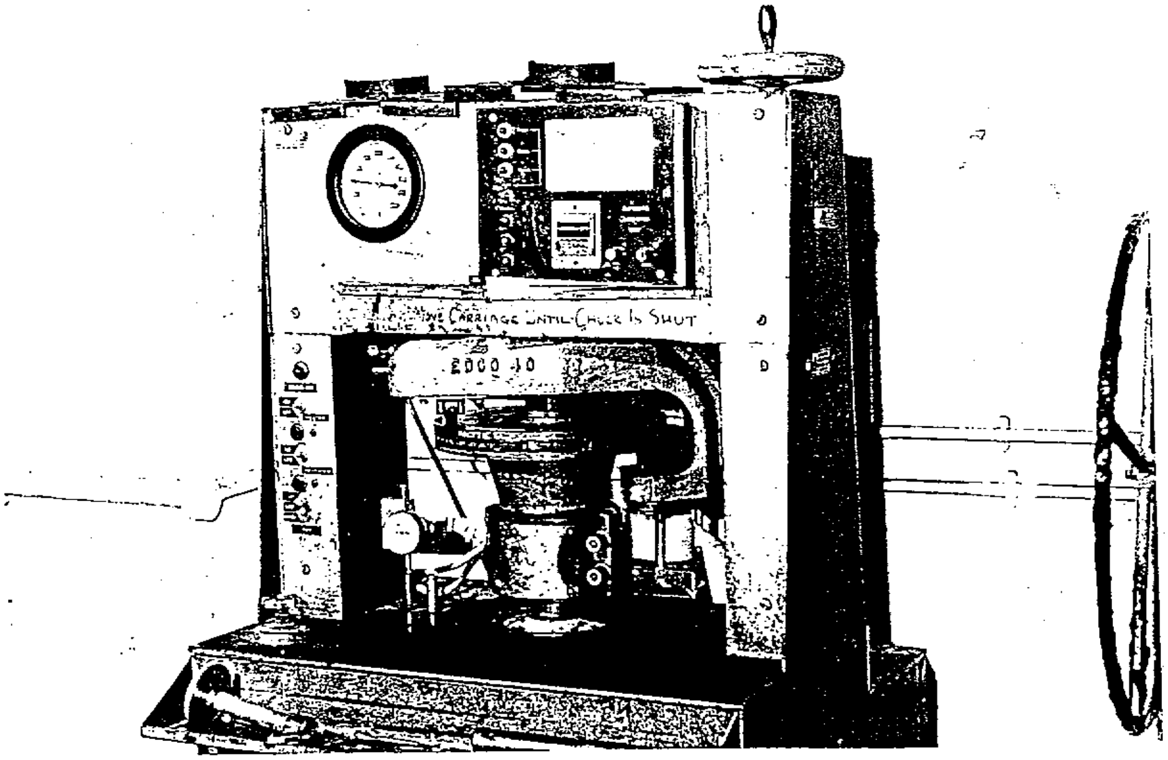


Plate 4.1. Gyratory Testing Machine (GTM).

The main variables were as follows to investigate the effect of waste shredded rubber on the properties of asphaltic concrete:

1- binder content (5.0, 5.5, 6.0, 6.5, 7.0% by weight),

2- aggregate gradation (DCT and n=0.5),

3- penetration grade base bitumen (50 and 100 penetration),

4- rubber concentrations (0, 2, 5, 10, 20% by weight in bitumen),

5- shredded rubber sizes (40's, 12/16's, 12's, 3's mesh).

Every point obtained is based on the average of two readings. The number of specimens per point were four. As a result of this, 840 specimens were prepared using the Gyratory Testing Machine to investigate the effect of rubber concentration and shredded rubber size on the properties of shredded rubber modified asphaltic concrete.

4.2.4. Workability of Bituminous Mixture

Cabrera (1991) defines the workability of a bituminous mixture as the property which allows to produce, handle, place and compact a mix with minimum application of energy. It is an important practical requirement that bituminous materials are tolerant to a wide variation of laying conditions and that in relatively adverse condition, they will still spread easily and compact sufficiently well to give a durable surfacing. Too harsh a mix, may cause it to tear under the screed of the spreading machine and render compaction more difficult. Badly compacted mixtures will develop undesirable properties of low stiffness, bad fatigue resistance, bad durability, etc. that is not expected from a good surfacing. To account for it's importance in highway constructions, the Leeds Design Method has included an assessment of workability in it's design procedure (Cabrera, 1992).

The workability of a bituminous mixture is very difficult to quantify. It is influenced mainly by aggregate type (shape, size and texture), aggregate gradation, binder grade, binder content, binder viscosity and mixing and compaction temperature (Napiah, 1993). An assessment of workability is not included in any current UK design procedure. The workabilitymeter developed by Marvillet and Bougault (1979) consist of chamber connected to a rigid frame, into which the test introduced, and a speed controller which drives a blade in the mix. They measured the torque required for mixing and proposed the reciprocal of the resistance moment produced in the mix against the rotation of the blade as the mixture workability. Thus;

W=1/(moment of spring tension/axis of rotation)

The value is expressed in 1/(kN.m).

In this investigation, the Leeds Design Method (Cabrera, 1991) was used to quantify the workability. This method is based on experimental relations between compaction energy and mix porosity. Unlike the Marvillet (1979) apparatus, the Leeds Workability Method employs a Gyratory Testing Machine (GTM) and it's determination is in conjunction with specimen compaction. The GTM is equipped with a dial gauge to continually monitor the changes in specimen height as it is being compacted. The picture of Gyratory Testing Machine is given in Plate 4.1..

The workability of the mixtures is expressed by the Workability Index (WI). WI can be used to effectively assess the influence of mix variables, temperature and compaction energy. Generally, for any hot mixed bituminous material, the workability index of the mixture can increase by increasing binder content and compaction temperature. A typical plot of the relationship between porosity and the logarithm of the number of GTM revolutions is given in Figure 4.2.. The linear relationships is of the form:

$$\mathbf{y = a - b \times \log x} \qquad \mathbf{(4.1.)}$$

where;

y, porosity (%)

a, b, constants

x, ($\log_{10}$) the number of GTM revolutions.

This equation is a statistical fit of the porosity obtained during compaction process. The Workability Index (WI) is defined as the inverse of the constant a, which is the porosity at 0 revolution, multiplied by 100.

$$\mathbf{WI = 100\ (1/a)} \qquad \mathbf{(4.2.)}$$

Detailed calculation procedure has been given in reference (Siswosoebrotho, 1988). The workability index results of the shredded rubber modified asphaltic concrete are given in Chapter 6. These results are the average of four standard test specimens.

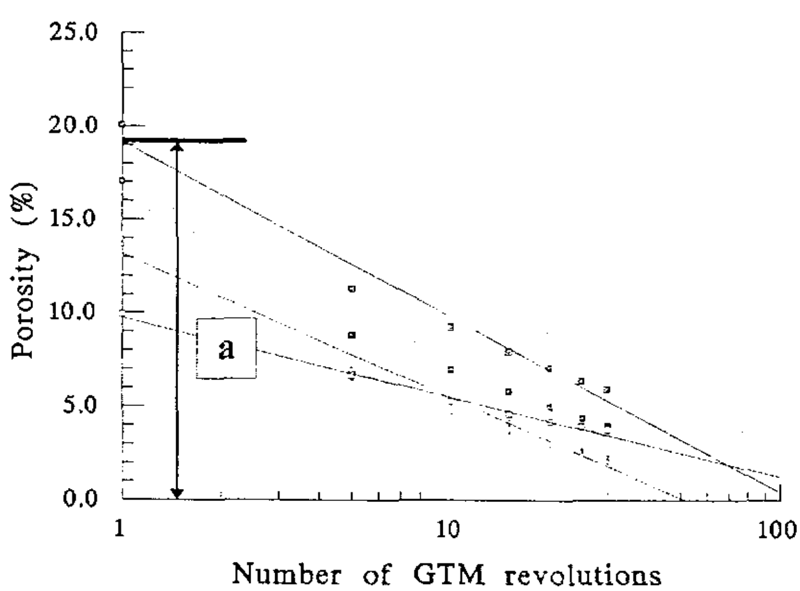


Figure 4.2. A typical example of the relationship between porosity and number of GTM revolution for asphaltic concrete made with 50 pen. base bitumen.

4.2.5. Density of Compacted Specimens

The density of the compacted specimen was determined by first weighing the dry sample in the air then in the water. The density of the specimen is calculated by dividing the weight of the sample in air by the volume of the sample as follows;

$$\mathrm{Daw} = \frac{\mathrm{Wa}}{\mathrm{Wa} - \mathrm{Ww}} \tag{4.3.}$$

where;

Daw, density of compacted specimen (gr/cm^3),

Wa, weight of specimen in air (gr),

Ww, weight of specimen in water (gr).

Also comparison was made with the density, which is calculated by dividing the weight of sample in air by the geometrical volume of specimen as follows;

$$\mathrm{Dg} = \frac{\mathrm{Wa}}{\dfrac{\pi . d^2}{4}\, h} \tag{4.4.}$$

where;

Dg, density of compacted specimen (gr/cm^3),

d, diameter of the standard specimen (mm),

h, height of the specimen (mm).

The density of the asphaltic concrete decreases with the increasing rubber concentration. The detailed descriptions are given in Chapter 6.

4.2.6. Voids Calculation

The compacted mixture was analysed for porosity and voids in the mixed aggregate (VMA). The porosity (P) and VMA provide some indication of the probable service performance of the designed mixture. The porosity is the small air spaces presents between the coated aggregate particles and is defined as the ratio of the total volume of a specimen (solid + bitumen + voids) and total voids space. It is calculated by the formula (4.5) given below. The VMA consist of the intergranular void space between the particles of aggregate in the compacted mixture (porosity plus bitumen volume gives the VMA).

$$P = (1 - \frac{D}{SG})x100 \qquad (4.5.)$$

where;

P, porosity (%),

D, density of the compacted specimen (gr/cm^3),

SG, specific gravity of the mix (gr/cm^3).

Where he specific gravity of the mix was calculated by the following equation:

$$SG = \frac{100}{\sum \frac{Pw_i}{SG_i}} \qquad (4.6.)$$

where;

Pwi, Percentage weight of individual material involved into the mix (%) (by weight),

SGi, specific gravity of individual material involved into the mix (gr/cm^3).

The VMA is calculated on the basis of specific gravity of the aggregate (Table 3.5.) (pycnometer method) and is expressed as a percentage volume of the compacted mixture. Therefore, the VMA can be calculated from the equation given below;

$$VMA = 100 - \frac{D \times P_{wa}}{SG_{agg}} \qquad (4.7.)$$

where;

VMA, voids in mineral aggregate (%),

D, density of the compacted mixture (gr/cm^3),

P_{wa}, percentage weight of aggregate in compacted mixture (%),

SG_{agg}, specific gravity of aggregate (gr/cm^3) calculated as explained in Chapter 3.

When a mixture is deficient in binder content and is placed in a pavement, it is brittle and cracks in early life, and it may ravel seriously under traffic. When the porosity is too low (approaching one per cent or less) the pavement is likely to flush or bleed. On the other hand, when the porosity is too high water and air can enter the pavement too easily and may cause damage.

Porosity can be the most important factor effecting the long term performance and durability of bituminous mixture. The porosity of the asphaltic concrete made with shredded rubber modified bitumen is discussed extensively in Chapter 7.

4.2.7. Testing of Specimens for Permeability

Permeability of bituminous mixtures has been recognised as an important parameter which effects directly the performance and durability of materials. The permeability of bituminous mixture and their durability are assumed to be more or less proportional to the porosity. Kandhal and Wegner (1975) outlined that the permeability of bituminous mixture is a better indicator of durability than porosity, because permeability measures the capacity of the porous medium to transmit fluids, while porosity says nothing about their exposure to the forces producing disintegration.

Ideally, a mix should contain disconnected voids. Dimensions of the individual voids and their lack of inter-connections could produce a specimen whose void content is relatively high but of low permeability. A wearing course with low permeability will resist the ingress of air and water and hence will low down the process of binder hardening due to oxidation and evaporation. Cabrera and Hassan (1992) concluded that the air permeability of hot rolled asphalt is influenced by porosity and binder content. It was suggested that there is a first or second order polynominal relationship between permeability and porosity.

Currently, no design method, except LDM, includes permeability as a design parameter, but as it is well correlated statistically with engineering properties that describe the structural properties of a bituminous mixture and with parameter related to its composition (Cabrera, 1992).

In this study, the Leeds Air Permeameter (LAP) was used to measure the permeability of the shredded rubber modified asphaltic concrete. The apparatus used is shown in Plate 4.2.. Details of the experimental procedure can be seen in related reference (Cabrera *et al.*, 1992).

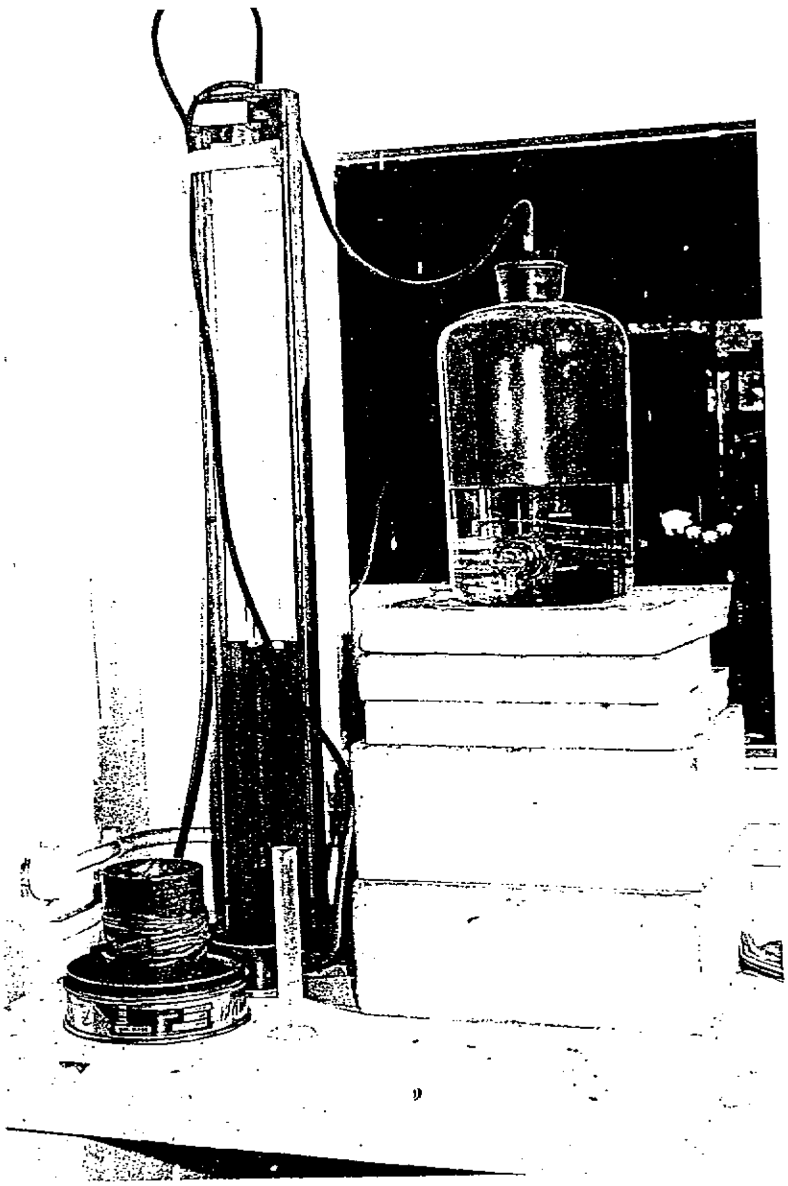


Plate 4.2. The Leeds Air Permeameter.

The coefficient of permeability is obtained from the following equation;

$$k = \frac{\mu V L}{A\, dP\, dT} \times 10^5 \qquad (4.7.)$$

where;

k, coefficient of permeability (cm/sec),

V, volume of water collected (cm^3),

μ, viscosity of air (dyne sec/cm^2),

L, height of specimen (cm),

A, cross sectional area of specimen,

dP, differential pressure (dyne/cm^2),

dT, time taken to collect water of volume V (sec).

This test was made for all standard specimens before Stability and Flow test and creep test.

4.2.8. Testing of Specimens for Stability and Flow

Stability is a measure of the ability of bituminous mixture to resist deformation under imposed loads. The stability of bituminous mixture is depend on internal friction, cohesion and ability of the ingredients to the deformation. The Marshall stability test is a type of unconfined compressive strength test.

The test specimen is compressed radially at a constant rate of strain of 50.8 mm per minute at 60 °C. The Marshall stability of each specimen is the maximum load in kilo-

Newtons that the specimen could withstand. The Marshall flow value is the total movement or strain occurring in the specimen up to the maximum load during the stability test. The test was carried out as described in BS 598: Part 107 (1990) as follows;

Two specimens at all binder contents from each binder type was subjected for stability and flow test.

Testing is carried out in the Marshall apparatus by applying load at a constant rate of strain (50.8 mm./min.) until failure. The Marshall apparatus is adapted with two transducers to a plotter for automatic recording of the load and deformation. The picture of Marshall testing machine is given in Plate 4.3..

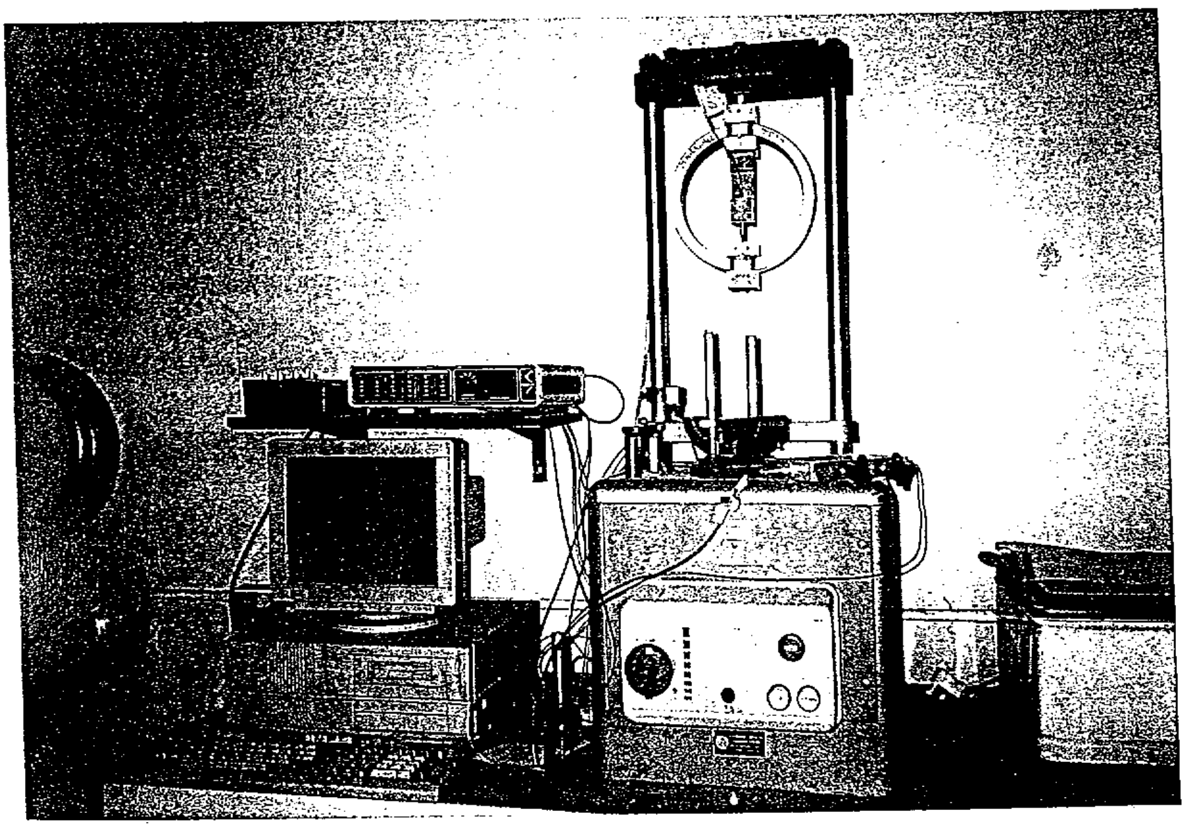


Plate 4.3. Marshall Test Equipment.

The horizontal and vertical scales are set to different magnification so as to provide a continuous suitable record of load versus deformation, from which stability and flow values can be obtained. The results of the measured stability value for samples that depart from the standard 63.5 mm. thickness is later converted to an equivalent value by means of a conversion factor. The detailed information is given in Chapter 6.

4.2.9. Testing of Specimens for Creep

The permanent deformation on the bituminous layer of a pavement is considered one of the most important problems, which result in rutting and cause potentially dangerous hydroplaning as well as reduction of the service life through disintegration of pavement structure. A further important factor is the need for a suitable test method to measure resistance to permanent deformation of the bituminous material.

Berksdale (1977) defined permanent deformation as the progressive accumulation of plastic strain in each layer of the pavement system that occurs under each load repetition. According to Monismith *et al.* (1991), permanent deformation in a bituminous layer may be caused by densification or volume change and shear deformation which is plastic deformation without any change in volume.

A model proposed by Hills (1973) to measure the permanent deformation in a laboratory is in common use in the University of Leeds. The model is based on a creep test carried out applying a static uniaxial load to a specimen maintained at constant temperature and monitoring the time deformation relation. Cabrera and Nikolaides (1987) designed

"CANIK UL" a new creep testing machine and they found that the repeatability of results acceptable.

The effect of the binder on the stiffness of a mix, in terms of prediction of creep performance, can be evaluated by applying a static load. It is possible to determine the optimum binder content at which the creep stiffness is maximum. The creep stiffness mainly dependent on temperature and hence binder viscosity (RRL, 1962) and the porosity of the mix.

The CANIK UL creep testing machine was used in this investigation to evaluate the creep properties of the asphaltic concrete. The machine consists of a loading frame to accommodate two specimens to test simultaneously. The specimens flat surfaces are treated with silicon grease to minimize the friction between loading plate and specimen. The specimens were kept in the pre-heated chamber of the apparatus at 40 °C for 60 minutes. The specimens were then placed between two loading plates and a low pressure of 0.01 MPa was applied for two minutes followed by testing a high pressure of 0.1 MPa for 1 hour. Readings are taken at intervals of 10, 20, 30, 40, 60, 120, 360, 840,1200, 2400 and 3600 seconds. For precise measurements a camcorder was used to register the dial-gauges during the first two minutes of testing.

The essential characteristic of rubber is its instantaneous recovery when the load is removed. It was assumed that the strain recovery measurements on the shredded rubber modified mixtures may be the most important factor. Bearing in mind that, after the loading period of 1 hour the load is lifted at once to measure the elastic strain recovery.

The specimens are then left for another 1 hour to read the recovered deformation at intervals of 10, 30, 60, 120, 360, 840, 1200, 2400 and 3600 seconds. The results of loading time against strain obtained from the creep test for the asphaltic concrete made with 50 pen. base bitumen for the aggregate gradation of dry compaction test result is given in Figure 4.3.. The creep stiffness of the mixture was calculated dividing the applied stress by the corresponding compressive strain.

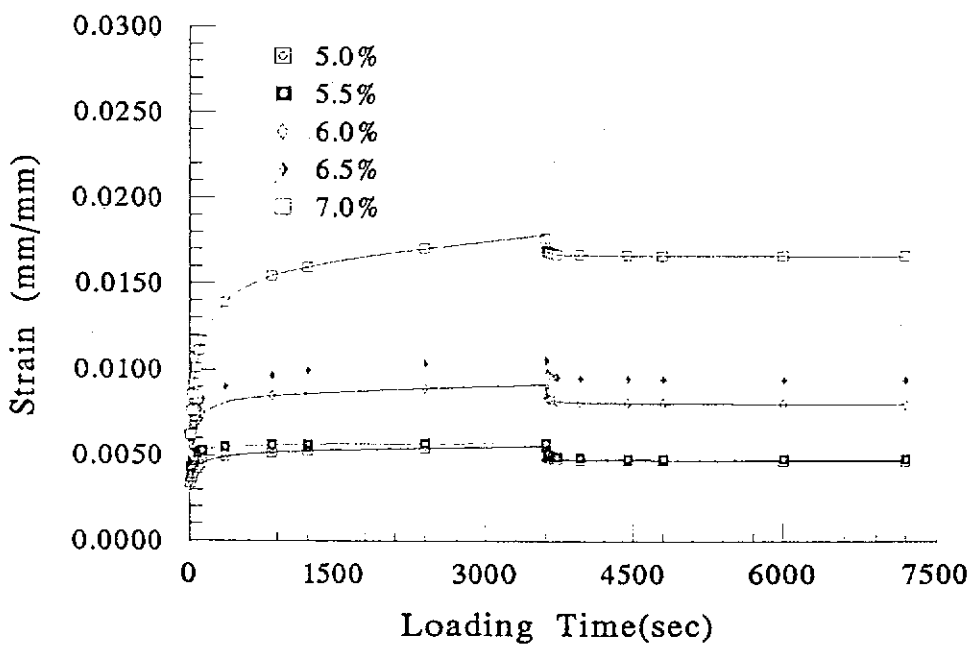


Figure 4.3. Loading Time vs. Strain for asphaltic concrete made with 50 pen. base bitumen (for the aggregate gradation type of DCT result).

The plot of creep stiffness of the mixture against stiffness of bitumen in a logarithmic scale can give an idea for the behavior of the mixture with changing binder content. The stiffness of base bitumen is normally determined using Van der Pool's nomograph for

7- air permeability,

8- workability index,

9- creep stiffness,

10- optimum binder content.

CHAPTER FIVE

RHEOLOGY OF WASTE SHREDDED TIRE RUBBER-BITUMEN BLENDS

5.1. Introduction

To take full advantage of developments in asphalt technology designers need a proper understanding of binders and its use. The use of modified binders in bituminous mixtures is likely to be an increasing feature of pavement engineering and the knowledge of the rheology of modified binders will allow a good correlation with the behavior of end-products.

One of the goals of this research is to determine which properties of rubberised bitumens offered the best correlation with the performance of asphaltic concrete. The vulcanised shredded tire rubber materials chosen for the study were of four different particle sizes and the concentration used were 2, 5, 10, 20 % mixed with two penetration grade base bitumen i.e., a 50 pen. and 100 pen. as indicated in Chapter 3.

5.2. Test on Binders

Three laboratory tests; namely Specific Gravity, Penetration and Softening Point (Ring and Ball) were conducted on all rubberised and base bitumen used in this research. All tests and determinations were made in accordance with the appropriate British Standard specifications (BS 4699, 1985; BS 2000 Part 49, 1983; BS 2000 Part 58, 1983). The elastic recovery of the binder was determined by using a Modified Ductility Test (Economou, 1992). Oscillatory shear analysis (Turpin *et al.,* 1991) was used to test the rheology of the rubberised and the two base (50 pen. and 100 pen. grade) bitumens.

5.2.1. Specific Gravity

The specific gravity of the binders was determined following British Standard Test Method (BS 4699, 1985) and also using the Helium Pycnometer Micromeritics AccuSorb 2100E model, which is available at the Civil Engineering Materials Unit of the University of Leeds. Determinations were made separately on the bitumen and on each of the shredded rubber materials. From the results obtained, the specific gravity of the rubber-bitumen blends was calculated using equation 5.1.. These values are compared with the values obtained directly from the Helium Pycnometer on the rubber-bitumen blends.

$$Sg_c = \frac{100}{\sum \frac{Pw_i}{Sg_i}} \qquad (5.1.)$$

where,

Sg_c, Calculated Specific Gravity,

Pw_i, Percentage weight of components,

Sg_i, Specific gravity of components.

The differences between the calculated values obtained by using equation 5.1. and the measured values obtained by using Helium Pycnometer can be seen in Table 5.1.. The values obtained from the Helium Pycnometer were found to be lower than the values obtained from equation 5.1.. These differences may due to the trapped air voids inside the rubber-bitumen blends. The results given in the table are the nearest 0.0001 gr/cm^3 of the obtained values as indicated BS 4699 (1985). The results obtained from Helium pycnometer were below the repeatability range of the British Standard (0.0006 gr/cm^3).

Table 5.1. Specific Gravities of Binders Used in This Investigation (in gr/cc).

Rubber size	Rubber content	Helium pic.		Calculated		BS 4699:1985		Calculated	
		p50	p100	p50	p100	p50	p100	p50	p100
	pure	1.0121	1.0116	1.0121	1.0116	1.0203	1.0182	1.0203	1.0182
40's	2%	1.0156	1.0137	1.0156	1.0151	1.0230	1.0185	1.0237	1.0216
40's	5 %	1.0194	1.0181	1.0210	1.0205	1.0263	1.0246	1.0289	1.0268
40's	10%	1.0261	1.0233	1.0300	1.0295	1.0316	1.0325	1.0377	1.0357
40's	20%	1.0374	1.0350	1.0485	1.0481	1.0411	1.0388	1.0556	1.0538
12/16's	2%	1.0159	1.0143	1.0149	1.0145	1.0251	1.0233	1.0231	1.0210
12/16's	5 %	1.0176	1.0181	1.0193	1.0188	1.0257	1.0250	1.0272	1.0251
12/16's	10%	1.0195	1.0213	1.0265	1.0260	1.0277	1.0291	1.0341	1.0322
12/16's	20%	1.0323	1.0295	1.0413	1.0409	1.0393	1.0415	1.0483	1.0465
12's	2%	1.0176	1.0134	1.0150	1.0145	1.0232	1.0219	1.0231	1.0210
12's	5 %	1.0237	1.0159	1.0193	1.0188	1.0249	1.0260	1.0272	1.0251
12's	10%	1.0321	1.0249	1.0266	1.0261	1.0251	1.0322	1.0342	1.0322
12's	20%	1.0337	1.0320	1.0414	1.0410	1.0427	1.0378	1.0484	1.0466
3's	2%	1.0146	1.0149	1.0149	1.0144	1.0238	1.0192	1.0230	1.0209
3's	5 %	1.0197	1.0166	1.0191	1.0186	1.0268	1.0252	1.0270	1.0249
3's	10%	1.0275	1.0221	1.0261	1.0257	1.0306	1.0308	1.0337	1.0318
3's	20%	1.0298	1.0306	1.0405	1.0401	1.0341	1.0338	1.0475	1.0457

5.2.2. Penetration

The consistency of the bitumens was determined by employing the standard penetration test. The test was carried out according to BS 2000 Part 49 (1983). Tests were conducted on the base and rubberised bitumens. The penetration test was carried out first to evaluate the effect of mixing time on the rubber-bitumen blend. This evaluation was made heating the binder at 160 °C and mixing the shredded rubber for various length of time. The study was made with the 50 penetration grade base bitumen and with the smallest size (40's) and the biggest size (3's) shredded rubber samples. The procedure to prepare rubber-bitumen blends was given in detail in Chapter 3. The results obtained from the penetration tests are presented in Figure 5.1.. Results presented here are the mean value of six measurements. The nomenclature used in the Figures and Tables is as follows;

A- rubber concentration in bitumen : 2, 5, 10 or 20,

B- rubber size: 3 (3's), 12 (12's), 26 (12/16's) or 40 (40's),

C- penetration grade base bitumen: 50 or 100.

Thus a code **ABC**, for example, 24050 shows a binder made with 2% 40's size rubber and 50 penetration grade base bitumen. In some cases base bitumen is indicated as p50 or p100.

The results of the penetration test show that the hardness of the binder increased with increasing mixing time up to 2 hours. As the percentage of rubber increased the penetration decreased. The differences were larger when the size of the rubber particles was smaller. This trend was as expected since in a solid liquid system viscosity increases as the volume concentration of the solid increases. After 2 hours of mixing, increasing mixing time had practically no major effect on the penetration value until a mixing time of 4 hours was reached. After 4 hours the penetration decreased sharply. To avoid oxidation of the bitumen and to reduce the energy input is was decided to prepare the blends in mixing time of 0.5 hour.

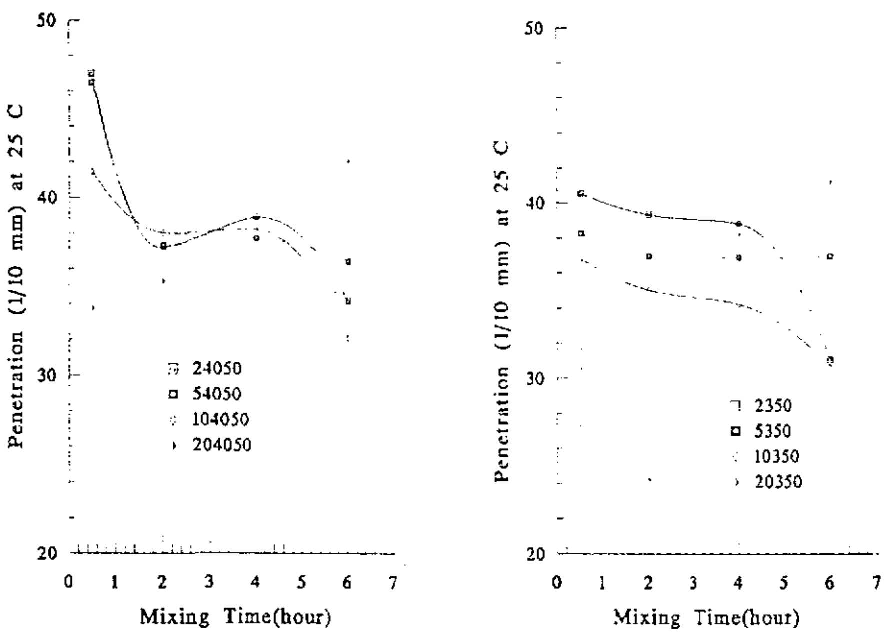


Figure 5.1. Effect of mixing time on the penetration value (different rubber size in 50 pen. grade base bitumen).

The effect of rubber concentration on the penetration test of rubber-bitumen blends is also given in Figure 5.2.. The blends were made at 160 °C for 0.5 hour. These two figures show clearly that the hardness of the rubber bitumen blends increases with the addition of rubber. The most important trend in the figures is the slope of statistical equation relating the penetration values to shredded rubber concentration. While the hardness of the blends made with 50 penetration grade base bitumen decreases with a slope of 0.97 to 0.46 with the addition of rubber (Figure 5.1. a.), the blends made with 100 penetration grade base bitumen decreased with a higher slope of 1.86 to 2.45. It can be said that the effect of rubber additives is greater on the soft bitumen than the hard one.

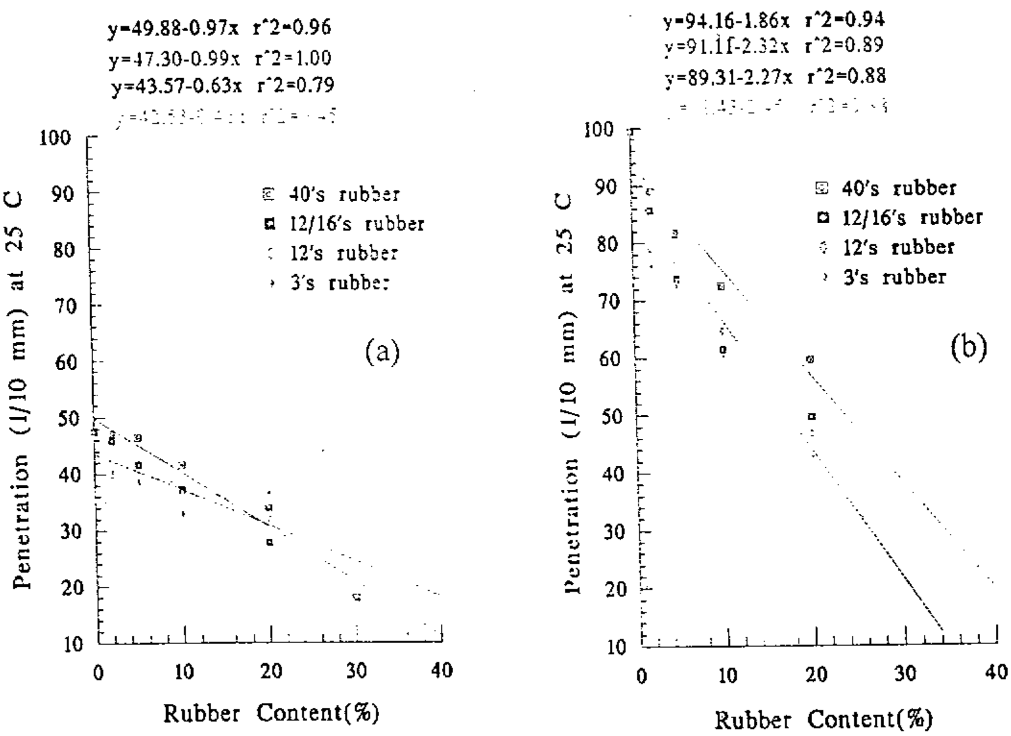


Figure 5.2. Effect of rubber concentration on penetration value (a- 50 pen. grade base bitumen) (b- 100 pen. grade base bitumen).

5.2.3. Softening Point

Bituminous binders do not exhibit a melting point but their consistency change gradually from the solid to the liquid state by the application of heat. The amount of heat required to change the consistency of the binder depends on the bitumen chemical composition.

According to Pooley (1980), the softening point is a measure of a combination of viscosity, elastic properties, density, thermal conductivity and heat capacity, all of which can vary with temperature. The ring and ball test, softening point of a binder is defined as the temperature at which a sample of binder contained in a brass ring; suspended in a water or glycerol bath, soften enough to allow a 9.55 mm. diameter steel ball of mass 3.5 grams, placed on the middle of sample, to fall through a height of 25 mm..

The softening point test was carried out in accordance with BS 2000 Part 58 (1983). The effects of mixing time and rubber concentration on the softening point test are given in Figure 5.3. and Figure 5.4. respectively.

As with the penetration test, to explore the effect of mixing time on the softening point value of the rubberised bitumen a series of tests were carried out. The results presented here are the mean values of two measurements. A logarithmic relationship was found between the softening point and mixing time. Figure 5.3. shows that the major change in Softening Point value occurs at 0.5 hour of mixing time. This was the time selected for mixing based on the Penetration test.

A linear correlation with a correlation coefficient of over 89% was obtained between the Softening Point and rubber concentration on the rubber-bitumen blends which were mixed at 160 °C for 0.5 hour. Figure 5.4. shows that the Softening Point of the blends increased with increasing rubber content.

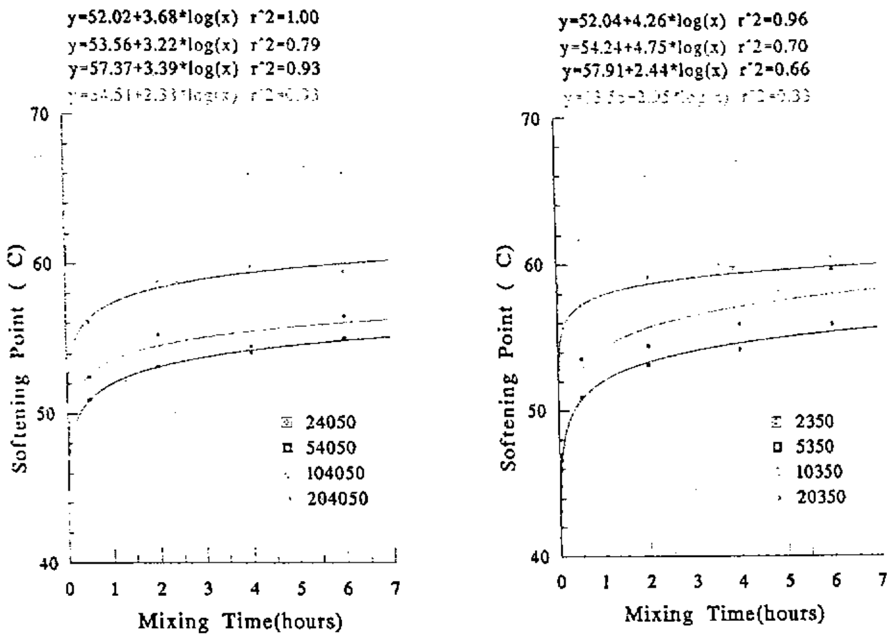


Figure 5.3. Effect of mixing time on softening point of rubber bitumen blends.

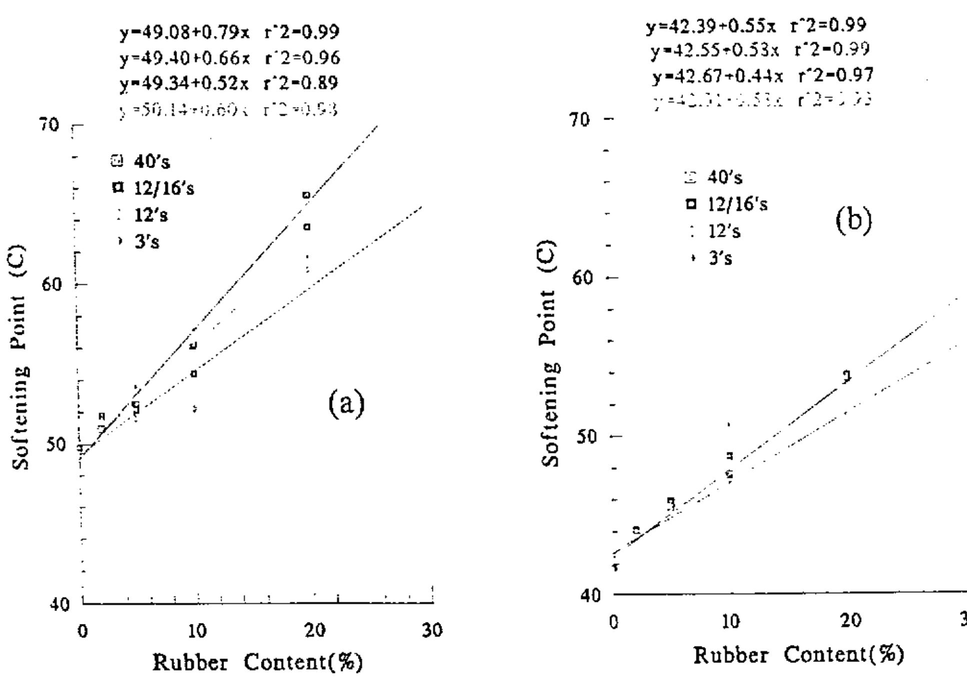


Figure 5.4. Effect of rubber concentration on softening point of rubber bitumen blends

(a- 50 pen. grade base bitumen) (b- 100 pen. grade base bitumen).

5.2.4. Penetration Index

All bitumens display thermoplastic properties, i.e. they become soft when heated and hard when cooled. The best known expression to define the consistency changes with the temperature was developed by Pfeiffer and Van Doormaal (1936). If the logarithm of penetration, P, is plotted against temperature, T, a straight line is obtained as follows;

$$\log P = \alpha T + k \qquad (5.2.)$$

where;

α, the temperature susceptibility of the logarithm of the penetration,

k, constant.

The value of α varies from about 0.015 to 0.06, it shows that there may be a considerable difference in temperature susceptibility (Shell, 1990). Pfeiffer and Van Doormaal developed an expression for the temperature susceptibility which would assume a value of about 0 for road bitumens and defined the penetration index (PI) as;

$$PI = \frac{20(1-25\alpha)}{1+50\alpha} \qquad (5.3)$$

The value of PI ranges from about -3 for highly temperature susceptible bitumens to about +7 for highly blown low temperature susceptible bitumens (Shell, 1990). The value α can be derived from penetration measurements at two temperatures T_1 and T_2 using the equation below:

$$\alpha = \frac{\log \text{pen } T_1 - \log \text{pen } T_2}{T_1 - T_2} \tag{5.4.}$$

Pfeiffer and Doormaal found that at the Softening Point temperature most bitumens had a penetration of about 800. Replacing in the above equation, T_2, by the Softening point temperature and the penetration at this temperature by 800 they obtained the following equation;

$$\alpha = \frac{\log \text{pen } T_1 - \log 800}{T_1 - T(R\&B)} \tag{5.5.}$$

Pfeiffer and Doormaal developed a nomograph based on this approach. The nomograph is given in Figure 5.5.. The Penetration Index (PI) values of the rubberized binders are obtained using this nomograph by keying the Softening Point temperature and Penetration at 25 °C. The values are given in Table 5.2..

Table 5.2. shows that the temperature susceptibility increases with the addition of certain amount of rubber when the rubber is mixed with 50 pen. grade base bitumen used in this investigation. This amount is around 5%. The temperature susceptibility reduces when this value is exceeded.

The addition of rubber into the 100 pen. grade base bitumen, on the other hand, seems to be reduced the temperature susceptibility of the binder. A gradual reduction was observed on the temperature susceptibility of the binders with increasing rubber content for all sizes of rubber used in this investigation.

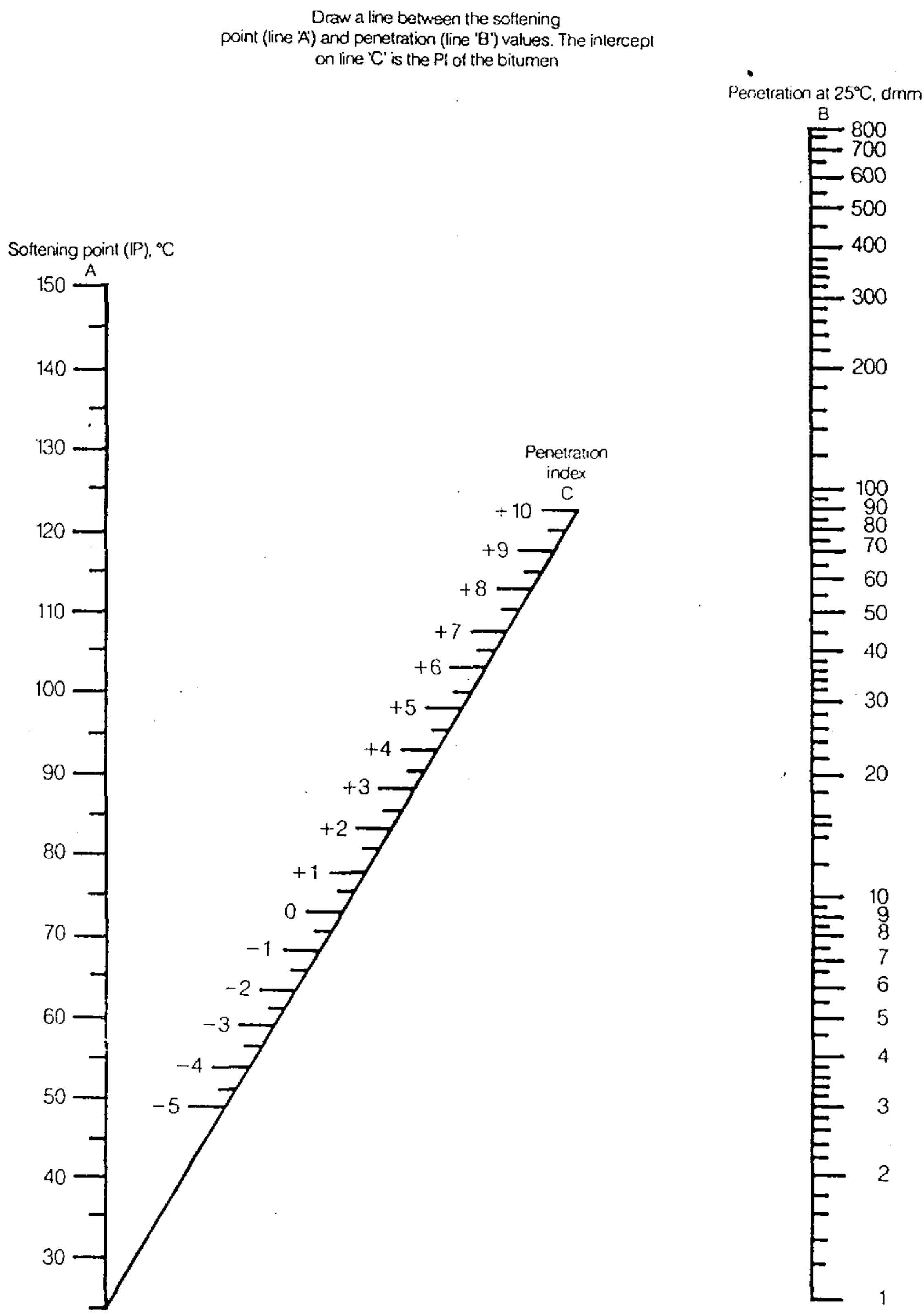


Figure 5.5. Nomograph for Penetration Index (Shell, 1990).

Table 5.2. Penetration Index of Rubber-Bitumen Blends Used in The Investigation.

Bitumen type	Penetration at 25 °C	Softening Point (°C)	Penetration Index
p50	47.50	53	-0.4
24050	47.00	51	-0.9
54050	46.50	52	-0.8
104050	41.50	56	0.0
204050	33.83	66	+1.3
304050	17.86	N/A	
22650	45.86	52	-0.9
52650	41.57	52	-1.0
102650	37.14	54	-1.0
202650	27.71	63	+0.5
21250	39.86	52	-1.1
51250	38.57	53	-1.2
101250	36.57	55	-0.9
201250	32.00	61	+0.2
2350	40.57	51	-1.2
5350	38.29	54	-0.9
10350	32.86	57	-0.6
20350	36.57	62	+0.8
p100	99.43	42	-1.8
240100	89.00	44	-1.4
520100	81.71	46	-1.0
1040100	72.43	48	-0.6
2040100	59.57	53	±0.0
226100	85.71	44	-1.5
526100	73.71	46	-1.2
1026100	61.43	49	-1.0
2026100	49.57	54	-1.0
212100	79.00	44	-1.5
512100	72.71	46	-1.4
1012100	64.71	47	-1.4
2012100	46.86	51	-1.0
23100	76.00	45	-1.5
53100	73.00	46	-1.2
103100	60.29	51	-0.5
203100	42.86	54	-0.5

5.2.5. Elastic Recovery

Permanent deformation is one of the main problem on the bituminous pavement mixtures. The reason caused to the permanent deformation by the wheel passes on the pavement is the bitumen between the aggregate particles. Any improvement on the elastic properties of the binder will achieve that. A mixture made with rubber and bitumen may have higher elastic recovery.

The Elastic Recovery of rubber-bitumen blends was measured using a Modified Ductility Test (Economou, 1992). Plate 5.1. shows the equipment used for the measurement of recovery of the rubberised bitumen. This test is a modification of the ductility test (ASTM D113, 1986). The standard ductility test specimen conditioned to 18 °C ±0.2 is subjected to a tensile stress in the standard ductility test bath and allowed to break. The elongation at break is then measured. The test sample is allowed to rebound for 30 minutes after the break then the recovery is measured. Plate 5.2. shows the samples removed from the ductility test mould. The tests were carried out on the mixes, which were made for penetration and softening point to predict the mixing time effect. The elastic recovery of binders is calculated as percentage of the elongation. The results obtained are given in Table 5.3..

From the limited number of results obtained the only clear conclusion is that the percent recovery, which is a measure of rubber-bitumen blends elasticity, increases with increasing rubber content and mixing time. Probably, because the rubber particles are digested to a greater effect as the mixing time increases.

Table 5.3. Elastic Recovery of Rubber-Bitumen Blends.

Binder	0.5 h. mixing time			2 h. mixing time			4 h. mixing time			6 h. mixing time		
type	R(1)mm	D(2)mm	% R	R(1)	D(2)mm	% R	R(1)	D(2)mm	% R	R(2)	D(1)mm	% R
50PBB	0	34.5	0									
24050	3.1	8.9	35%	4	9.5	42%	4.2	9.9	42%	4.15	9.35	44%
54050	4.55	10.2	45%	6.85	10.85	63%	7	11.55	61%	6.55	10.7	61%
104050	7.5	11.1	68%	10	14.35	70%	9.5	13.35	71%	10.3	14.25	72%
204050	9.1	13.05	70%	18.7	23.1	81%	19.65	24.25	81%	19.15	23.15	83%
22650	4	11.55	35%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
52650	5.25	12.35	43%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
102650	7.05	11.4	62%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
202650	8.5	11.45	74%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
21250	3.6	8.4	43%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
51250	6.5	9.75	67%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
101250	7.4	11.25	66%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
201250	7	7.7	91%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2350	1	17.5	6%	3.05	10.25	30%	4.85	14.75	33%	3.2	7.5	43%
5350	6.65	12.3	54%	8	25.2	32%	6.75	10.1	67%	8.5	11.25	76%
10350	8.35	12.75	65%	8.85	10.95	81%	8.7	11	79%	8.85	11.5	77%
20350	7.45	9.1	82%	7.5	9.5	79%	12.1	14.5	83%	10.9	13.35	82%
100PBB	0	100	0%									
240100	7.4	22.5	33%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
540100	8.1	24.5	33%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
1040100	9.2	22.75	40%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2040100	8.9	16.15	55%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
226100	0	29.1	0%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
526100	5.55	14.1	39%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
1026100	8.95	16.9	53%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2026100	8	12.65	63%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
212100	3.5	14.05	25%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
512100	4.75	15.25	31%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
1012100	7.25	14	52%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2012100	9.15	13.25	69%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
23100	0	63.25	0%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
53100	5.9	14.75	40%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
103100	6.3	11	57%	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
203100	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

(1) Recovery (mm),
(2) Ductility (mm).

From the results obtained, it was concluded that the rest of the programme should be carried out on mixes which will be prepared at 160 °C for 0.5 hour mixing time. Hence, the number of binder tested at different mixing time were 8 rubber modified bitumen at different rubber concentrations.

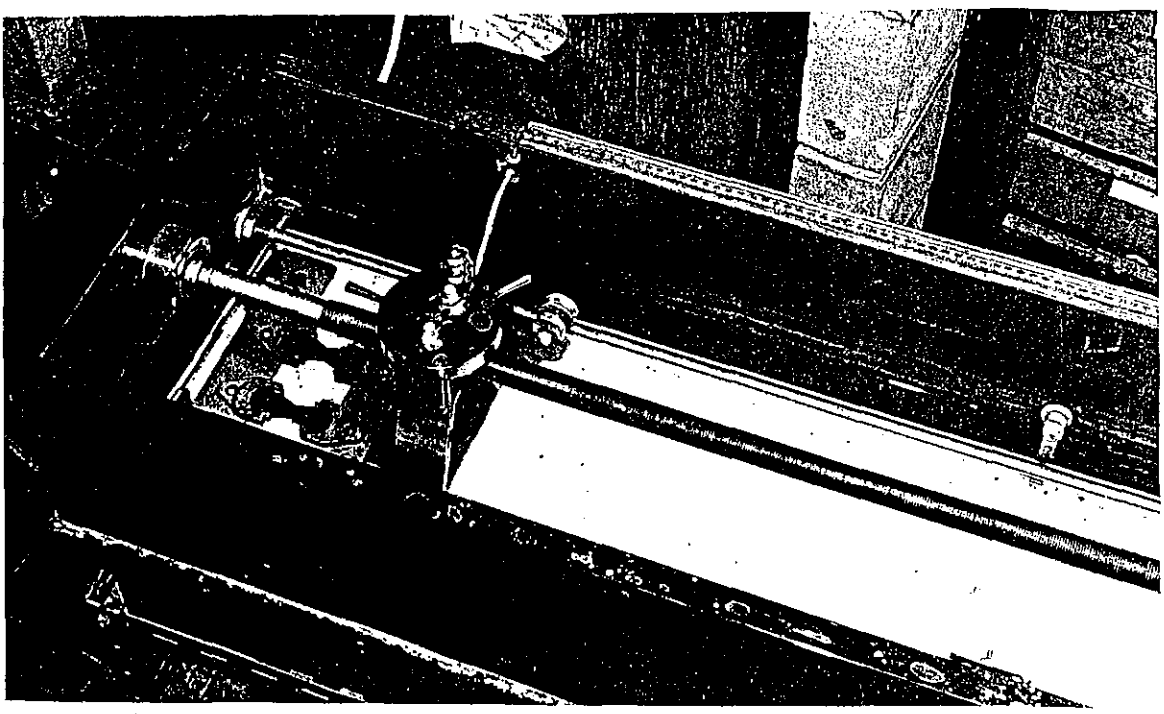


Plate 5.1. Ductility Testing Machine.

Plate 5.2 shows the rubber bitumen blends removed from the ductility mould after the elastic recovery test. Four different combination are shown (as an example) in the Plate 5.2., which are 12/16's rubber in 50 pen. base bitumen (top right), 12's rubber in 50 pen. base bitumen (top left), 3,s rubber in 50 pen. base bitumen (bottom left) and 40's rubber in 100 pen base bitumen (bottom right). Two samples were tested for each point. In the plate, each two samples represent the different rubber concentration from left to right as 2, 5, 10 and 20%. It can be say from the plate that the binder made with lower rubber content has higher ductility but lower recovery.

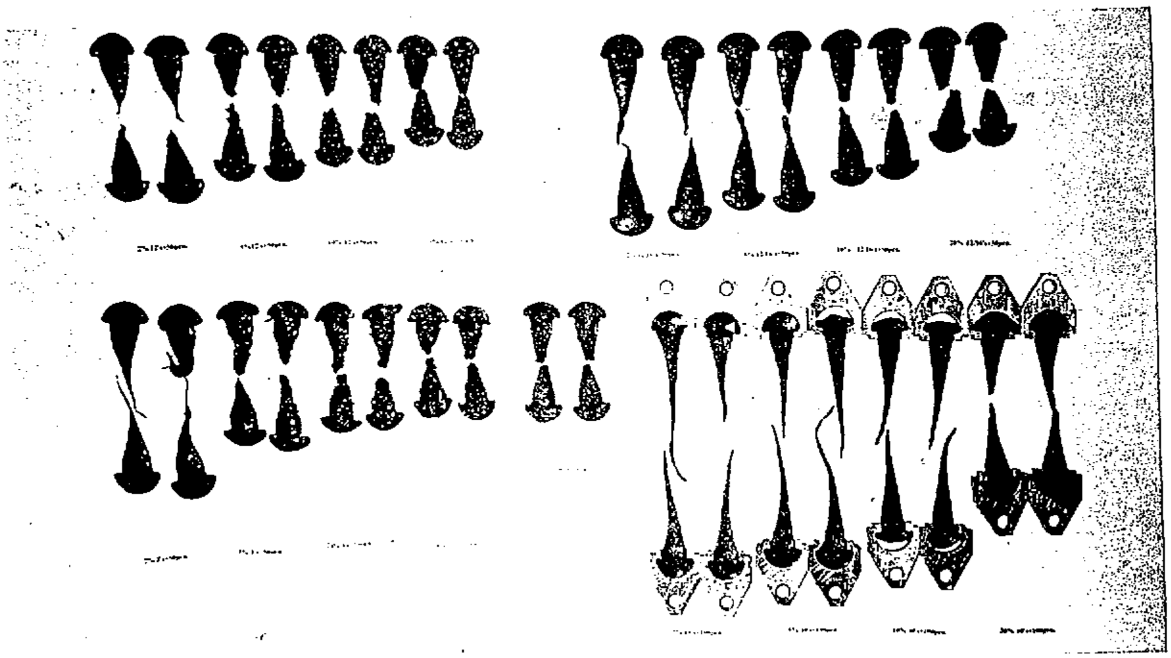


Plate 5.2. Elastic Recovery of Shredded Rubber Modified Bitumens.

5.2.6. Oscillatory Shear Analysis

The classical theory of elasticity deals with mechanical properties of perfectly elastic solids, for which according to Hooke's law stress is always proportional to strain but independent of the rate of strain. The theory of hydrodynamics deals with the properties of perfect liquids, for which according to Newton's law the stress is always directly proportional to rate of strain but independent of the strain itself. These categories are idealizations, however any real solid shows deviations from Hook's law under suitably chosen conditions, and it is probably safe to say that any real liquid would show deviations from Newtonian flow if subjected to sufficiently precise measurements.

5.2.6.1. Description of Process

The use of an oscillatory, dynamic rheological method is able to obtain the complex rigidity modulus, G* of the material into storage (elastic) G' and loss (viscous) G" components. This allows to understand the behavior of a binder in bituminous mixtures at different temperatures and frequencies.

Rheological measurements were performed using a Carri-Med CSL500 controlled stress rheometer. All measurements were made with 2.5 mm. diameter parallel plate geometry with a 1 mm. gap. The measuring system is shown in Figure 5.6.. The sample is placed between two plates filling the gap of 1 mm. as shown in the Carri-Med manual(1989). The stress is then selected within the linear viscoelastic region (LVER) (Turpin et al., 1991) and a frequency sweep is conducted over the range of 0.05 to 5 Hz (0.1π to 10π rd/sec). Measurements of this kind are made at temperature in the range of 30 °C to 100 °C in 10 °C steps and at 25 °C.

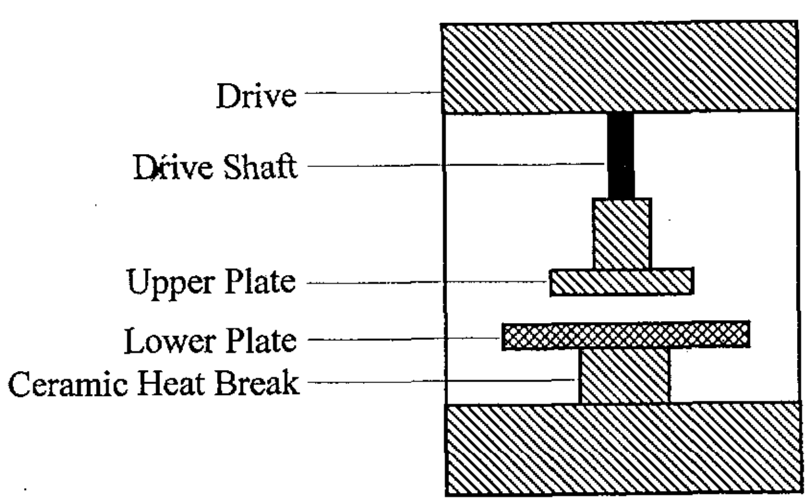


Figure 5.6. Schematic diagram of Carri-Med measurement system.

In this experiment, sinusoidal shear stress was applied to the binder samples. When the binder is cold and brittle, it may behave as nearly ideal solid, the strain exactly follows the sinusoidal input stress. At elevated temperature most bitumen will approach ideal liquid, Newtonian behavior (Verga *et al.*, 1975). In this situation the phase lag between strain and stress is 90 °C. Viscoelastic materials, such as bitumen, fall between the ideal elastic solid and ideal viscous fluid. Depending on the temperature and stress frequency, the peak strain of viscoelastic materials can be anywhere between 0° and 90° behind the maximum applied stress (Figure 5.7.). The lag, or phase angle, is referred to as delta.

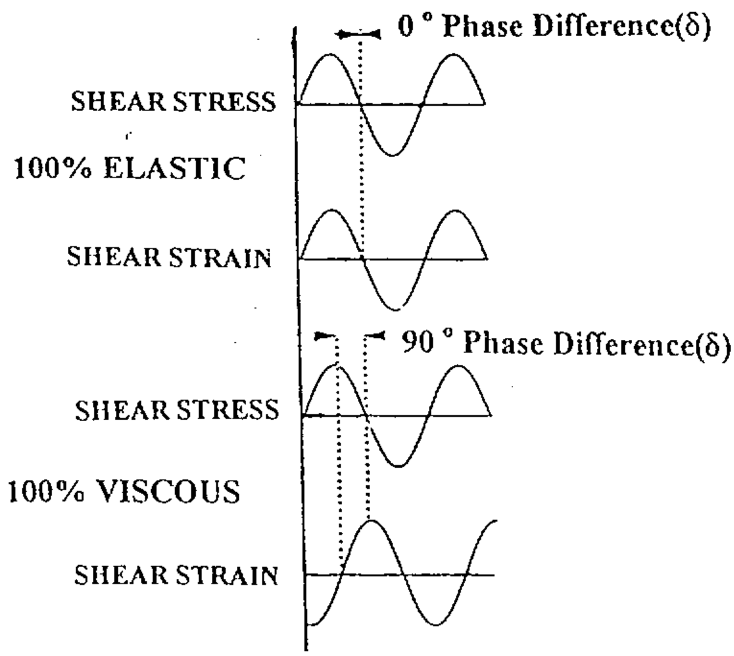


Figure 5.7. Oscillatory Shear Analysis.

At different temperature and by varying frequency of the sinusoidal stress, the viscoelastic behavior of binders can be measured. If the dynamic testing is done by using oscillatory shear stress (in the linear viscoelastic region) the data obtained at higher and lower temperatures can be equated simply and graphically with lower and higher temperatures respectively. The data were analysed using the reduced variable time-temperature superposition procedure of Ferry (1961).

In this procedure time-temperature superposition analysis packages program (Turpin, 1991) which based on Williams Landel and Ferry (WLF) (1955) equation is used to analyse the data. In this analysis the ratio of peak stress to the peak strain, at each steps, is the absolute value of complex shear modulus G*;

$$G^* = \frac{\textbf{Peak Stress}}{\textbf{Peak Strain}} \qquad (5.6.)$$

The in-phase component of G* is called the shear storage (elastic) modulus (G');

$$G' = G^* \cos.\delta \qquad (5.7.)$$

Delta (δ) is the phase angle between the applied stress and strain (Figure 5.6.).

The out-of-phase component of G* is called shear loss (viscous) modulus (G");

$$G'' = G^* \sin \delta \qquad (5.8.)$$

The units of G*, G' and G" are Pascal.

The complex viscosity η^* is obtained by dividing the complex shear modulus by the frequency (w);

$$\eta^* = \frac{G^*}{w} \qquad (5.9.)$$

and the dynamic viscosity;

$$\eta' = \frac{G''}{w} \tag{5.10.}$$

and the dynamic elasticity;

$$\eta'' = \frac{G'}{w} \tag{5.11.}$$

These forms of all the parameters were calculated via the software developed by Turpin (1991). The rheological parameters calculated for the 50 and 100 pen. grade base bitumen are given in Figure 5.8. (a) at 30 °C, 5.8. (b) at 60 °C and 5.8. (c) at 90 °C and the rest in Appendix A.

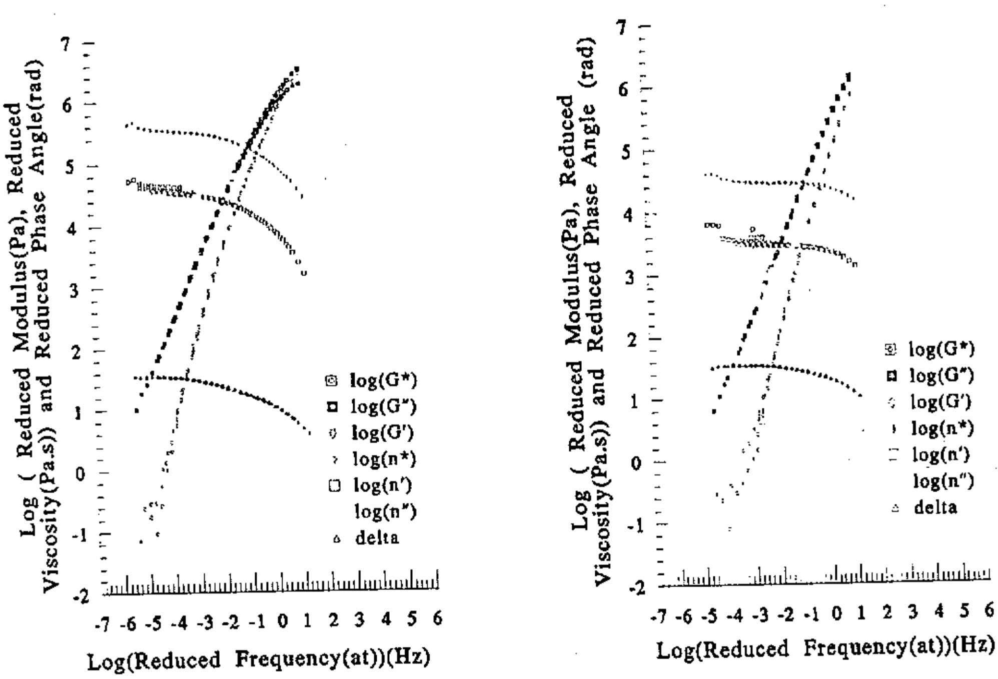


Figure 5.8. (a). Rheological parameters for 50 (left) and 100 (right) pen. base bitumen at 30 °C.

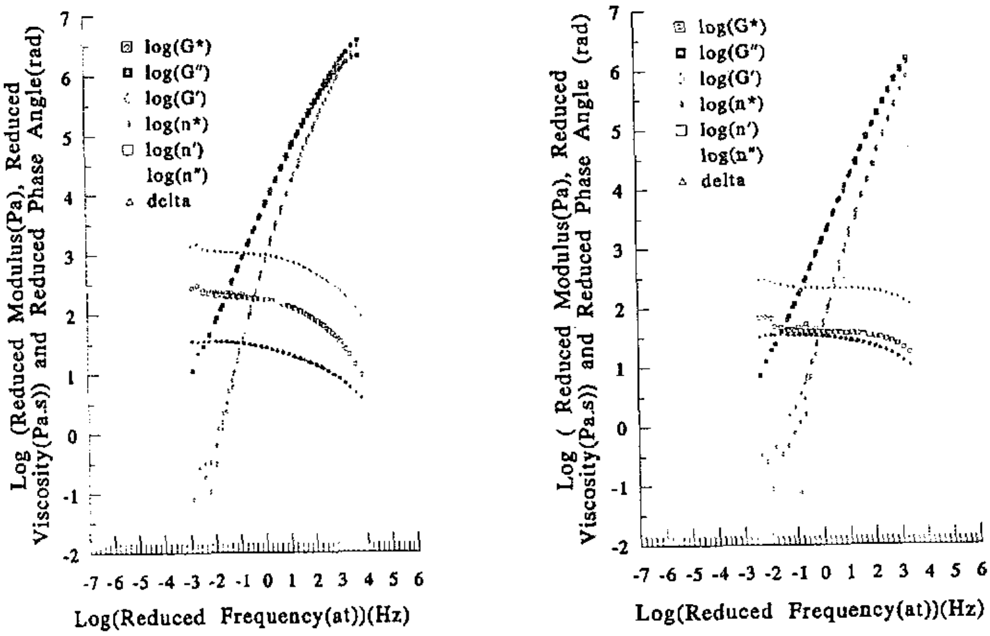


Figure 5.8. (b). Rheological parameters for 50 (left) and 100 (right) pen. base bitumen at 60 °C.

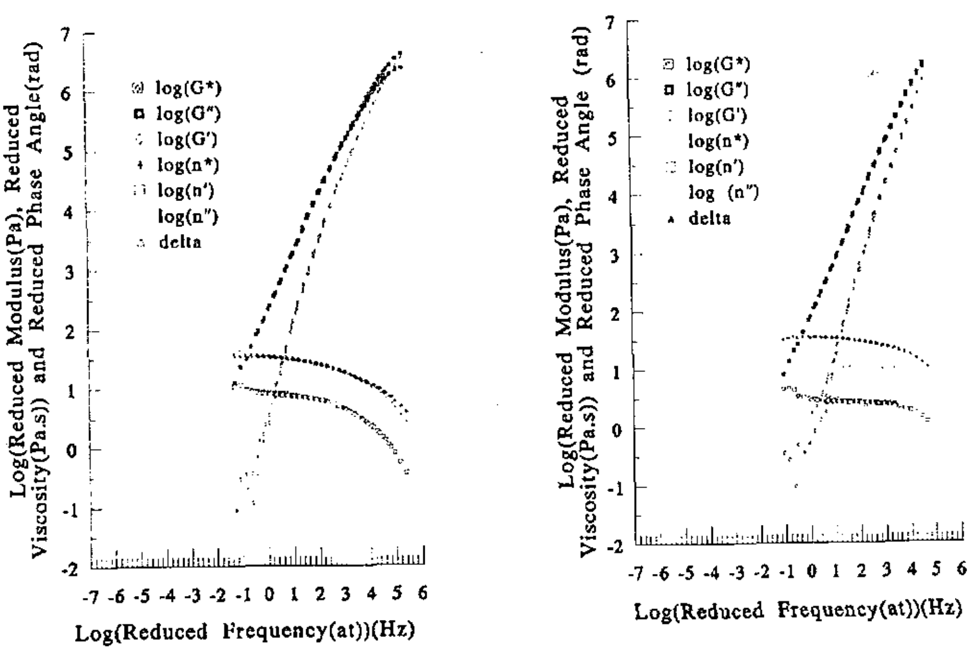


Figure 5.8. (c). Rheological parameters of 50 (left) and 100 (right) pen. base bitumen at 90 °C.

5.2.6.2. Results and Discussion

Phase angle is an important parameter in any dynamic measurements. It gives a good idea of the deviation of the material from elastic to viscous. The behavior of the binder can be explained by the changing phase angle. The changes on the properties of binder are shown by the changing phase angle, from viscous character to elastic character. The graphs show that at low frequency or long loading time base bitumen shows nearly viscous behavior, the behavior of base bitumen changes from viscous fluid to elastic solid character with the increase in frequency. For example, at the test temperature of 30 °C (Figure 5.8. (a) left), the phase angle δ is about 1.57 rad. (90^o) for the frequency of between 4×10^{-6} Hz and 4×10^{-5} Hz. for 50 pen base bitumen. It means that bitumen shows nearly viscous behavior. The phase angle then starts to decrease up to 0.6 rad (34.38^o) with increasing frequency of up to 10 Hz. The same behavior was observed for the 100 pen. base bitumen with different values. These findings show that, when loading time is large or loading frequency is low the base bitumen behaves as nearly as an ideal viscous material. The decrease on loading time changes the behavior of bitumen from viscous to viscoelastic character after a certain amount of loading time. This value depends upon the type of bitumen and the temperature.

The rubberised bitumen, on the other hand, showed more elastic behavior specially at higher rubber concentration and with large size of rubber modifier. The rubberised bitumen modified with 40's size rubber in low concentration, did not show too much differences when compared with the base bitumen (Figure 5.8.(a), 5.8.(b), 5.8.(c) and in Appendix A.1., A.2., A.3., A.4., A.5., A.6.). The increasing rubber content increased the elastic behavior of the binder. For example, when the phase angle was about 1.57 rad. for the rubberised bitumen made with 50 pen. base bitumen plus 2, 5 and 10% 40's size rubber at low loading time between 4×10^{-6} Hz. and 4×10^{-5} Hz. (Figure A.1., A.4. and A.7. in Appendix A), this value was remarkably low for the rubberised bitumen made with 20% 40's size rubber ($\delta \approx 1.4$ rad. 80^{o}). The other important difference observed was the lower phase angle at lower frequency. It was seen particularly on the rubberised bitumen made with low penetration grade bitumen plus high rubber content and large rubber sizes. For example, at the test temperature of 30 °C (Figure 5.8. (a) right), the phase angle δ is about 1.57 rad. (90^{o}) for the frequency of between 1×10^{-5} Hz and 0.01 Hz. for 100 pen base bitumen. This value changes between 1.1 to 1.2 rad. for the rubberised bitumen made with 100 pen base bitumen plus 20% 40's size rubber (Figure A.10. right), 0.3 to 1.1 rad. for the rubberised bitumen made with 100 pen base bitumen plus 20% 12/16's size rubber (Figure A.22. right), 0.2 to 0.6 rad. for the rubberised bitumen made with 100 pen base bitumen plus 20% 12's size rubber (Figure A.34. right), 0.2 to 0.4 rad. for the rubberised bitumen made with 100 pen base bitumen plus 20% 3's size rubber (Figure A.46. right). It shows that rubberised bitumen behaves as nearly as elastic behavior even at long loading time.

A focus of this investigation is to establish the significance of the viscoelastic properties obtained from oscillatory shear analysis on both conventional and waste shredded rubber modified bitumen in asphaltic concrete. The useful plots of the rheological properties are as a function of temperature (Goodrich, 1988). Goodrich (1988) was chosen a frequency of 0.1 rd/s (0.0159 Hz.). In this investigation the frequency used was 0.05 Hz (0.1π rd/s). The complex viscosity of the shredded tire rubber modified bitumen against temperature is given in Figure 5.9. (a), 5.9. (b), 5.9. (c) and 5.9. (d).

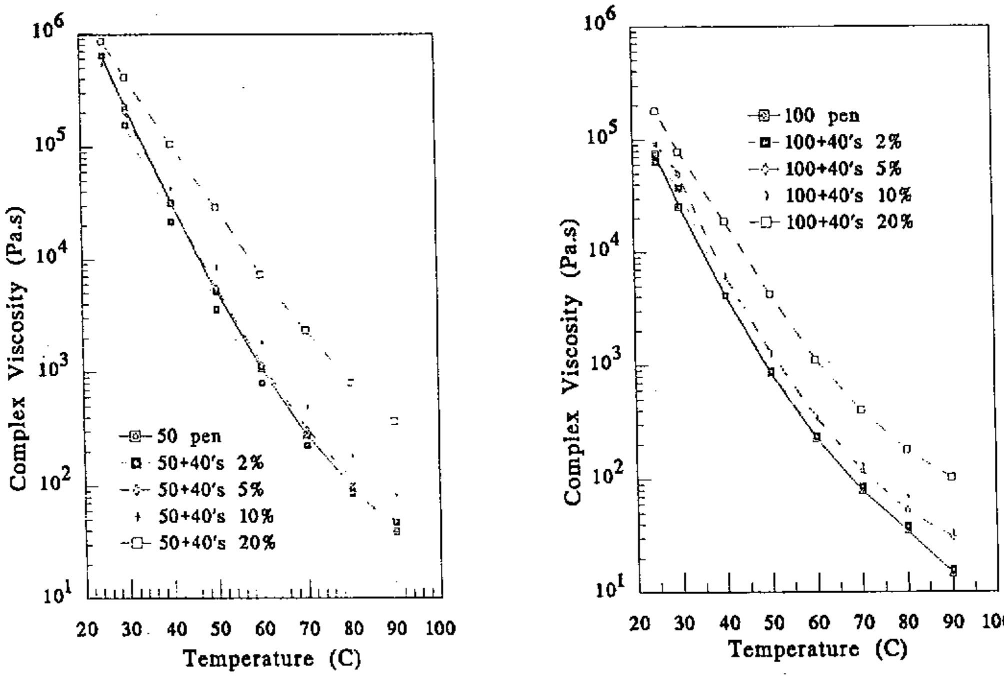


Figure 5.9. (a). Temperature vs. Complex Viscosity (μ*) (40's rubber size in 50 and 100 pen. grade base bitumen).

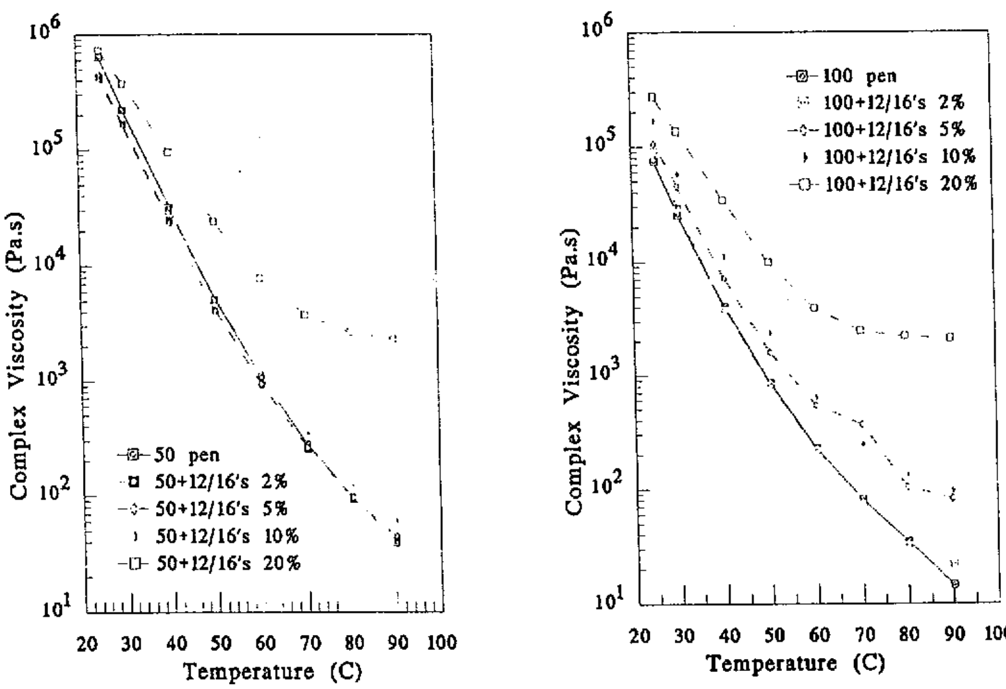


Figure 5.9. (b). Temperature vs. Complex Viscosity (μ*) (12/16's rubber size in 50 and 100 pen. grade base bitumen).

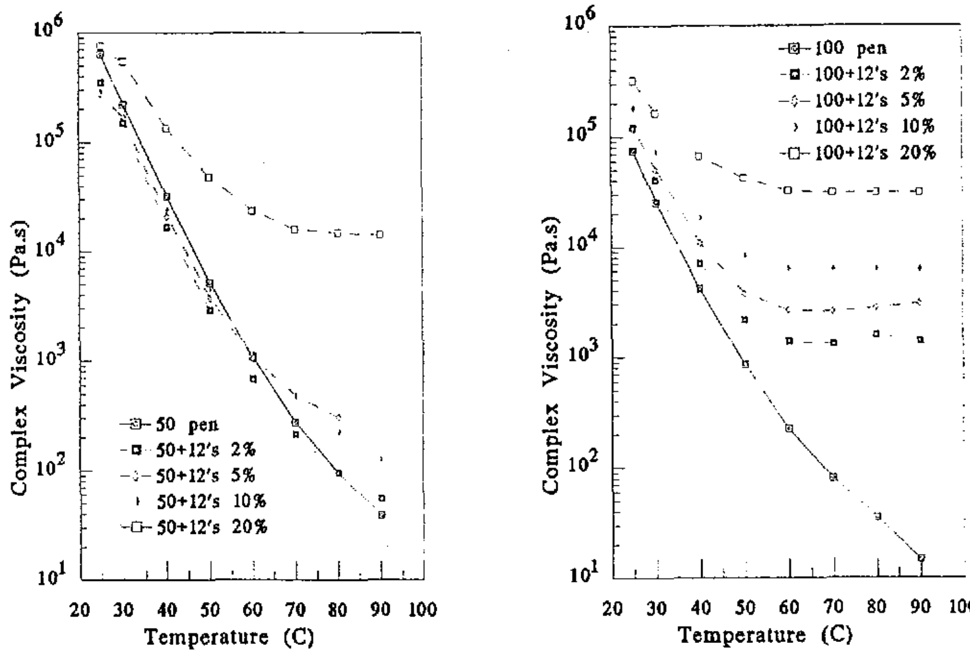


Figure 5.9. (c). Temperature vs. Complex Viscosity (μ*) (12's rubber size in 50 and 100 pen. grade base bitumen).

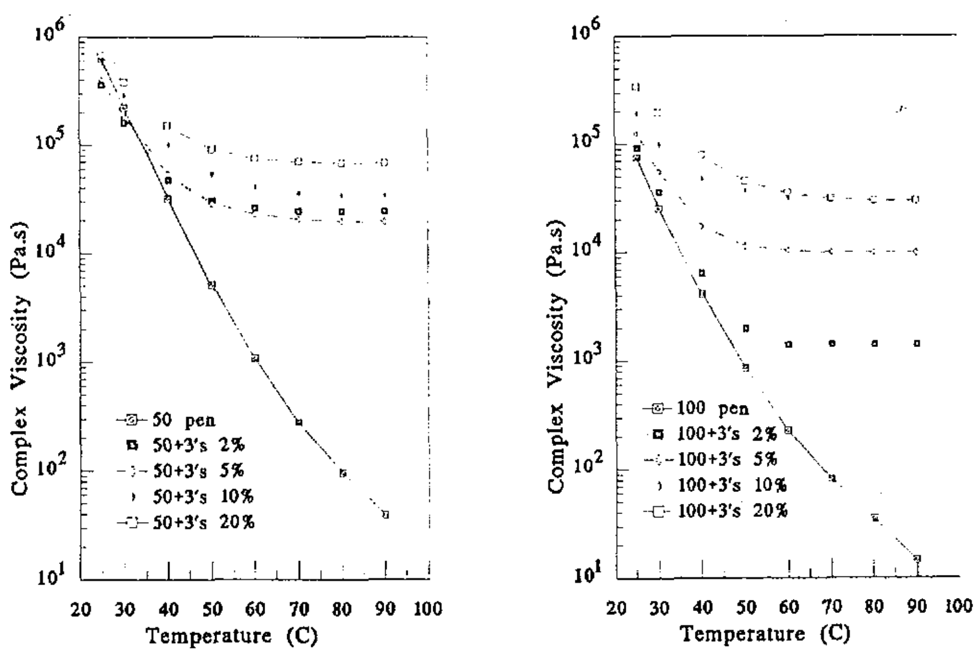


Figure 5.9. (d). Temperature vs. Complex Viscosity (μ*) (3's rubber size in 50 and 100 pen. grade base bitumen).

Carre and Laurent (1963) proposed relationships between standard penetration values and viscosity for penetration grade bitumen (these equations were given in Chapter 2.) The viscosity values calculated from equations (2.6) and (2.7), and the values measured using oscillatory shear analysis are given in Table 5.4. for comparison. The differences shown between measured and calculated values can be due to the rubber effect, the chemical composition of the binders or the frequency used. The frequency used in this study was 0.05 Hz.. A good relationship with a correlation coefficient of 85% was established between the viscosity values measured using the oscillatory shear viscometer and penetration values of the rubberised bitumen. Figure 5.10. shows the relationship between viscosity and penetration measured.

A linear relationship exist between the viscosity and penetration when plotted in log-log basis. This relationship is in the form of;

$$\log \eta = 3.01 - 2.10 \log \text{Pen} \qquad (5.12.)$$

It can be expressed also as below;

$$\eta = \frac{1.02 \times 10^{10}}{\text{Pen}^{2.10}} \qquad (5.13.)$$

This equation is slightly different than the equation given by Carre and Laurent (1963);

$$\eta = \frac{1.45 \times 10^{10}}{\text{Pen}^{2.15}} \qquad (5.14.)$$

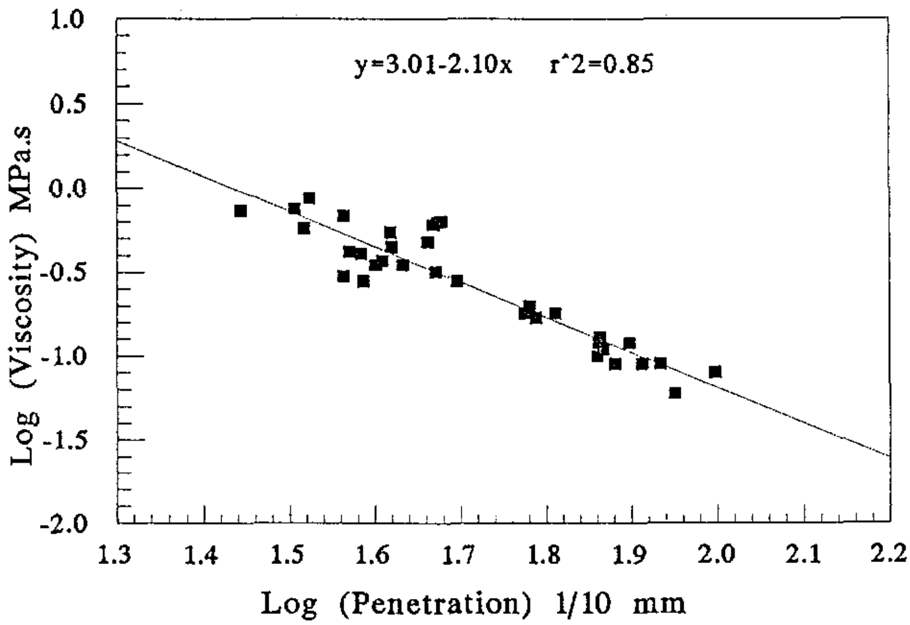


Figure 5.10. Viscosity vs. Penetration of rubberized bitumen investigated.

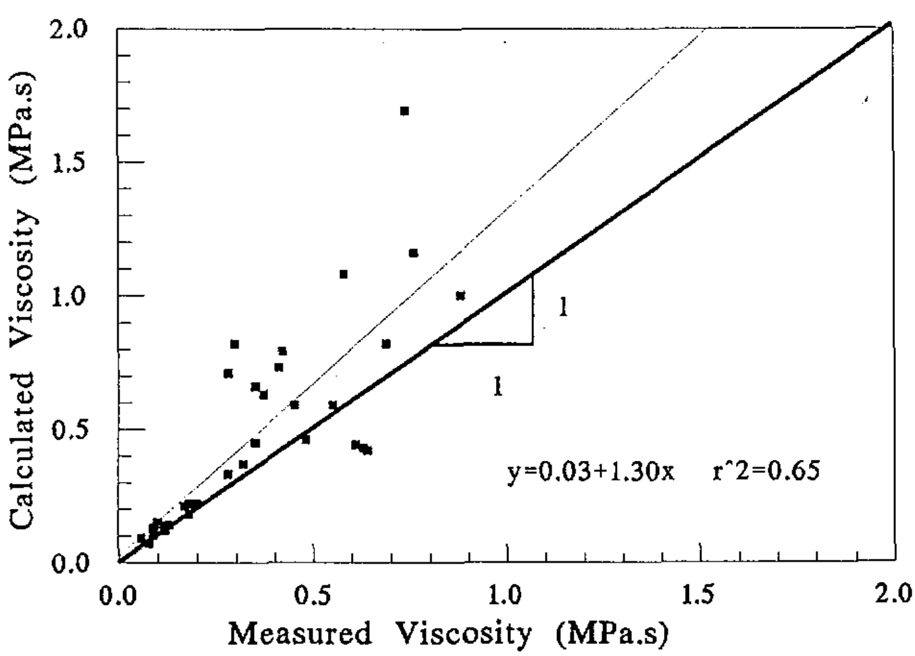


Figure 5.11. Measured Viscosity vs. calculated viscosity according to equations (2.6.) and (2.7.).

A graph, which shows the measured viscosity values against calculated viscosity values give an idea on the difference of viscosity with the addition of rubber. Figure 5.11. shows the measured viscosity values obtained from oscillatory shear analysis against calculated viscosity values using by equations (2.7.) and (2.6.). The measured values are lower than the calculated values. Viscosity of the bituminous binders depends upon the temperature changes and chemical compounds of the binders.

The rheological parameters of the 34 binders are plotted as a function of temperature for a given frequency of 0.05 Hz. The plots of loss modulus (G''), storage modulus(G') and dynamic viscosity (μ') versus temperature for the 50 and 100 pen base bitumen are given in Figure 5.12. and the rest is given in Appendix B.

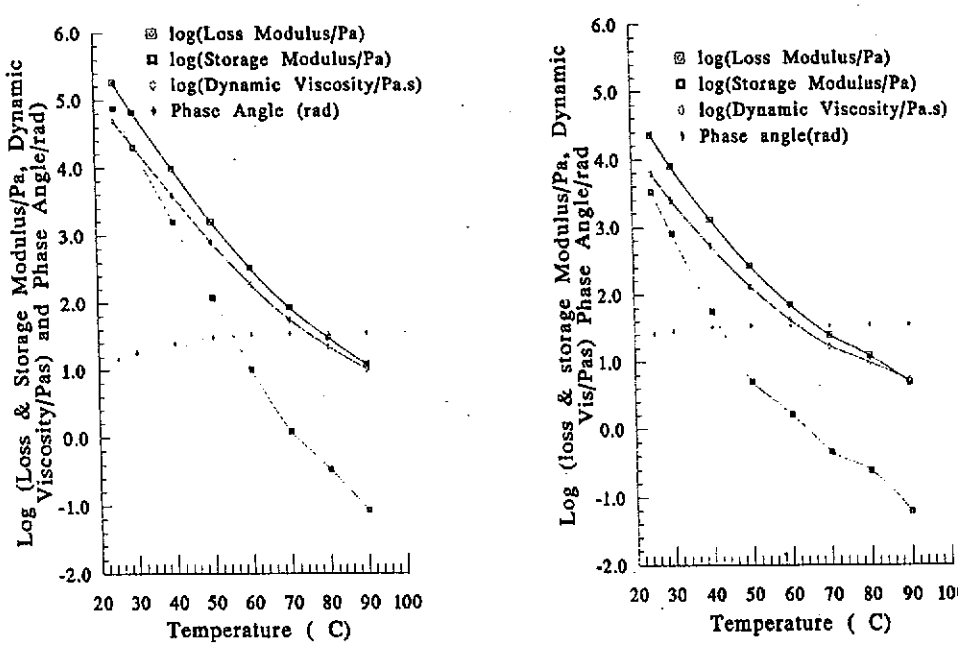


Figure 5.12. Temperature vs. Rheological Parameters for 50 (left) and 100 (right) pen. grade base bitumen.

Table 5.4. Variation on Viscosity With Varying Rubber Content at Penetration Temperature of 25 ° C.

Bitumen type	Pen. at 25° C	Viscosity in poise[1]	Viscosity in poise[2]	Viscosity MPa.s[1]	Viscosity MPa.s[2]	Viscosity MPa.s[3]
p50	47.50	3601444.62	4152601.15		0.42	0.64
24050	47.00	3684322.24	4268439.94		0.43	0.63
54050	46.50	3770024.33	4388801.39		0.44	0.61
104050	41.50	4814648.58	5899273.81		0.59	0.55
204050	33.83	7469245.06	10032915.57		1.00	0.88
304050	17.86	29510207.30	52846043.21		5.28	N/A
22650	45.86	3884570.15	4550566.08		0.46	0.48
52650	41.57	4796880.08	5872955.88		0.59	0.45
102650	37.14	6111334.99	7871324.28		0.79	0.42
202650	27.71	11469779.50	16853651.58		1.69	0.74
21250	39.86	5251456.24	6552508.32		0.66	0.35
51250	38.57	5635036.00	7135636.57		0.71	0.28
101250	36.57	6318483.60	8195106.46		0.82	0.30
201250	32.00	8419679.28	11596679.69		1.16	0.76
2350	40.57	5054688.03	6256779.14		0.63	0.37
5350	38.29	5725837.00	7274917.03		0.73	0.41
10350	32.86	7954519.31	10826451.52		1.08	0.58
20350	36.57	6318483.60	8195106.46		0.82	0.69
p100	99.43	735730.77	608407.40	0.07		0.08
240100	89.00	933640.08	811539.36	0.09		0.06
540100	81.71	1121830.71	1013325.18	0.11		0.09
1040100	72.43	1453990.85	1386621.13	0.15		0.10
2040100	59.57	2213279.54	2304755.00	0.22		0.18
226100	85.71	1012286.86	894922.15	0.10		0.09
526100	73.71	1400012.55	1324613.83	0.14		0.11
1026100	61.43	2071913.44	2127945.62	0.21		0.17
2026100	49.57	3285643.33	3716395.07	0.33		0.28
212100	79.00	1206340.07	1106351.60	0.12		0.12
512100	72.71	1441735.38	1372499.75	0.14		0.12
1012100	64.71	1852326.98	1858331.48	0.18		0.18
2012100	46.86	3708514.86	4302357.71	0.37		0.32
23100	76.00	1311048.56	1223512.09	0.13		0.09
53100	73.00	1429630.65	1358576.72	0.14		0.13
103100	60.29	2157282.49	2234426.83	0.22		0.20
203100	42.86	4492810.39	5425793.33	0.45		0.35

1 According to the equation (2.6)

2 According to the equation (2.7)

3 Measured at oscillatory shear viscometer (for frequency of 0.05 Hz).

5.3. Summary

The basic rheological requirements expected from a binder are that it readily deforms without rupture in order to resist thermal stresses due to volumetric changes, and that it must recover elastically, not by plastic flow from these small deformation in order to adsorb shock loads. The absence of fracture at low temperature is highly desirable, in addition it must limit permanent deformation at high temperatures.

Penetration test shown that hardness of base bitumen increases proportionally with the addition of shredded rubber (Figure 5.1.). It was found that the slope of the relationship between penetration and rubber content depends on the rubber size and mixing temperature of the base bitumen. The mixing time affects the penetration value due to the degree of digestion obtained and to the interaction between rubber particles and bitumen.

The changes on the softening point test with the addition of rubber show some similarity to the penetration test. The softening point of the binder increases linearly with the increasing in shredded tire rubber concentration (Figure 5.3.). The softening point also increases with the increasing mixing time in a logarithmic relation (Figure 5.4.). The optimum mixing time found was 0.5 hour.

The elastic recovery of the shredded rubber modified bitumen increases with increasing rubber content but not ductility. It may be due to the absence of digestion (Table 5.3.).

The oscillatory shear analysis showed that the elastic components of the viscosity of shredded rubber modified binders increases with the increasing rubber content. The viscosity increased for all rubberised bitumen composition used in this investigation when compared with the base bitumen.

CHAPTER SIX

ENGINEERING PROPERTIES OF ASPHALTIC CONCRETE MADE WITH WASTE THREAD TIRE RUBBER MODIFIED BINDERS

6.1. Introduction

As it has been shown in Chapter 5 the addition of waste shredded rubber alters the rheological properties of bitumen. These changes are studied in this chapter with relation to the changes that they may produce on the properties of asphaltic concrete.

The main concern during this part of the investigation will be the effects of varying rubber concentration and rubber size on the engineering properties of asphaltic concrete. The discussions on the laboratory results, which were obtained in accordance with Leeds Design Methods, include the following properties;

1-workability,

2-density of the compacted mixture,

3-compacted aggregate density,

4-Marshall stability,

5-Marshall flow,

6-porosity,

7-voids in the mineral aggregates,

8-air permeability,

9-creep stiffness,

10-optimum binder content.

In addition to the codes used in Chapter 5 the codes used in the Figures includes numbers 1, 2, 3 and 4. They represent the following;

1- Aggregate gradation after DCT mixed with 50 pen. grade base bitumen,

2- Aggregate gradation according to equation (3.1.) (n=0.5) mixed with 50 pen. grade base bitumen,

3- Aggregate gradation after DCT mixed with 100 pen. grade base bitumen,

4- Aggregate gradation according to equation (3.1.) (n=0.5) mixed with 100 pen grade base bitumen.

For example, 2401 shows the asphaltic concrete made with 2% 40's size rubber and 20261 shows the asphaltic concrete made with 20% (26) 12/16's size rubber mixed with 50 pen. base bitumen and aggregate gradation type DCT result.

6.2. Workability of the Mixtures

The workability of bituminous mixtures is very difficult to quantify. It is influenced mainly by aggregate type (shape, size and texture), aggregate gradation, binder grade, binder content, binder viscosity and mixing and compaction temperature (Napiah, 1993). An assessment of workability is not included in any current UK design procedure.

Cabrera (1991) defines the workability of bituminous mixtures as the property which allows to produce, handle, place and compact a mix with minimum application of energy. It is an important practical requirement that bituminous materials are tolerant to a wide variation of laying conditions and that in relatively adverse condition, they will still spread easily and compact sufficiently well to give a durable surfacing. Too harsh a mix, may cause it to tear under the screed of the spreading machine and render compaction more difficult. Badly compacted materials will not develop the desirable properties of high stiffness, good fatigue resistance, good durability. To account for its importance in highway construction, the Leeds Design Method has included an assessment of workability in its design procedure (Cabrera, 1992).

The method to measure workability is based on experimental relations between compaction energy and mix porosity. The Leeds Workability Methods employs the Gyratory Testing Machine (GTM) and its determination is in conjunction with specimen compaction. The GTM is equipped with a dial gauge to continually monitor the changes in specimen height as it is being compacted. A detailed information can be found in reference (Cabrera, 1991).

The workability of the mixtures is expressed by the Workability Index (WI). WI can be used to effectively asses the influence of mix composition and other variables. Generally, for any hot mixed bituminous material, the workability index of the mixture increase with increasing binder content and compaction temperature.

The results of workability index against binder content of asphaltic concrete made with base bitumen and rubberised bitumen are plotted in Figures 6.1.(a), (b), (c), (d). The workability index increases as the bitumen content increases. A straight line with a correlation of about 99% can be fitted to the results obtained in this work for all the asphaltic concrete. The linear relationship is in the following general form;

$$WI = a + b \times BC \tag{6.1.}$$

where;

WI, workability index,

a, b, constants,

BC, binder content (% weight).

The addition of shredded waste rubber decreases the workability index of asphaltic concrete as it is shown by the constant b of the equations, i.e. the slope of the function decreases with increasing rubber content. The reduction of WI is greater for the mixes made with the 100 pen. base bitumen. It is also clear that the larger the size of the shredded waste rubber the larger the effect on WI as shown in Figures 6.1. (a), (b), (c) and (d). For instance, when the WI value was 10 for the asphaltic concrete made with 50 pen. base bitumen with 7 % binder content this value decreased gradually as the increasing rubber content up to 7 for the mixture made with 50 pen. grade base bitumen plus 20% 40's size shredded rubber modified binder (Figure 6.1. (a) (left)). Increase in rubber size also decreased the Workability Index (WI). WI was found below 4 when the mixture made with 50 pen. base bitumen plus the biggest size (3's) rubber (Figure 6.1. (d) (left)). Similar findings was observed when the mixture was made with 100 pen. base bitumen modified with shredded rubber.

The hardness of the base bitumen has also an effect on the workability of bituminous mixture. At the same mixing temperature, the constant **a** of 50 pen. base bitumen is lower than 100 pen. base bitumen. These results show that the viscosity of the binder has a large influence on the workability of mixtures.

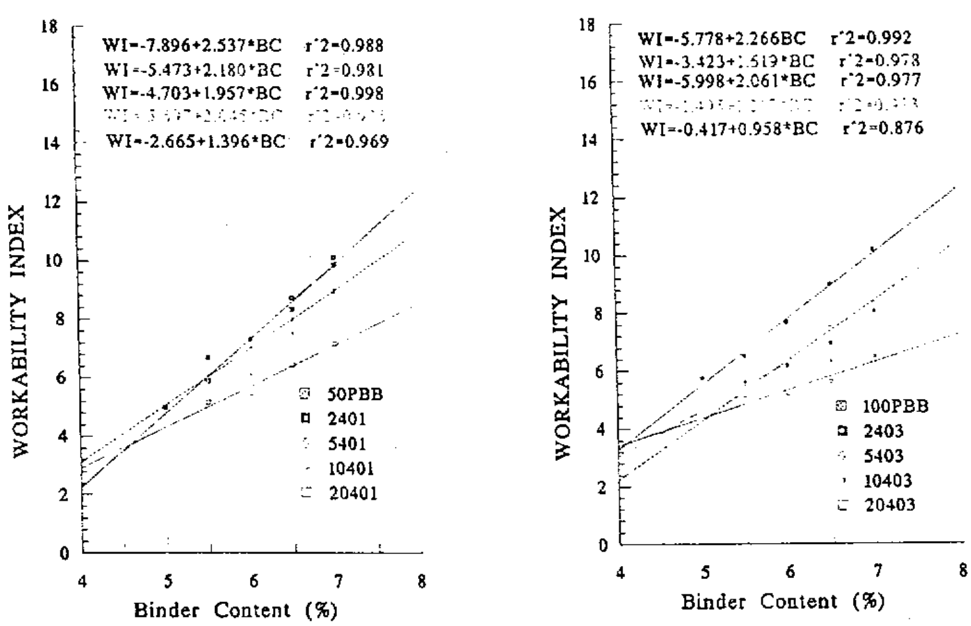


Figure 6.1. (a). WI vs. Binder Content for rubberised asphaltic concrete made with 40's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) grade base bitumen.

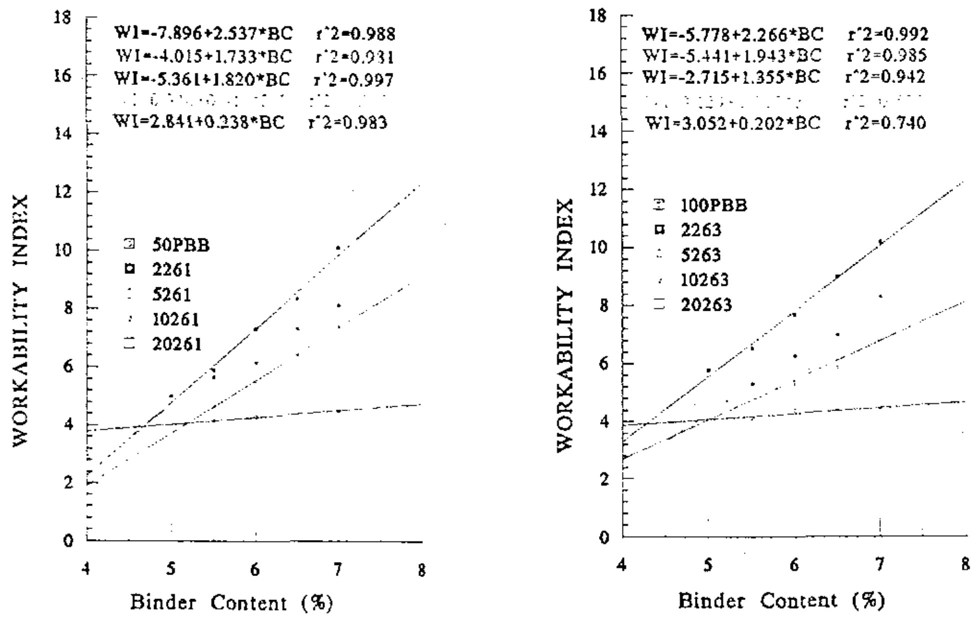


Figure 6.1. (b). WI vs. Binder Content for rubberised asphaltic concrete made with 12/16's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) base bitumen.

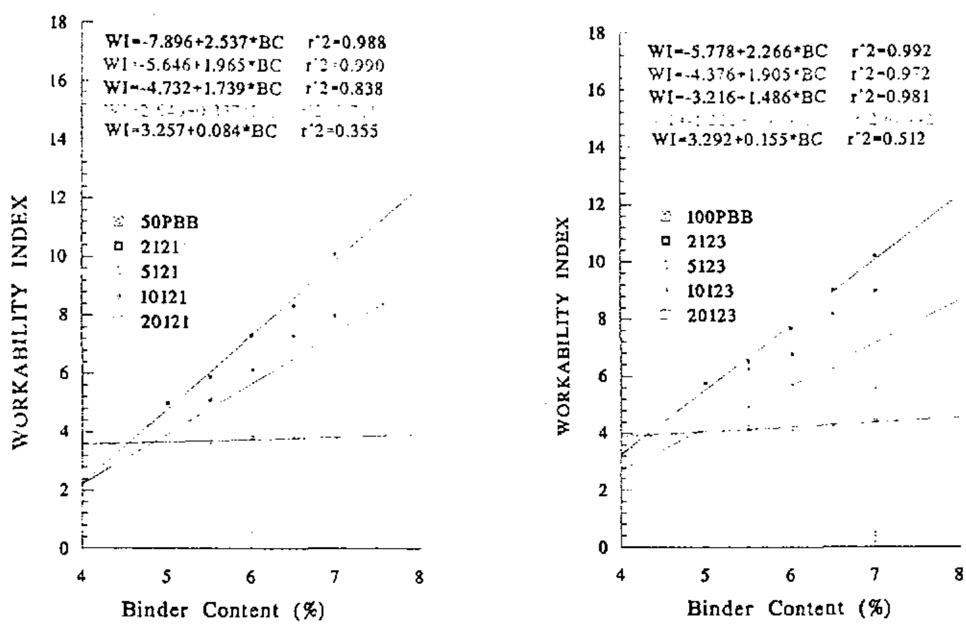


Figure 6.1. (c). WI vs. Binder Content for rubberised asphaltic concrete made with 12's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) grade base bitumen.

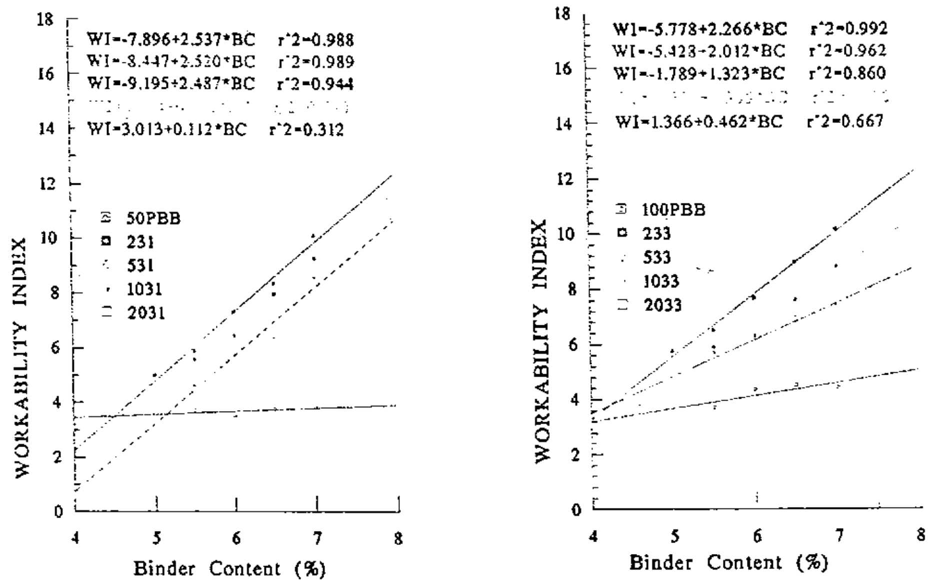


Figure 6.1. (d). WI vs. Binder Content for rubberised asphaltic concrete made with 3's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) grade base bitumen.

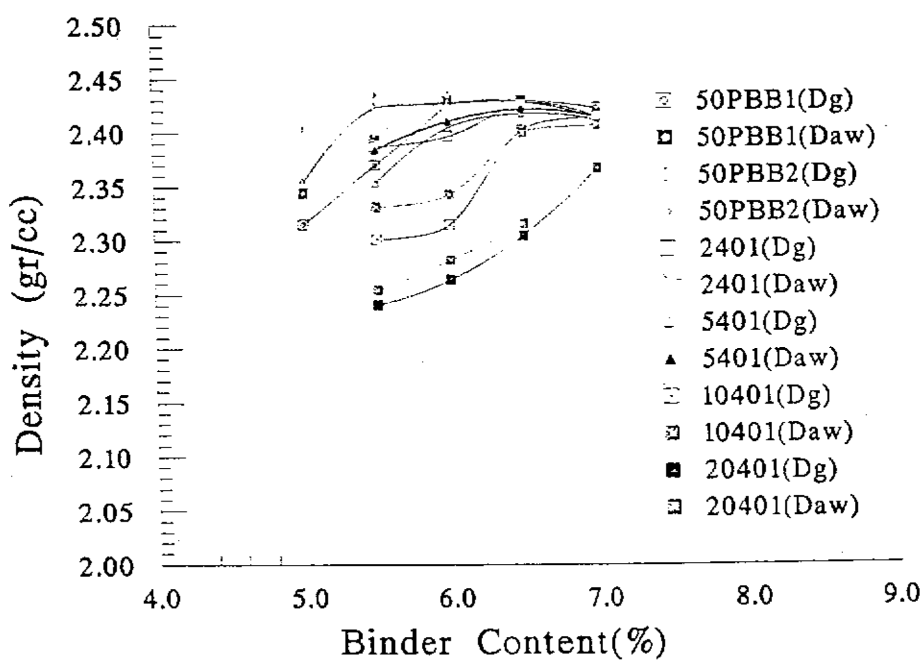
6.3. Density of the Compacted Mixture

Generally, the density of a bituminous mixture increases with increasing binder content up to a certain value beyond which it then starts to decrease. At low binder content, the binder acts as a lubricant and promotes the aggregate particles to slide over each other with ease. After an optimum amount of binder has been reached it only acts to displace the aggregate particles and since the specific gravity of the binder is less than half of the specific gravity of the aggregates, the final density of the compacted mixture starts to decrease.

The effect of variables investigated on the densities of asphaltic concrete is given in Figure 6.2.. Daw and Dg values were calculated as indicated in Chapter 4 equations (4.3.) and (4.4.) respectively. Daw values were found usually higher than Dg values because of small holes on the surface of the samples and the Dg values were taken into account in the rest of the study.

One of the variables investigated on the density of asphaltic concrete was the aggregate gradation. As mentioned earlier, two aggregate gradations, i.e. according the equation (3.1.) and based on the minimum VMA obtained using by DCT were used during this research. For both aggregate gradations the density values were found very close. The density values for both mixes at maximum binder content were 2.43 gr/cm^3 when the mixture made with 50 pen. base bitumen (the one according to equation (3.1.) is slightly over) and the density value found for the aggregate gradation obtained after DCT was 2.45 gr/cm^3, this value was 2.40 gr/cm^3 for the aggregate gradation obtained by using equation (3.1.) made with 100 pen. base bitumen. The other important differences observed between the used two aggregate gradation were the binder content which gives the maximum density. The aggregate gradation obtained by using equation (3.1.) is given lower binder content for maximum density than the other one.

The effect of rubber on the densities of the compacted mixtures obtained at various binder content, binder type, rubber sizes and rubber content are plotted in Figures 6.2. (a), (b), (c) and (d). As shown in these Figures the addition of rubber causes a decrease in the density of a compacted mixture. In both mix types, i.e. those made with 50 pen. and 100 pen base bitumen, rubber significantly affects the compacted mixture density. For example, 5% decrease in density was observed on the mixture made with 50 pen. base bitumen plus 20% 40's size rubber (6.2. (a). (left)). This decrease reaches up to 12% for the mixture made with 3's size plus 20% rubber content made with 50 pen base bitumen (6.2.(d). (left)).



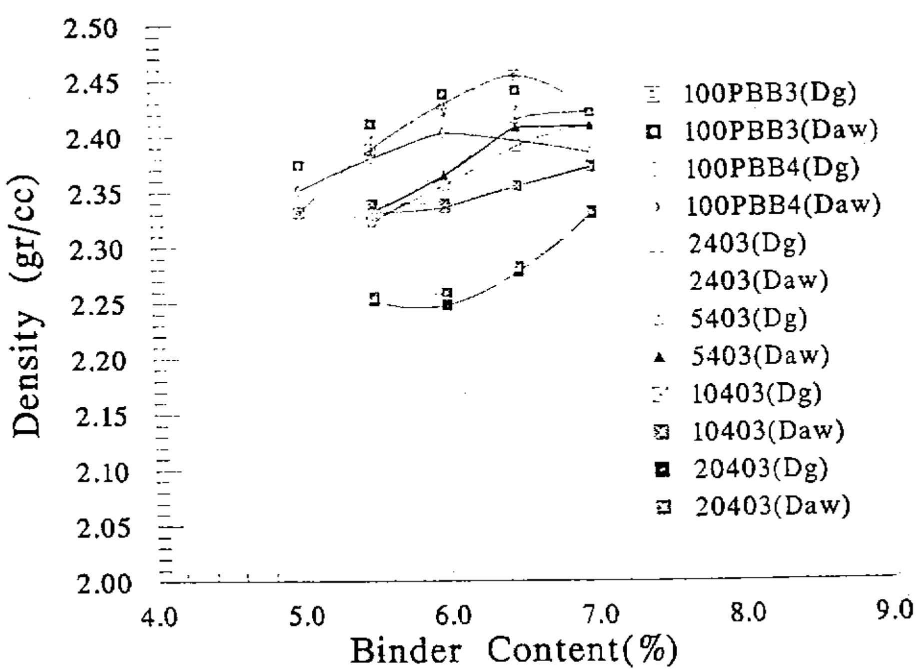
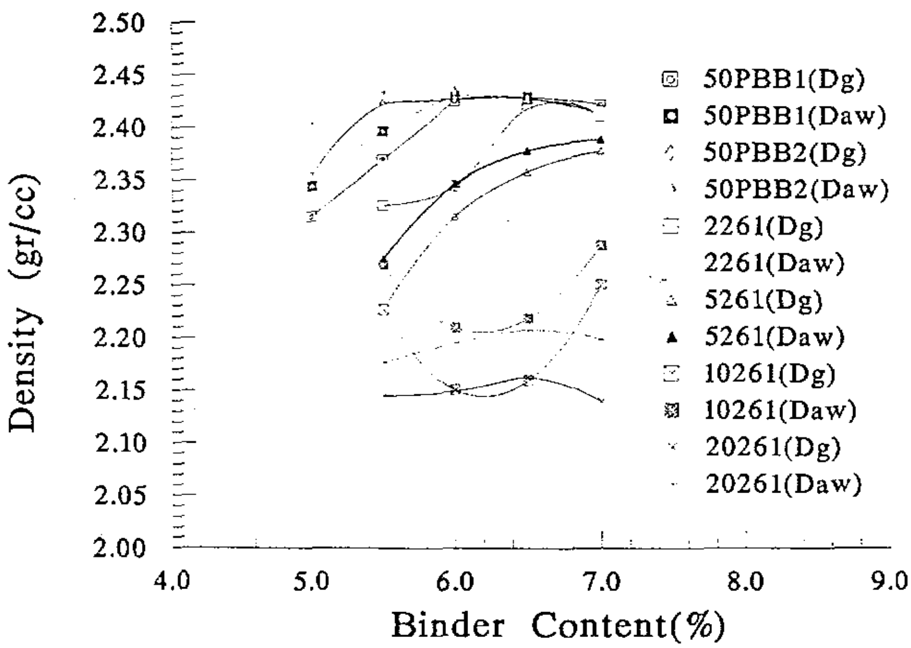


Figure 6.2. (a). Density vs. Binder Content for rubberised asphaltic concrete made with 40's size shredded tire rubber in 50 pen.(above) and 100 pen.(below) grade base bitumen.



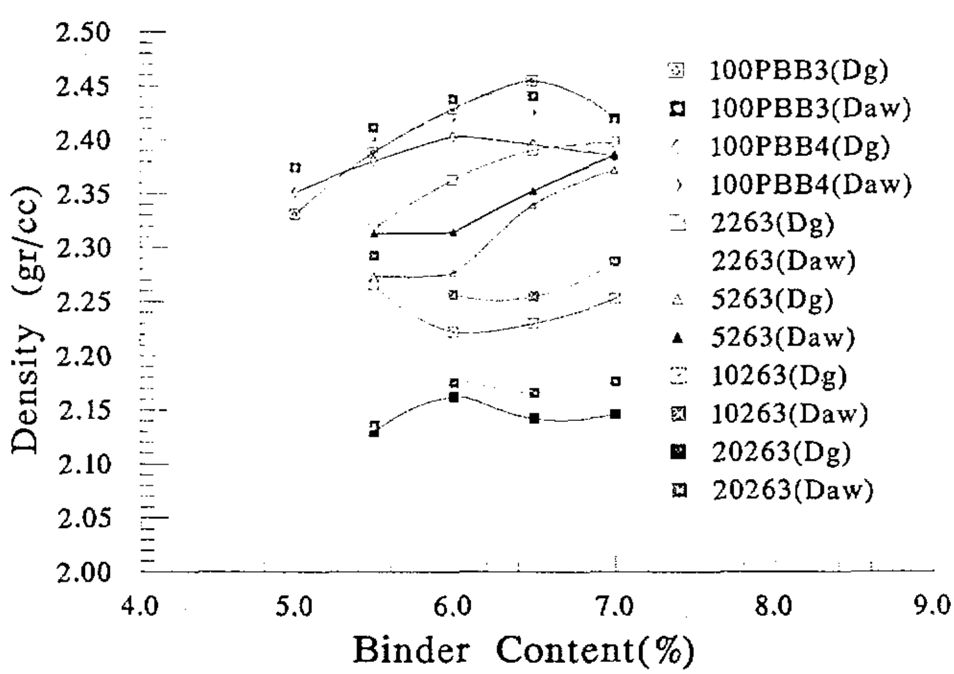
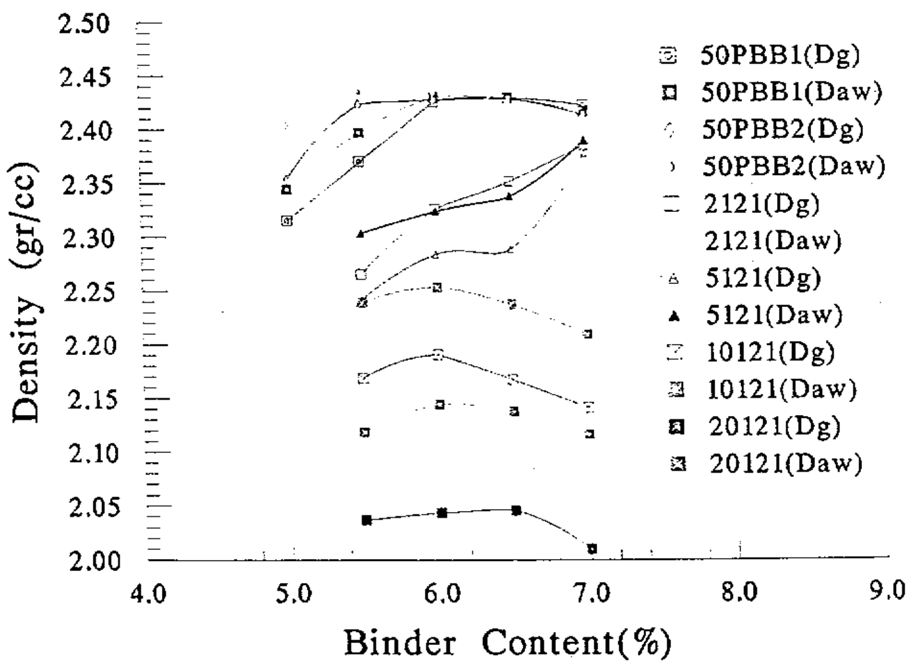


Figure 6.2. (b). Density vs. Binder Content for rubberised asphaltic concrete made with 12/16's size shredded tire rubber in 50 pen.(above) and 100 pen.(below) base bitumen.



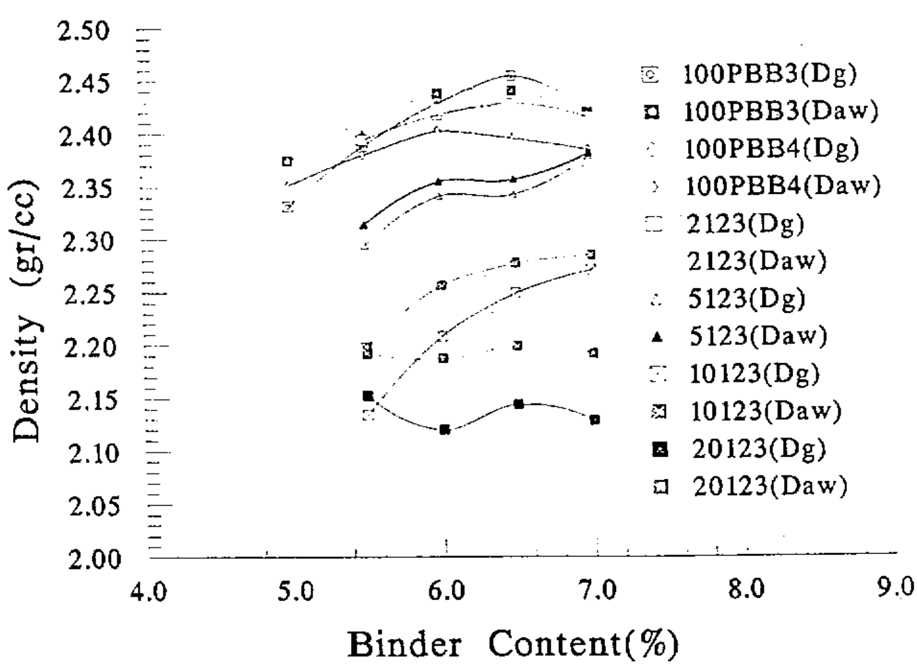
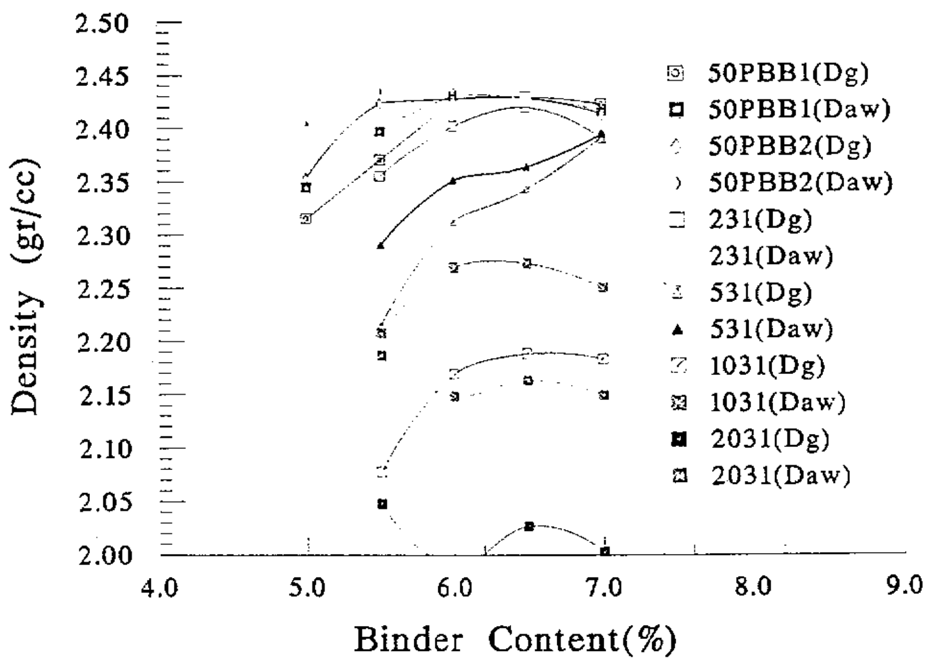


Figure 6.2. (c). Density vs. Binder Content for rubberised asphaltic concrete made with 12's size shredded tire rubber in 50 pen.(above) and 100 pen.(below) grade base bitumen.



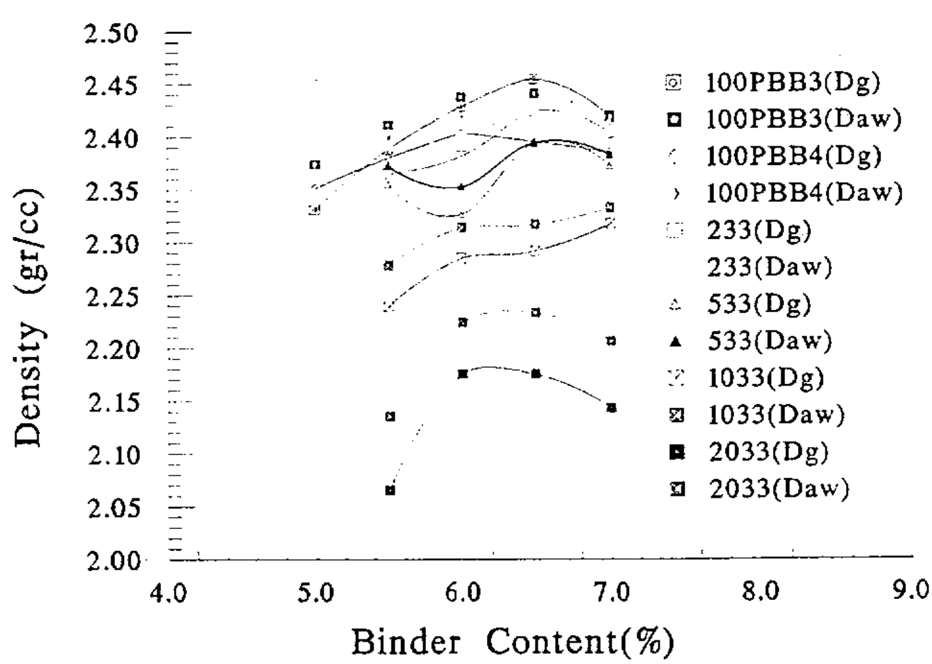


Figure 6.2. (d). Density vs. Binder Content for rubberised asphaltic concrete made with 3's size shredded tire rubber in 50 pen. (above) and 100 pen. (below) grade base bitumen.

It is known that rubber has higher elasticity. During compaction, the modified binder stores energy on the rubber particles inside the bitumen, this energy is released during cooling by expansion of the specimen. This increase in volume causes to the reduction in density. An important factor is the reduced workability which lead to lower density. The decrease in density of the compacted mixture means increase on the porosity.

A thorough study of the porosity of the mixture is given in Chapter Seven.

6.4. Compacted Aggregate Density

The Compacted Aggregate Density (CAD) and Voids in Mixed Aggregate (VMA) vs. binder content graphs of the bituminous mixtures show opposite trends because these two parameters are complementary. When equations (6.2.) and (6.3.) are analysed in detail it can be seen that, the binder content which gives maximum CAD value will be the same for minimum VMA. Figure 6.3. explains this idea. Therefore, in this research CAD and VMA of the asphaltic concrete was investigated and CAD was taken into account in the design of asphaltic concrete made with waste shredded tire rubber.

$$VMA = (100 - \frac{P_{wa} \times D}{Sg_{mix}}) \qquad (6.2.)$$

where;

P_{wa}, percentage weight of aggregate (%),

D, density of the compacted mixture (gr/cm^3),

Sg_{mix}, Specific Gravity of mixture (gr/ cm^3).

The Compacted Aggregate Density also has been defined as;

$$CAD = (P_{wa} \times D) / 100 \qquad (6.3.)$$

where;

P_{wa}, percentage weight of aggregate (%),

D, density of the compacted specimen (gr/cc).

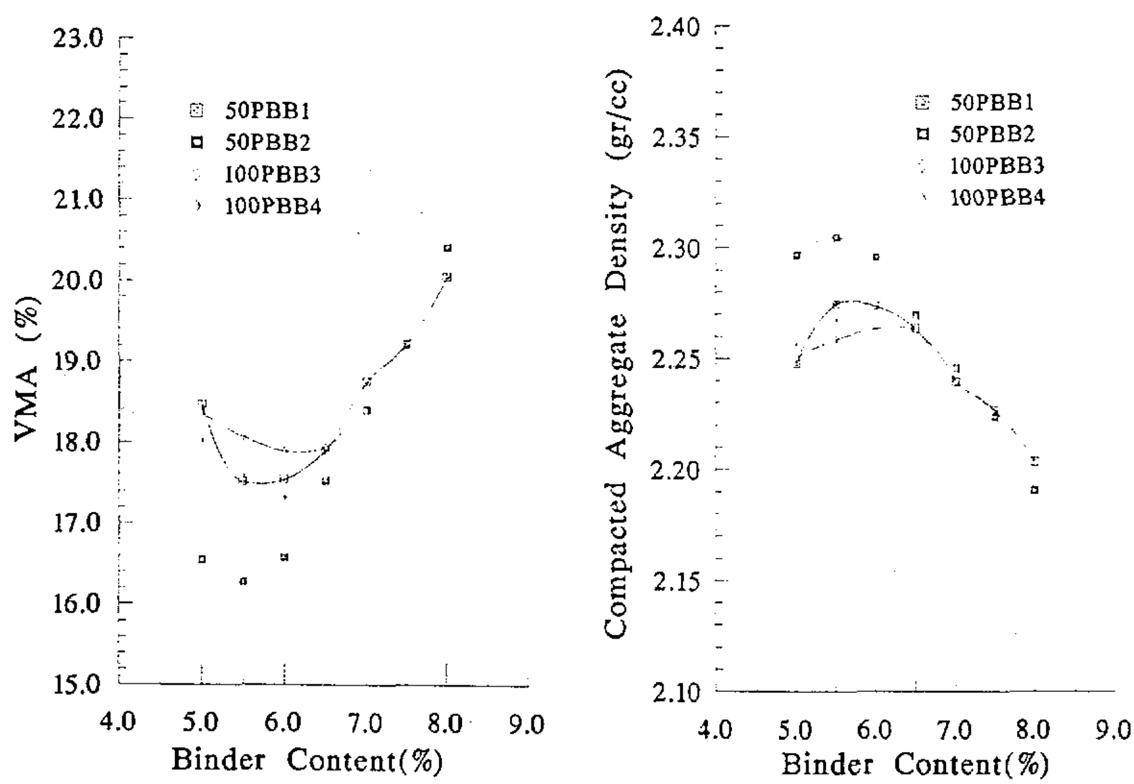


Figure 6.3. VMA and CAD vs. Binder Content for asphaltic concrete investigated.

The effect of rubber on the relationship between compacted aggregate density and binder content of the mixtures is given in Figure 6.4.(a), (b), (c) and (d). The results clearly show a significant effect of rubber on CAD. All shredded rubber sizes tested decrease the CAD values. The higher the rubber size the lower the CAD. A reduction in the CAD values was due to the lower porosity of the compacted specimen. The effect of shredded rubber concentration was also significant for all sizes tested. The increase on rubber concentration lead to further decrease on CAD of values.

As it is shown in the Figures increasing rubber content decreases the CAD values of the asphaltic concrete. In Figures 6.4.(b), 6.4.(c) and 6.4.(d) the CAD values are to low for the 20% rubber content and in Figure 6.4.(b).(left and right) the trend looks like opposite way for the 20% rubber content. However, these lines show that the CAD values will increase when the binder content increased. As a result of that it can be said that increasing rubber content increases the optimum binder content.

The effect of aggregate gradation on CAD values of asphaltic concrete is also given in Figure 6.4.. The differences between two aggregate gradations are remarkable on CAD values of asphaltic concrete made with 50 pen. base bitumen, this values were found very close for the mixtures made with 100 pen. base bitumen. When the CAD value was 2.31 gr/cm^3 for aggregate gradation according to equation (3.1.) at maximum point, this value was 2.27 gr/cm^3 for the aggregate gradation obtained after DCT for the mixture made with 50 pen. base bitumen, this value found was approximately 2.27 gr/cm^3 for both aggregate gradation made with 100 pen. base bitumen (the one according to equation (3.1.) is slightly over).

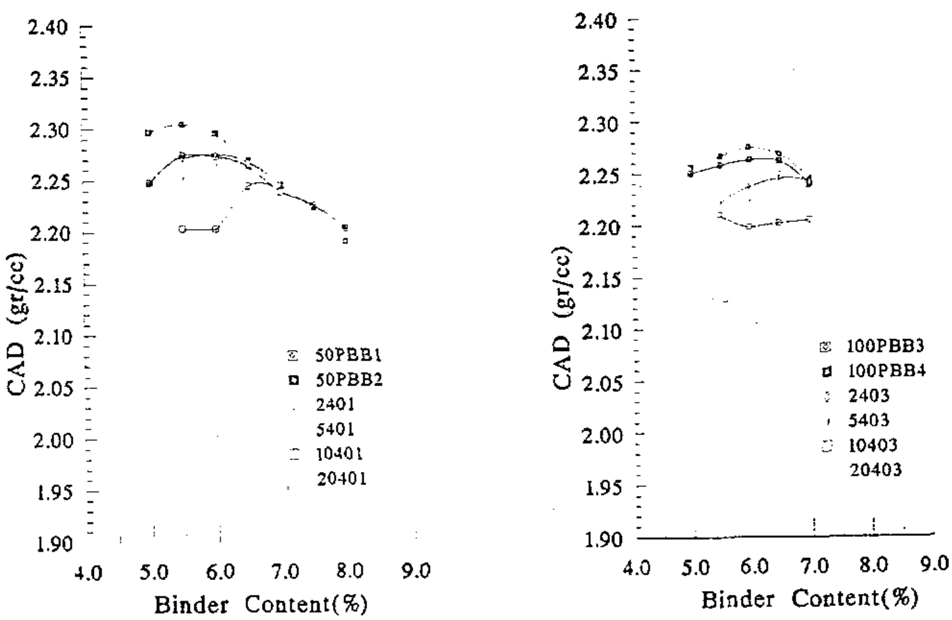


Figure 6.4. (a). CAD vs. Binder Content for rubberised asphaltic concrete made with 40's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) grade base bitumen.

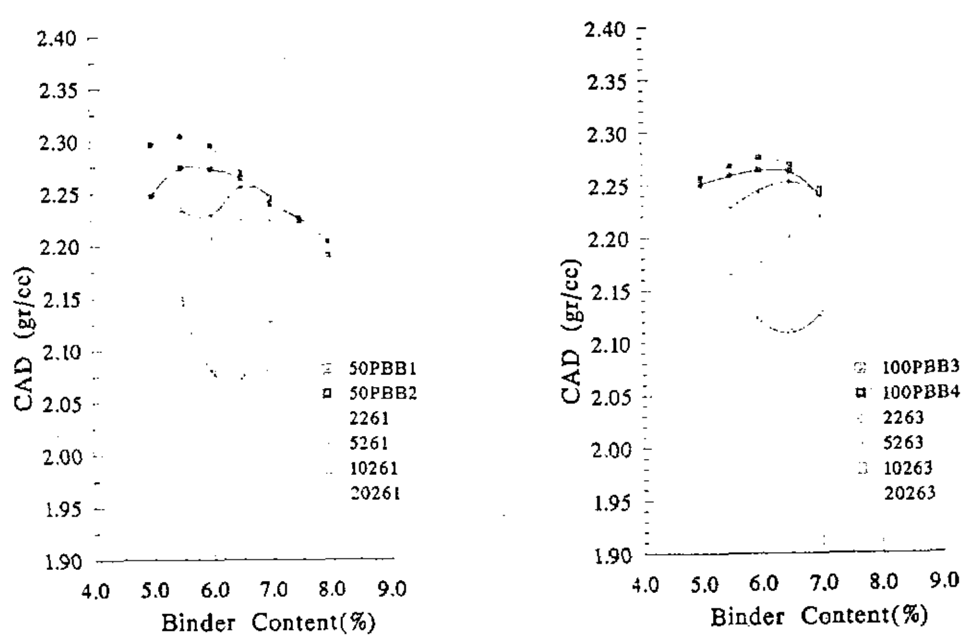


Figure 6.4. (b). CAD vs. Binder Content for rubberised asphaltic concrete made with 12/16's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) grade base bitumen.

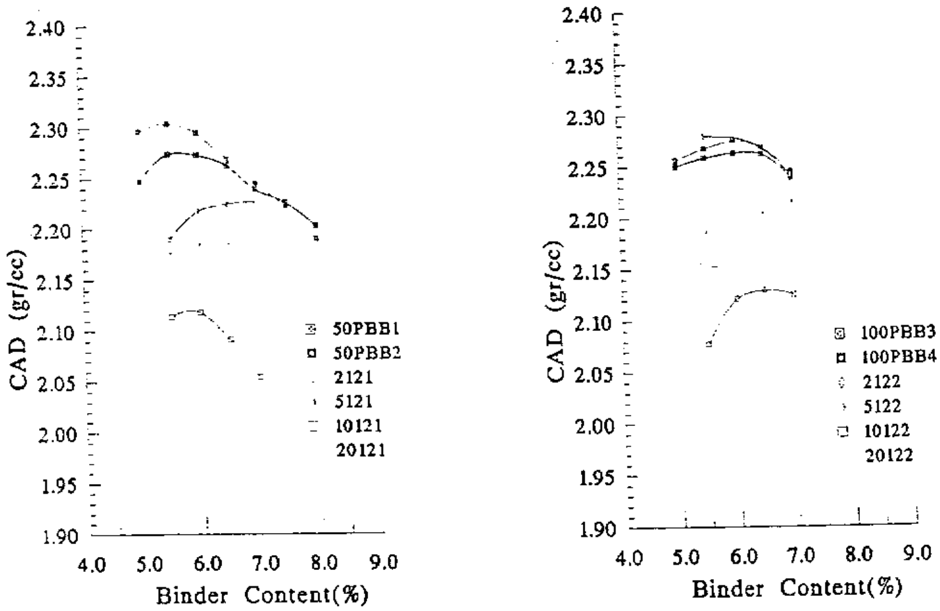


Figure 6.4. (c). CAD vs. Binder Content for rubberised asphaltic concrete made with 12's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) grade base bitumen.

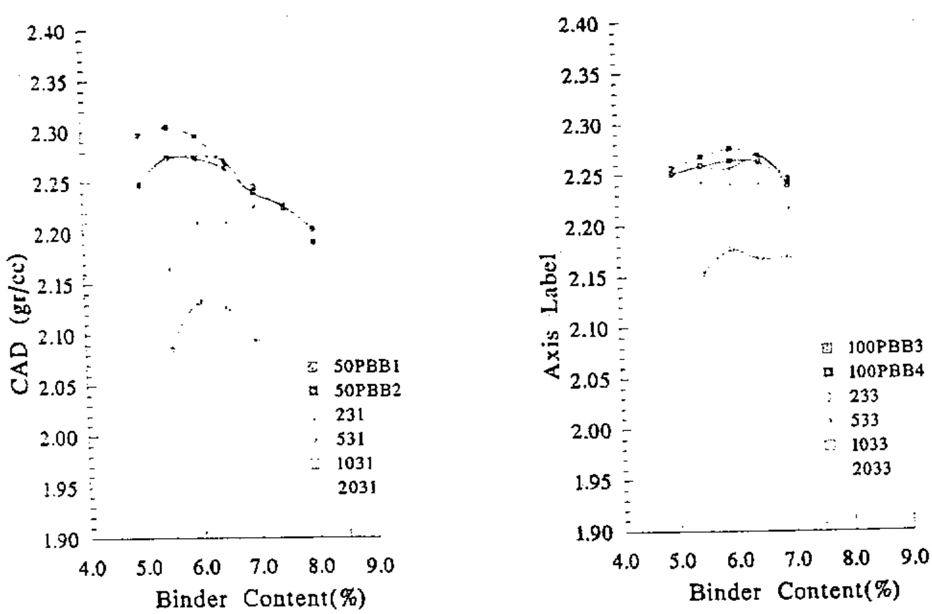


Figure 6.4. (d). CAD vs. Binder Content for rubberised asphaltic concrete made with 3's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) grade base bitumen.

6.5. Porosity of the Compacted Mixture

Porosity, (P) is determined as the ratio of the volume of porous divided by the total volume of a mixture. It is generally expressed as a percentage of the bulk volume of the compacted mixture. Chapter 7 deals with porosity measurements and porosity characteristics of the asphaltic concrete studied.

6.6. Voids in Mixed Aggregate (VMA)

Voids in Mixed Aggregate, VMA, are defined as the intergranular void space between the aggregate particles in a compacted mixture that includes the porosity and binder content, expressed as a percentage of the total volume. According to Khandal *et al.* (1985) the VMA should be considered as the most important mix design parameter affecting mix durability. High VMA values allow enough binder to be incorporated into the mix to obtain maximum durability without the mix flushing. Such mixes have lower stiffness modulus at low temperatures which is helpful in minimizing the thermal and reflection cracking (Khandal *et al.*, 1985). Maximum deformation resistance requires minimum VMA (Cooper *et al.*, 1985). Cooper *et al.*(1985) has worked on three main aggregate gradation, which are namely Hot Rolled Asphalt (HRA), Dense Bitumen Macadam (DBM) and Open-textured Bitumen Macadam, and they have found a good relationship between VMA and permanent deformation.

The effect of rubber on VMA of compacted asphaltic concrete mixtures is illustrated in Figures 6.5. (a), (b), (c) and (d). Usually, the VMA decreases as the binder content increases until it reaches a minimum value, after which the VMA increases as the binder content increases. An increase has been observed with increasing rubber content on the VMA of asphaltic concrete.

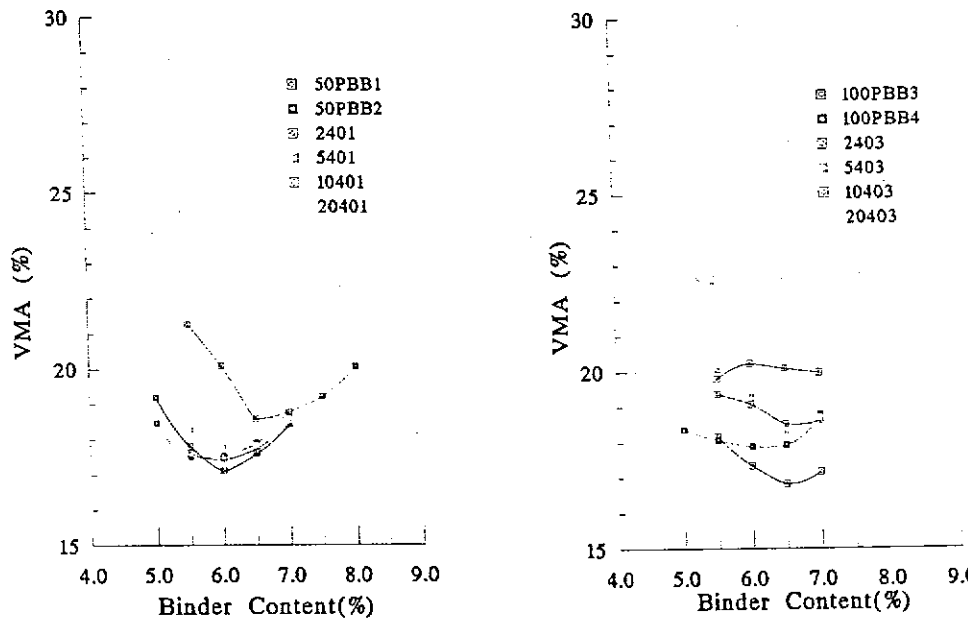


Figure 6.5. (a). VMA vs. Binder Content for rubberised asphaltic concrete made with 40's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) grade base bitumen.

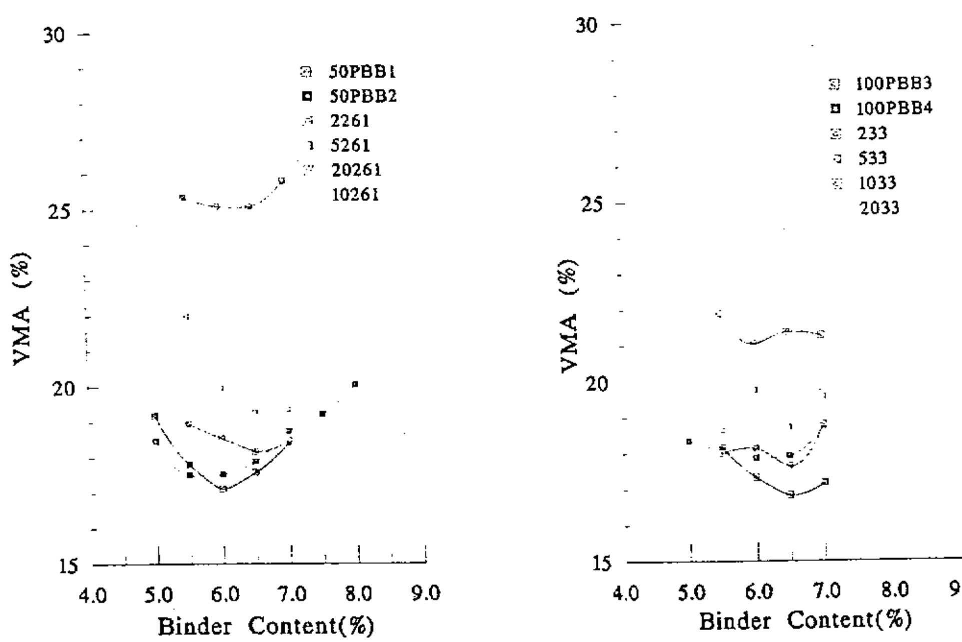


Figure 6.5. (b). VMA vs. Binder Content for rubberised asphaltic concrete made with 12/16's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) grade base bitumen.

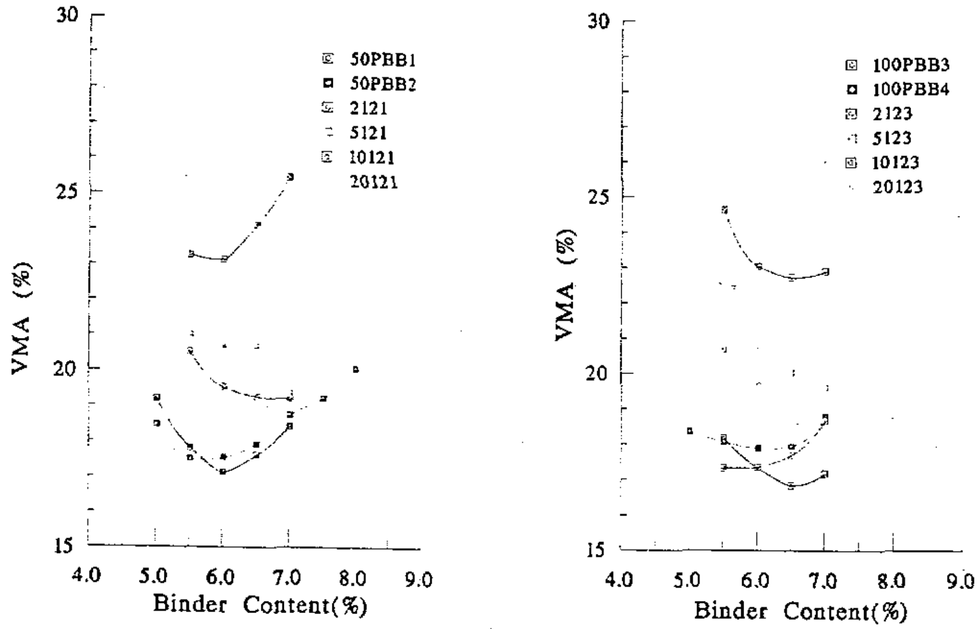


Figure 6.5. (c). VMA vs. Binder Content for rubberised asphaltic concrete made with 12's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) grade base bitumen.

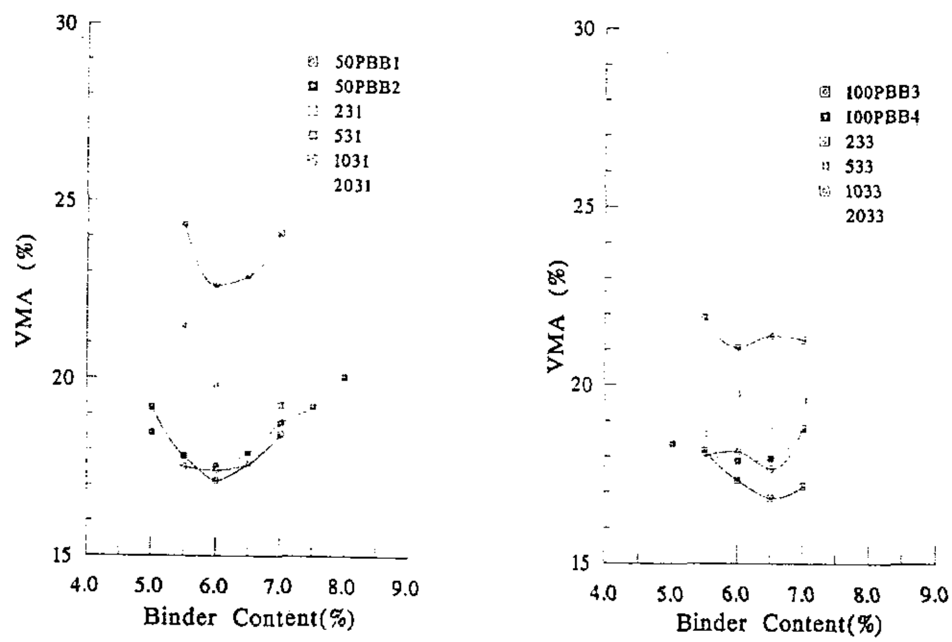


Figure 6.5. (d). VMA vs. Binder Content for rubberised asphaltic concrete made with 3's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) grade base bitumen.

6.7. Marshall Stability and Flow

Stability is an arbitrary value related to strength of bituminous mixture and its ability is to resist deformation under imposed loads. The Marshall Stability test procedure is given in Chapter 4.

The maximum Marshall Stability of a mixture depends on the binder content, as the binder content increases, stability increases up to a maximum value beyond which it starts to decrease. In general, the deformation (Marshall Flow) increases as the binder content increases.

The stability values of rubberised mixtures are plotted against binder content as shown in Figure 6.6. (a), (b), (c) and (d). The addition of small proportion of rubber increases the Marshall stability of the mixtures especially for mixtures made with low penetration grade base bitumen. The increase on stability is clear for the rubberised mixtures made with 100 pen. base bitumen for all rubber sizes. The stability values decrease with increasing rubber sizes and rubber content for the mixtures made with 50 pen. base bitumen. Only a small increase is observed for 40's size of rubber at lower content. In spite of high porosity, the reduction in stability is not proportional.

Figures 6.7. (a), (b), (c) and (d) show the results of Marshall flow for the mixtures made with rubber. The addition of rubber decreases the Marshall flow. A large decrease is observed at higher rubber content and large rubber sizes for the mixtures made with 100 pen. base bitumen. The change on Marshall flow is not significant on the rubberised mixture made with 50 pen base bitumen. While the mixes made with small size of rubber show a decrease on Marshall flow (Figure 6.7. (a)), as the rubber size increases the Marshall flow tends to increase (Figure 6.7.(b), (c) and (d)). Indeed, it increases with increasing rubber content until reaching a certain amount of rubber content, after which it starts to decrease. This can be seen clearly in Figure 6.7. (c) (left).

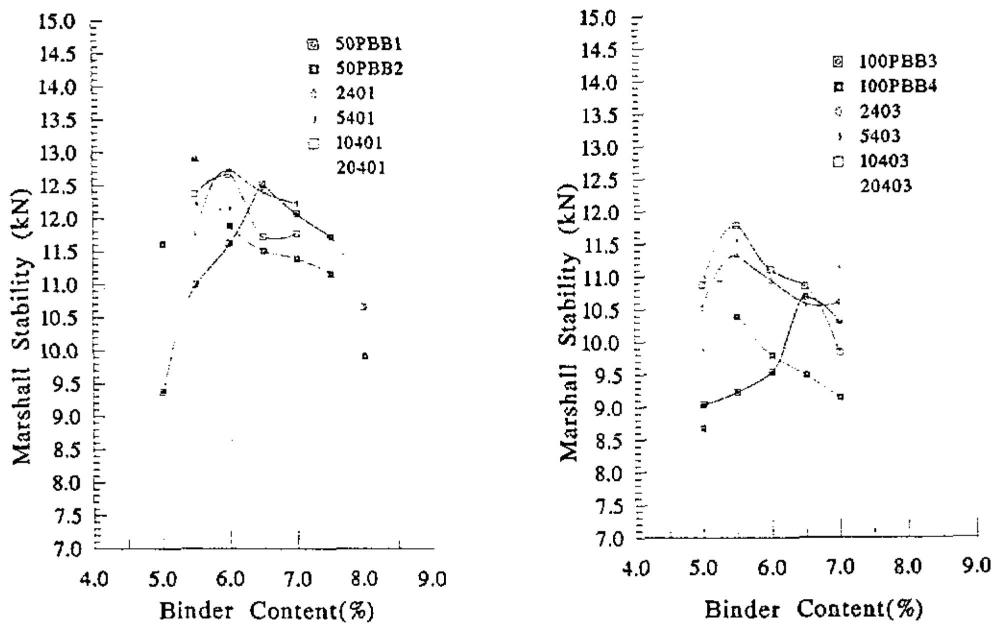


Figure 6.6. (a). Marshall Stability vs. Binder Content for rubberised asphaltic concrete made with 40's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) base bitumen.

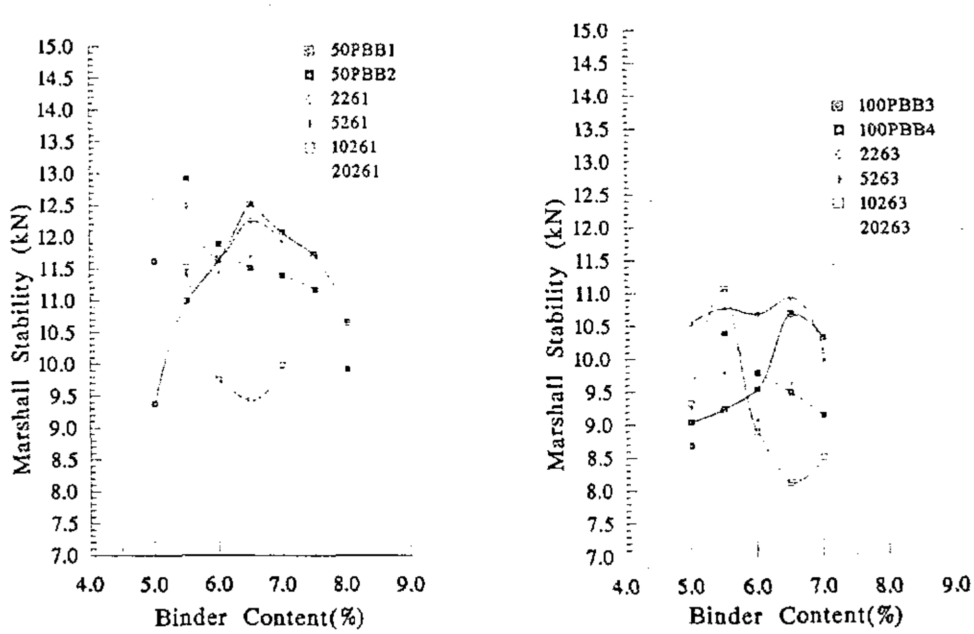


Figure 6.6. (b). Marshall Stability vs. Binder Content for rubberised asphaltic concrete made with 12/16's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) bitumen.

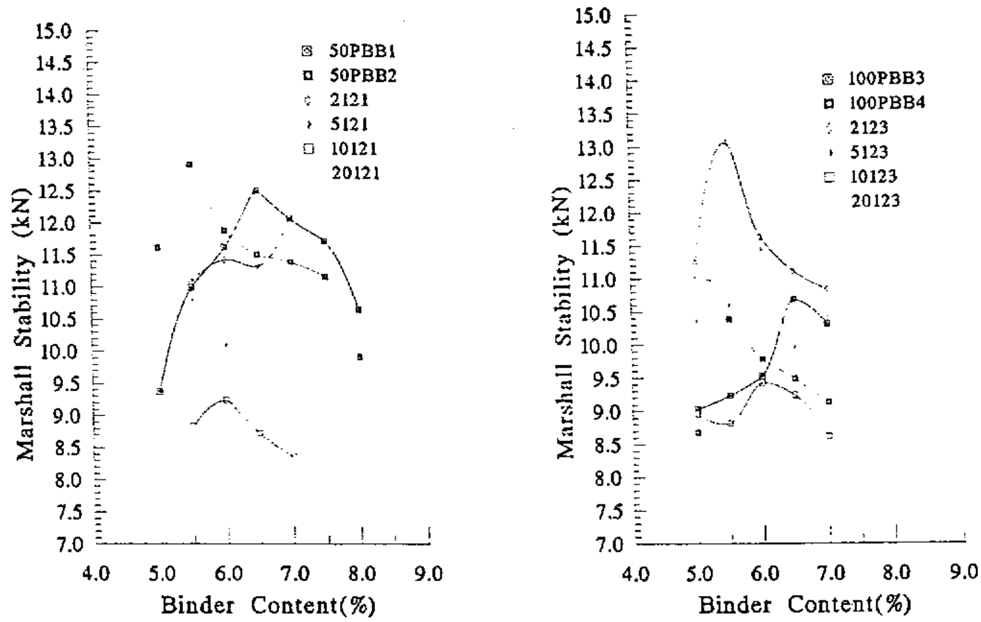


Figure 6.6. (c). Marshall Stability vs. Binder Content for rubberised asphaltic concrete made with 12's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) base bitumen.

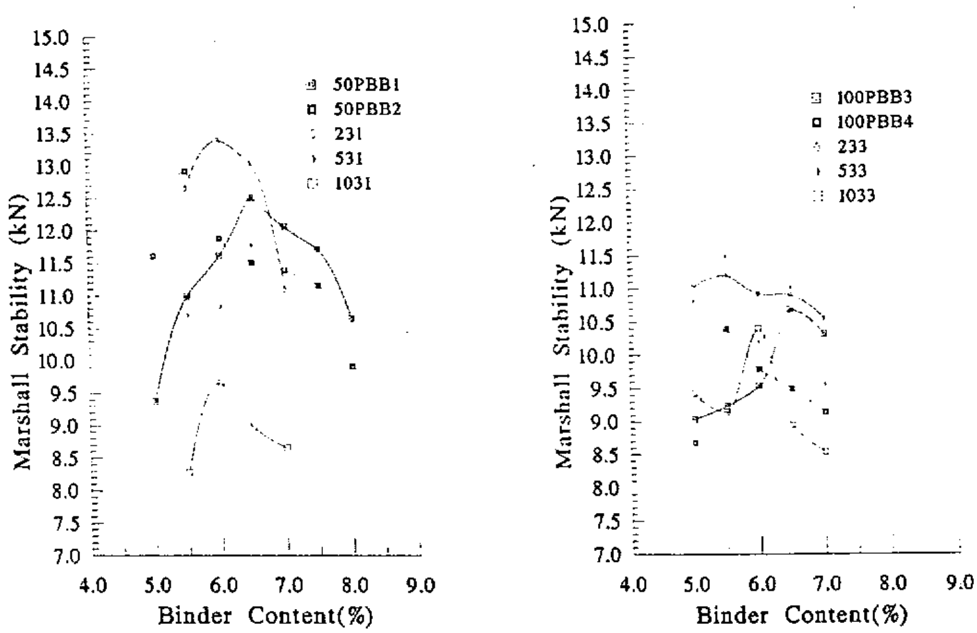


Figure 6.6. (d). Marshall Stability vs. Binder Content for rubberised asphaltic concrete made with 3's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) base bitumen.

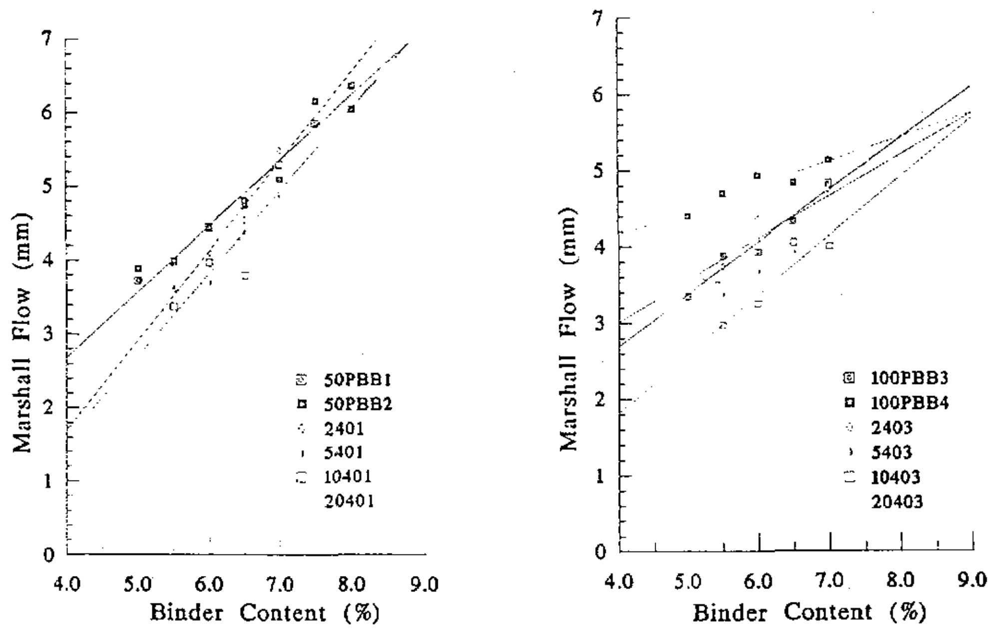


Figure 6.7. (a). Marshall Flow vs. Binder Content for rubberised asphaltic concrete made with 40's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) base bitumen.

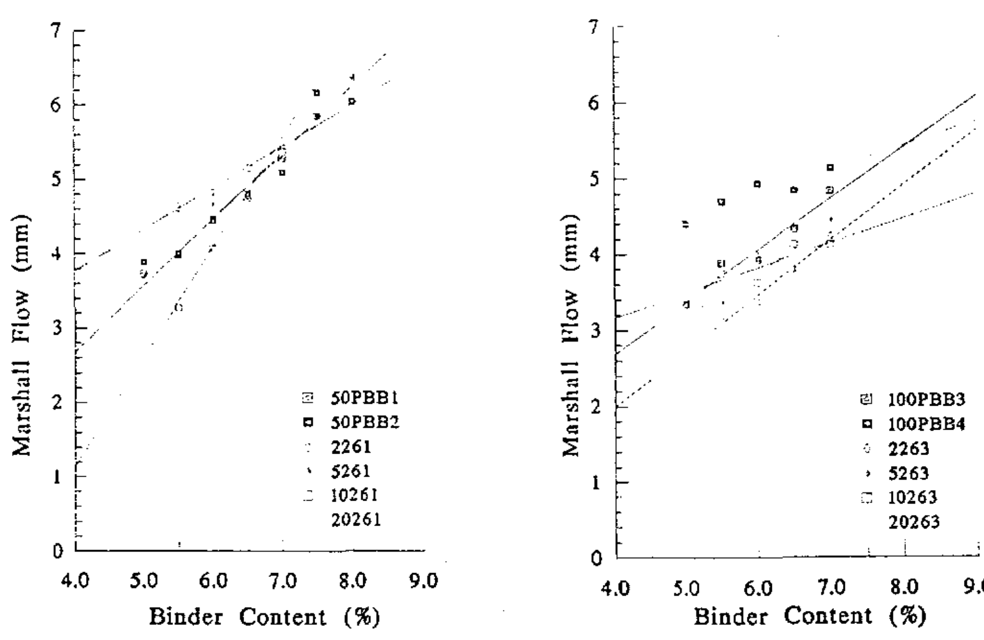


Figure 6.7. (b). Marshall Flow vs. Binder Content for rubberised asphaltic concrete made with 12/16's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) bitumen.

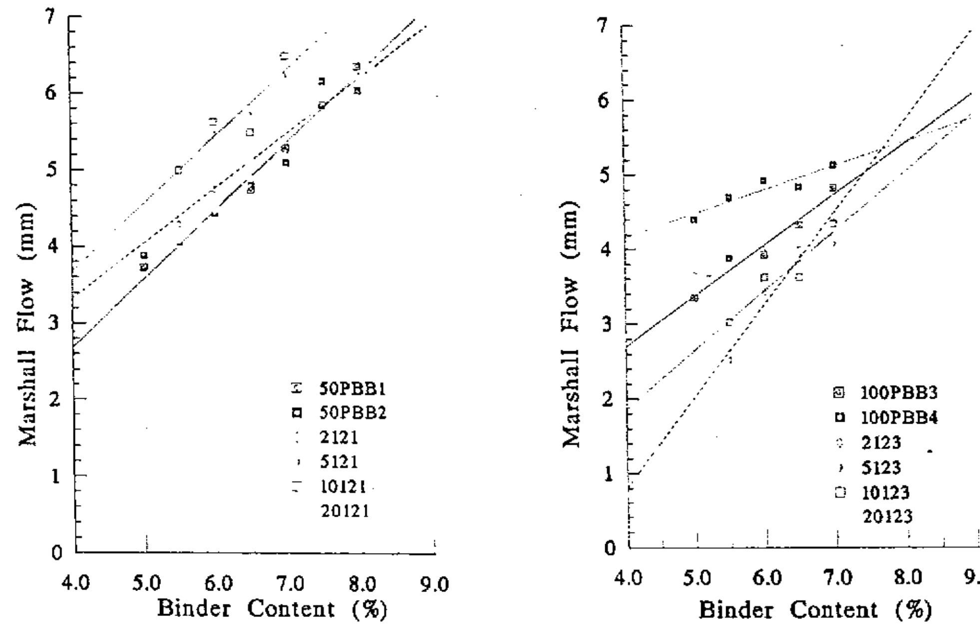


Figure 6.7. (c). Marshall Flow vs. Binder Content for rubberised asphaltic concrete made with 12's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) base bitumen.

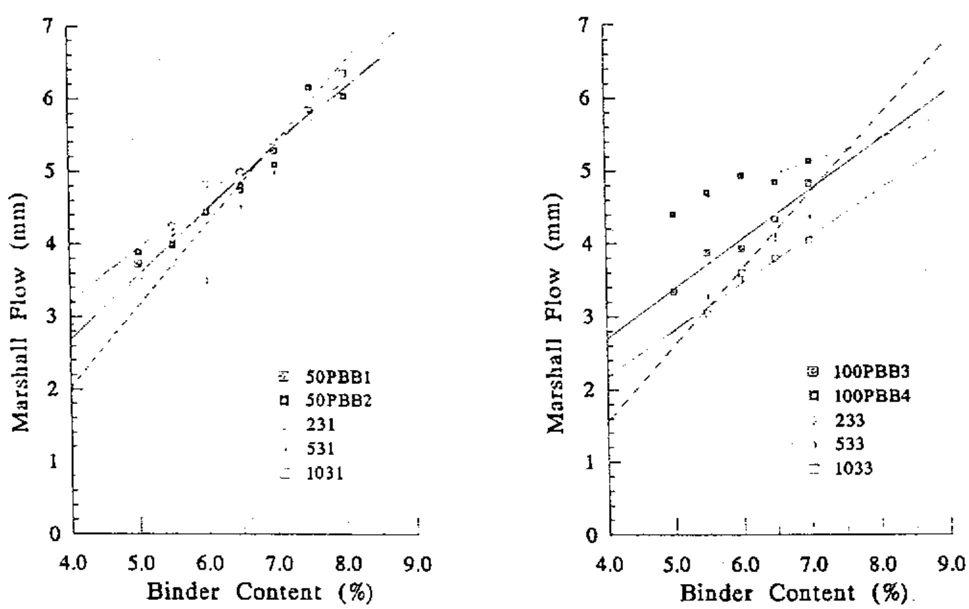


Figure 6.7. (d). Marshall Flow vs. Binder Content for rubberised asphaltic concrete made with 3's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) base bitumen.

6.8. Air Permeability of the Compacted Mixture

Most bituminous paving engineers and technologist recommended that high permeability, high air voids and thin bituminous coatings on the aggregate particles should be avoided in the design and construction of bituminous pavements (Goode *et al.*, 1965). The effect of permeability on the durability of compacted bituminous mixture has been discussed in Chapter Four. The measurement of air permeability provides information about the distribution of air voids and their degree of inter-connection. In order to produce a durable mixture without loss of stability, the mixture has to be impermeable or of low permeability.

The effect of rubber on the coefficient of air permeability vs. binder content for the rubberised asphaltic concrete is given in Figures 6.8. (a), (b), (c) and (d). The results shown that the relationships between air permeability and binder content of asphaltic concrete is significantly effected by the addition of rubber. Permeability of the mixture increases with increasing rubber content and rubber size. The permeability of the asphaltic concrete decreases as binder content increases until a minimum value is reached after which the permeability shows a certain amount of increase with increasing binder content. This increase is not proportional to the increase of binder content. From the result, it is apparent that rubber affect the permeability of the mixture significantly. It can be said that, the porosity and permeability of the asphaltic concrete made with shredded rubber modified bitumen show similarities due to high porosity.

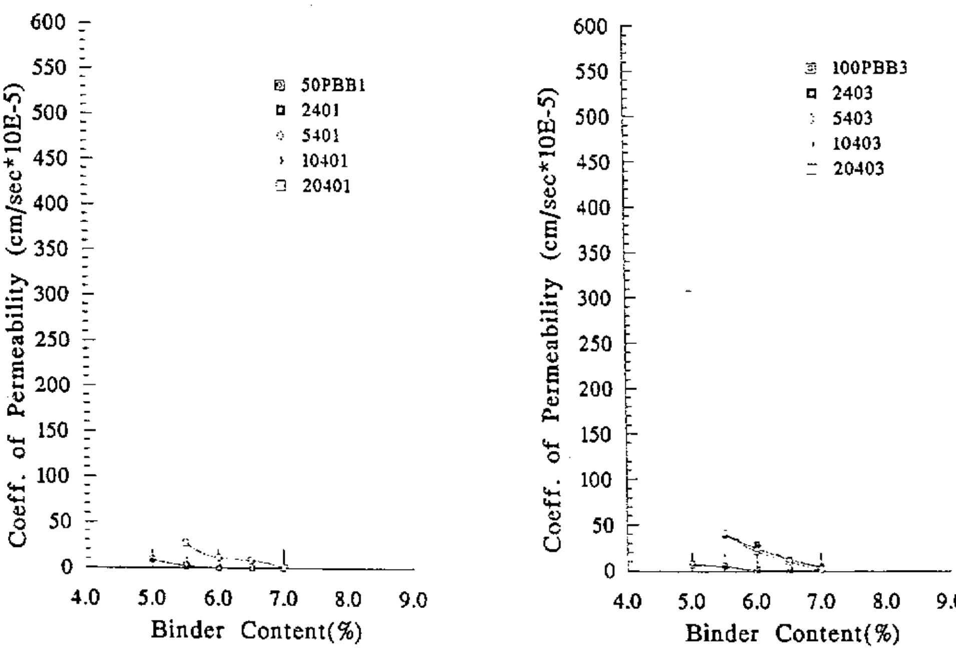


Figure 6.8. (a). Air Permeability vs. Binder Content for rubberised asphaltic concrete made with 40's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) base bitumen.

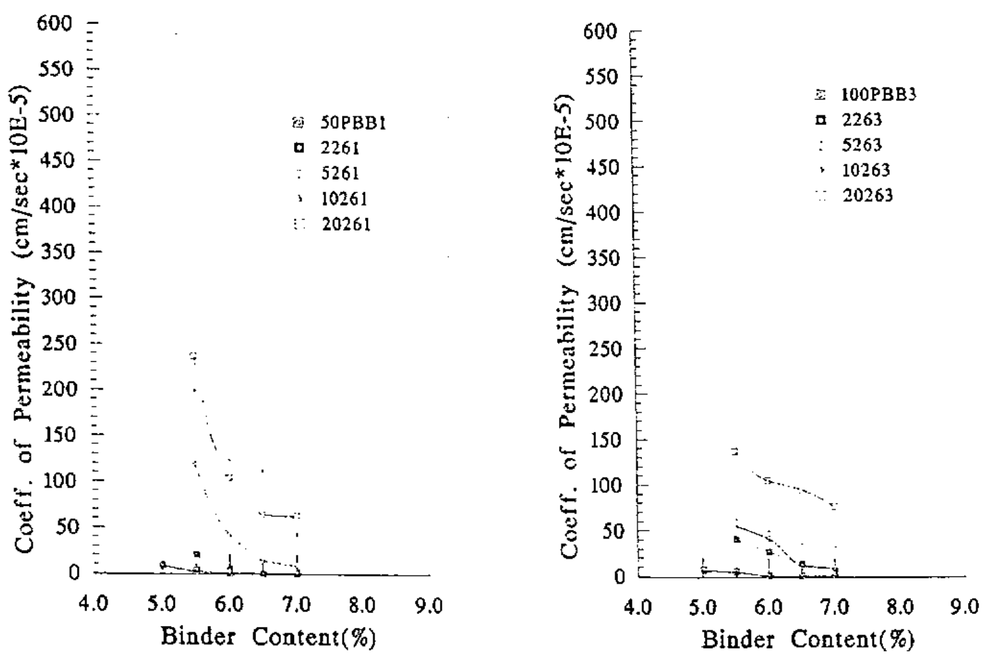


Figure 6.8. (b). Air Permeability vs. Binder Content for rubberised asphaltic concrete made with 12/16's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) bitumen.

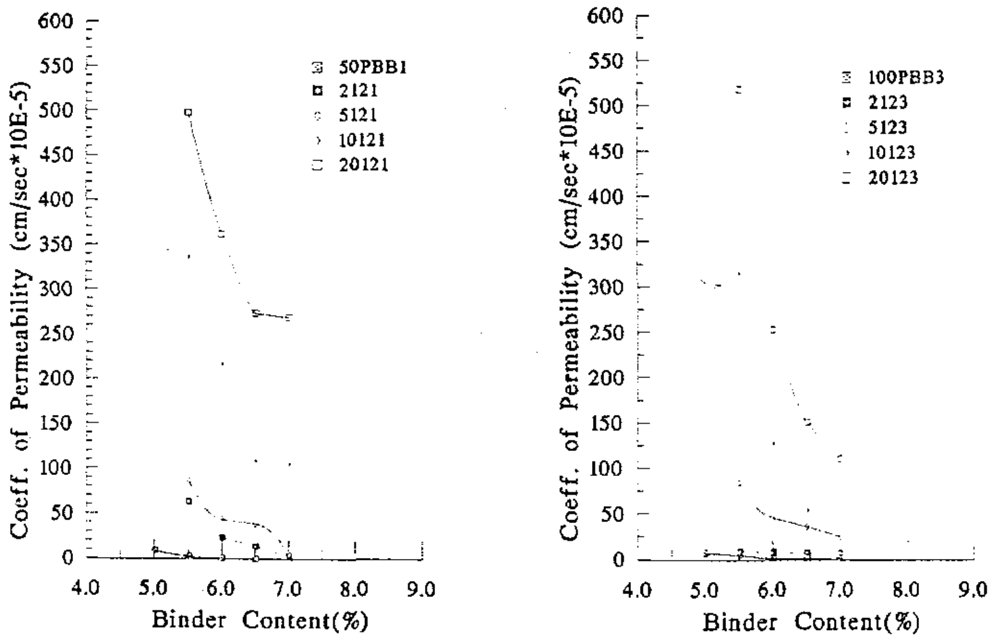


Figure 6.8. (c). Air Permeability vs. Binder Content for rubberised asphaltic concrete made with 12's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) base bitumen.

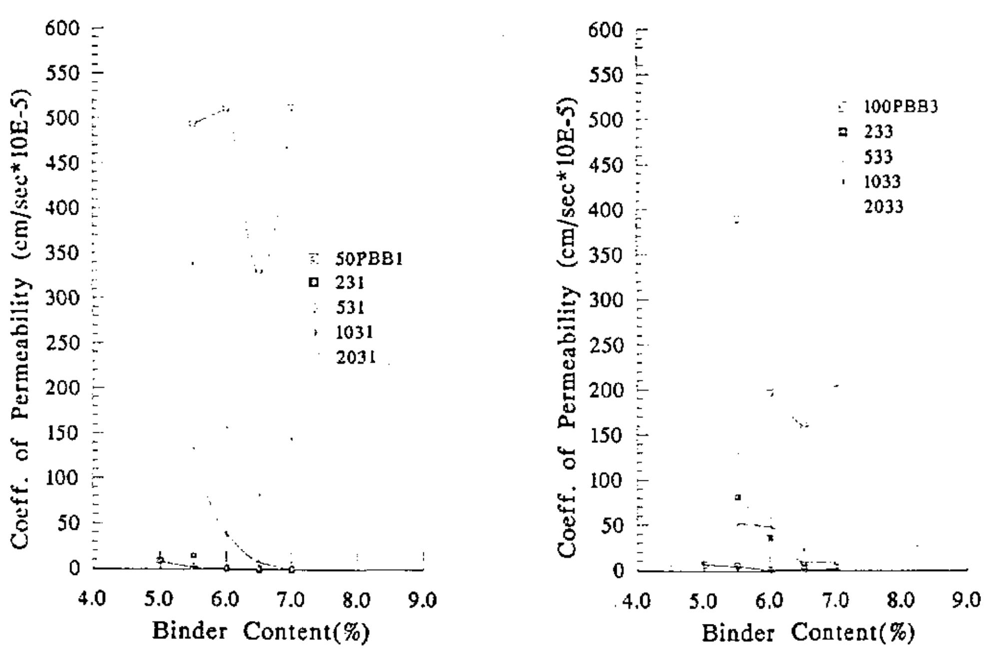


Figure 6.8. (d). Air Permeability vs. Binder Content for rubberised asphaltic concrete made with 3's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) base bitumen.

6.9. Creep Stiffness of the Compacted Mixture

The permanent deformation or rutting of bituminous mixture is a problem. Any deformation in the mix is the result of displacements between mineral particles separated by a thin film of bitumen. The rate of strain is depend on the magnitude of the stress, the thickness of the bitumen film and the bitumen properties (Van de Loo, 1978) and temperature (Napiah, 1993).

The creep properties of the asphaltic concrete are very useful for the estimation of permanent deformation. According to Brown (1980), Marshall stability test results are unsuitable for ranking mixes in order to predict their deformation resistance when different types of mixes are involved and the specimens prepared by the normal Marshall procedure. It has been suggested that mixes with generally good or bad deformation resistance can be distinguished by using unconfined creep test. To account its importance, the Leeds Design Method (Cabrera, 1992) has includes creep stiffness measured after 1 hour loading as one of its design parameter to determine the optimum binder content. The procedure for the determination of one hour creep stiffness of the mixture is given in Chapter 5.

Figure 6.9 shows change in creep stiffness with changing loading time for the asphaltic concrete made with base bitumen of 50 pen. The Creep stiffness of the same mixture against stiffness of binder is also shown in Figure 6.10. in log-log scale. Maximum 1 hour creep stiffness of the mixture given in Figure 6.11. As it is shown in Figure 6.11. the 1 hour creep stiffness of the mixture decreases with increasing binder content. It is clear that there is no maximum point except type 3 on the graph. The slope of the lines, which is obtained from stiffness of the mix against stiffness of the bitumen, is used to determine the optimum binder content in this investigation. This manner of characterizing a mix is based on the fact that more resistant mixtures have stiffness that are larger and decrease less rapidly with increasing loading time. Figure 6.12. shows the slope of the lines against binder content. The minimum values for the slope is chosen for the optimum binder content.

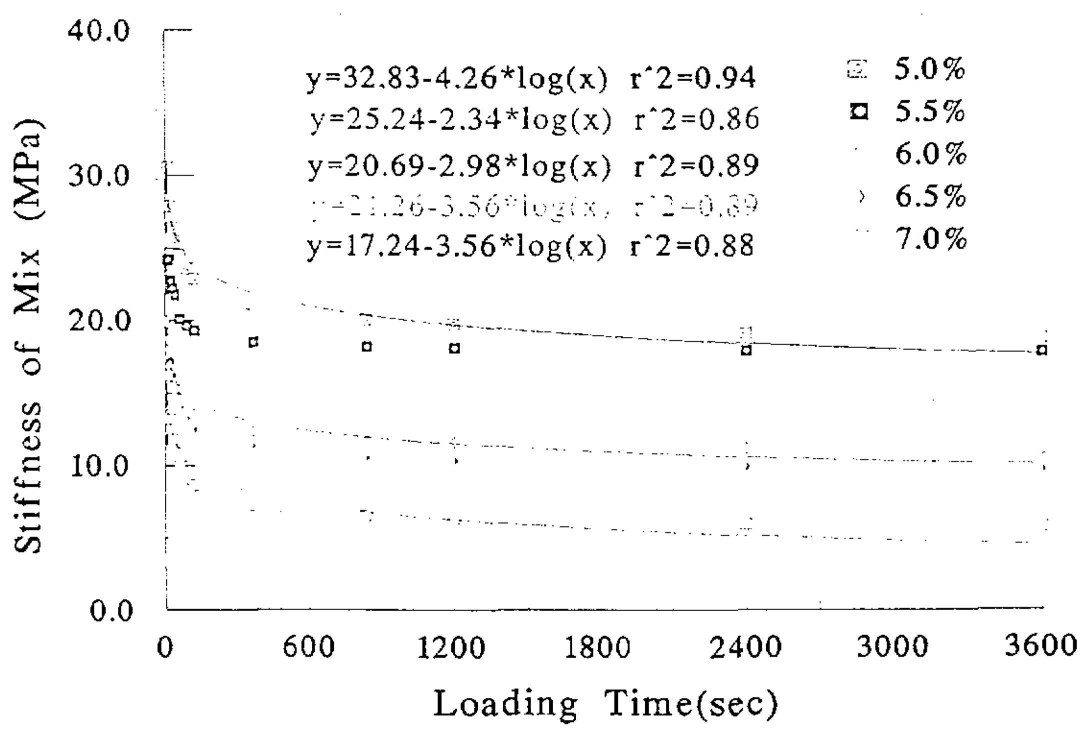


Figure 6.9. Loading Time vs. Stiffness of Mix for asphaltic concrete made with 50 pen base bitumen (for the aggregate gradation type DCT result).

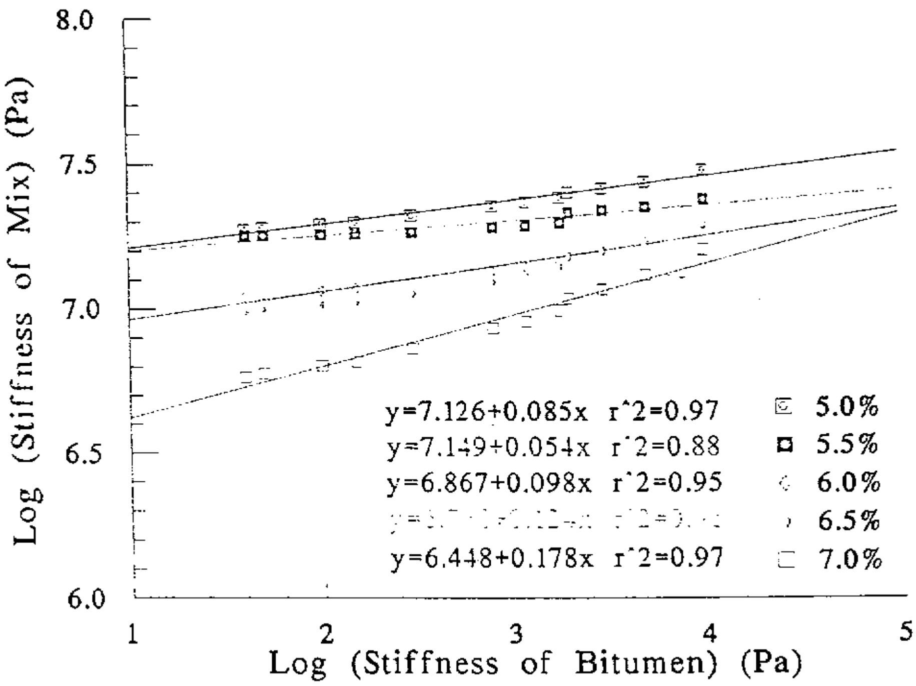


Figure 6.10. Stiffness of Mix vs. Stiffness of Bitumen for asphaltic concrete made with 50 pen base bitumen (for the aggregate gradation type DCT).

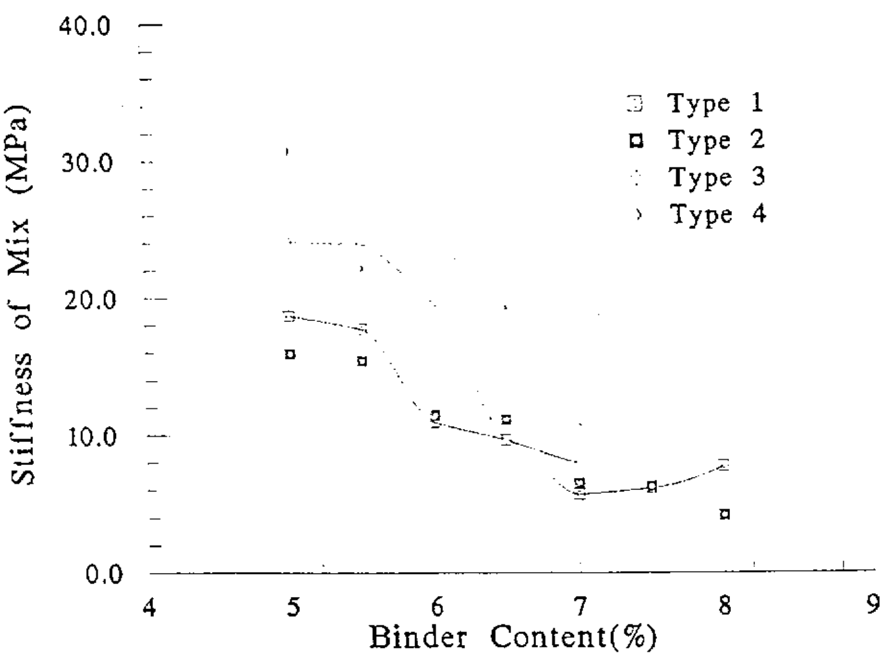


Figure 6.11. Stiffness of Mix vs. Binder Content.

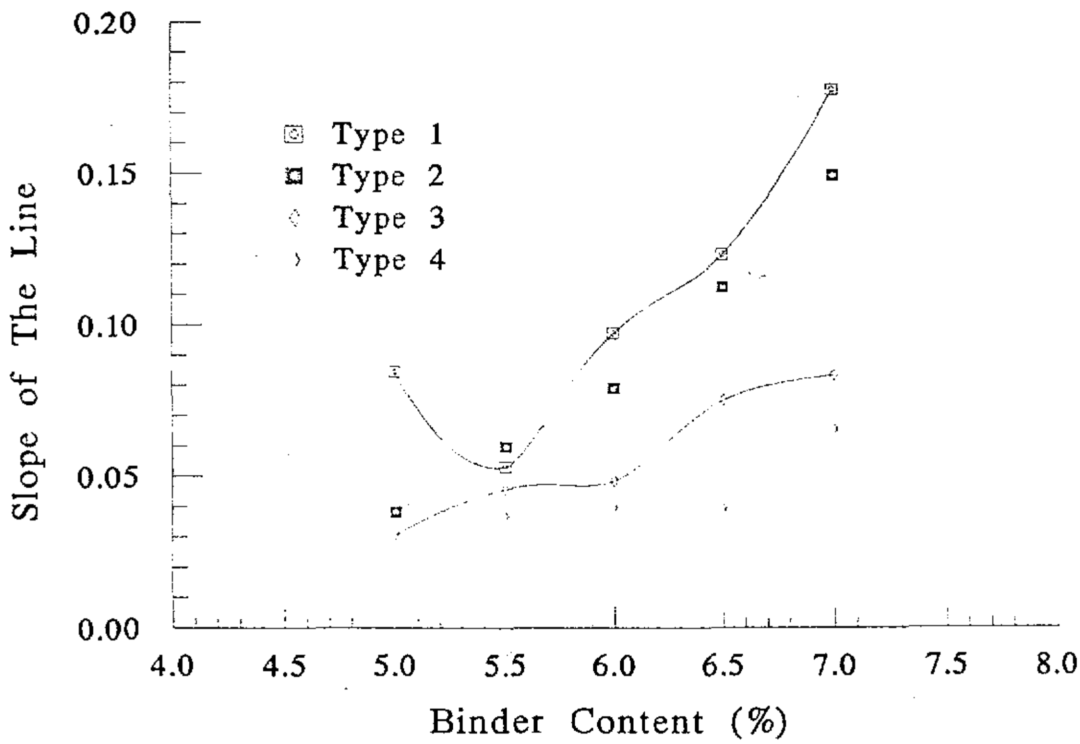


Figure 6.12. Slope of Mix Stiffness against Bitumen Stiffness Line vs. Binder Content.

To determine the effect of rubber on the creep stiffness of the mixtures, the results of the creep tests are presented in term of the relationship between the mixture stiffness at one hour loading time and bitumen content. Figures 6.13. (a), (b), (c) and (d) show the relationship between binder content and stiffness of the asphaltic concrete made with shredded rubber. It was observed that the creep stiffness of the mixture increases over 100% when small amount of fine rubber (40's rubber 2% by weight, Figure 6.13.a.(left)) was incorporated into the bitumen. High concentration rubber addition, on the other hand, reduces the creep stiffness of a mixture due to high porosity.

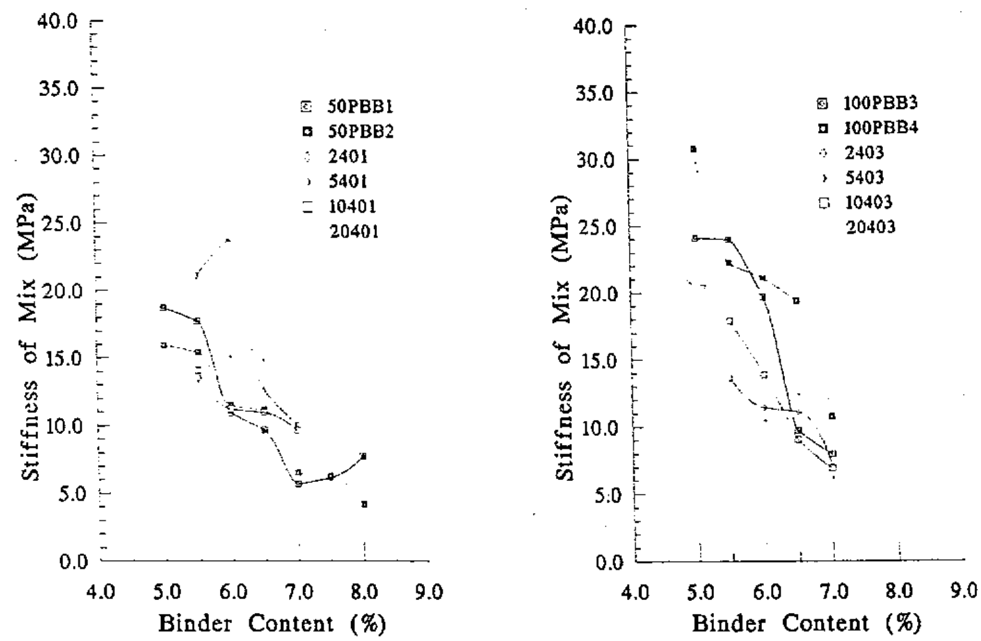


Figure 6.13. (a). Creep Stiffness vs. Binder Content for rubberised asphaltic concrete made with 40's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) base bitumen.

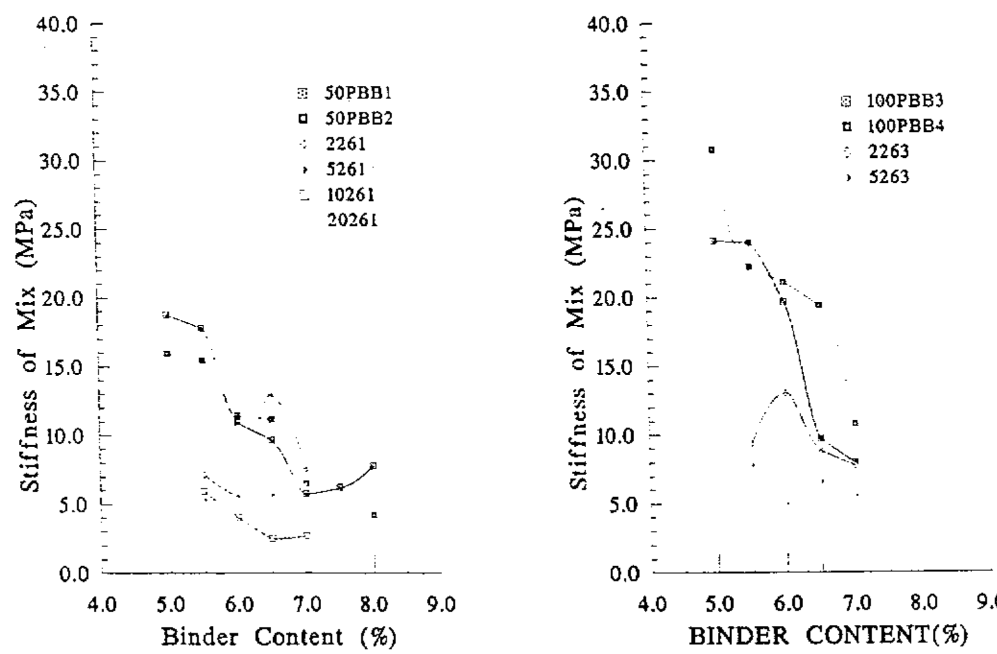


Figure 6.13. (b). Creep Stiffness vs. Binder Content for rubberised asphaltic concrete made with 12/16's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) bitumen.

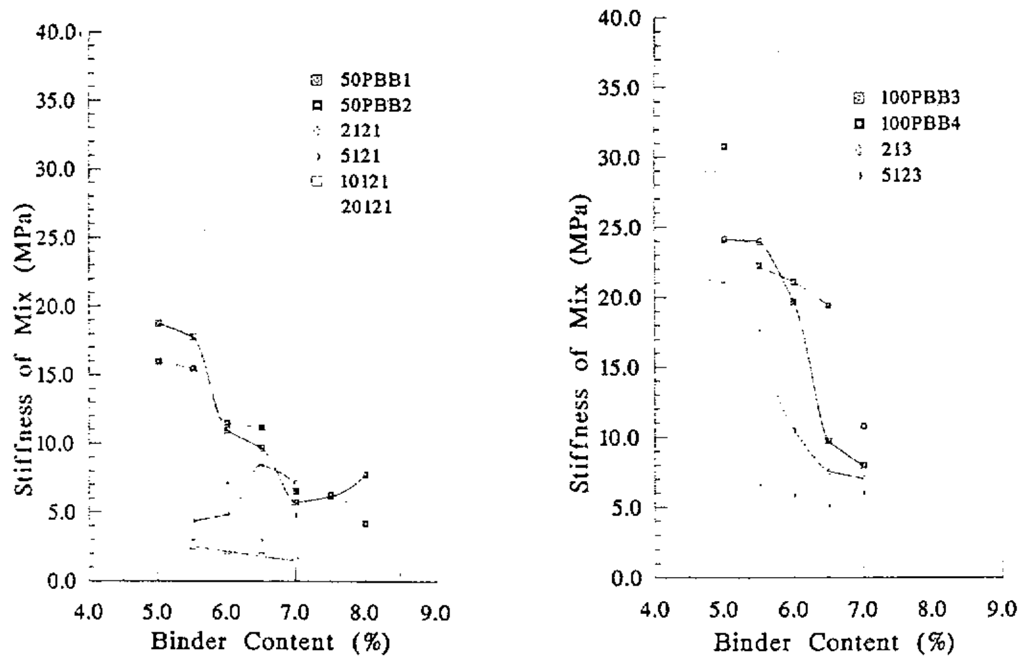


Figure 6.13. (c). Creep Stiffness vs. Binder Content for rubberised asphaltic concrete made with 12's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) base bitumen.

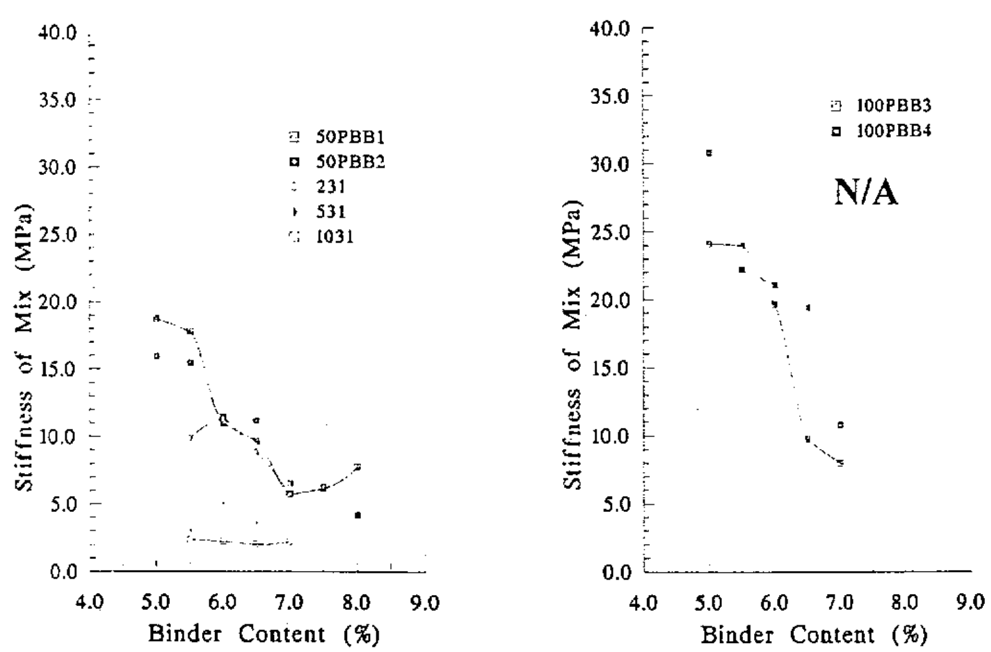


Figure 6.13. (d). Creep Stiffness vs. Binder Content for rubberised asphaltic concrete made with 3's size shredded tire rubber in 50 pen.(left) and 100 pen.(right) base bitumen.

6.10. Optimum Binder Content

Optimum binder content determination is one of the most important aspect in the design procedure. The mixture properties are very sensitive to the change of binder content. Too rich a mix would lack resistance to deformation and leaner mixes would lack resistance to tensile stresses.

Several methods are available to determine the optimum binder content of the mixture. In the UK, the method recommended in BS 598 (1990) is the most widely used. BS 598 (1990) recommends to obtain the optimum binder content from the average of the bitumen content at maximum stability, maximum density and the intended air voids. The mix design method for the asphaltic concrete by Asphalt Institute (1988) also recommends the same. However, in this investigation, The Leeds Design Method (Cabrera, 1992) is used to determine the optimum binder content of the asphaltic concrete made with tire shredded rubberised bitumen. In addition to the parameters outlined by BS and Asphalt Institute specification, the Leeds Design Method (Cabrera, 1992) includes air permeability, static creep stiffness of the mixture and compacted aggregate density as the parameters to determine the optimum binder content of the mix. The results for the mixes studied are shown in Table 6.1..

The effect of rubber on the optimum binder content (University of Leeds) (OBC-UL) is given Figure 6.14. for all mixes made with 50 pen. (right) and 100 pen. (left) grade base bitumen plus shredded tire rubber in different concentration. Analysis of results show that the addition of rubber into the bitumen increases slightly the optimum binder content when the rubber is added in any percentage or any size. The binder content was kept in the range of 5% to 7% during this research. There is a need to extend the investigation gap for higher concentration of shredded rubber modified bitumen. However, it can be said that the addition of high concentration rubber reduces the required properties of asphaltic concrete such as increase in porosity and decrease in Marshall Stability etc.

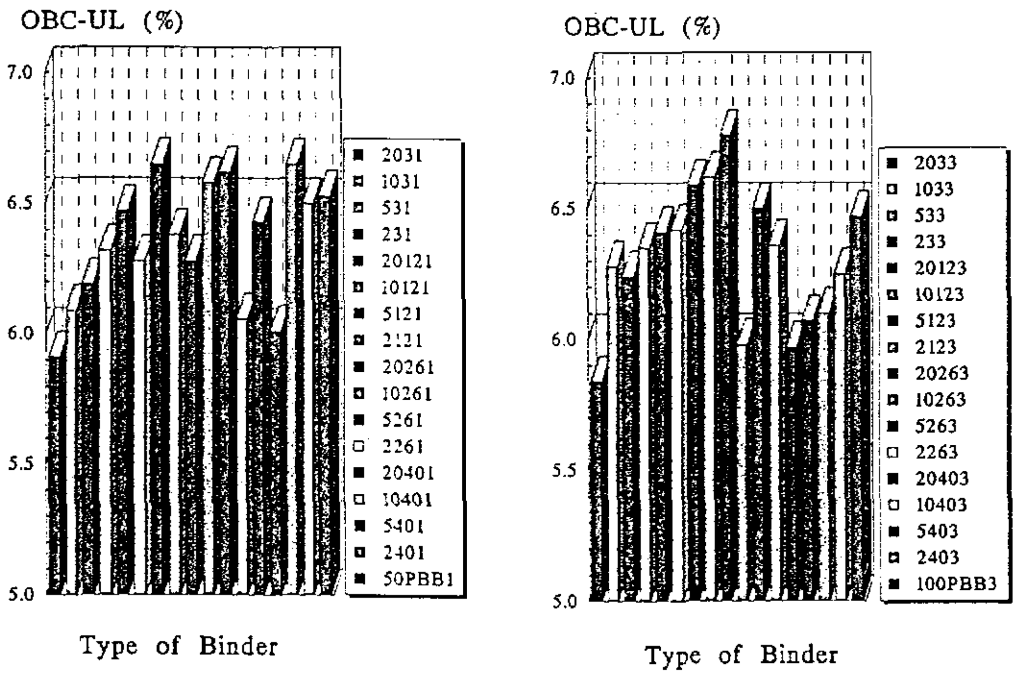


Figure 6.14. Optimum Binder Content for rubberised asphaltic concrete investigated.

Another important aspect related with the properties of rubberized asphaltic concrete is the WI. A workable mix should have a WI of over 6. (Cabrera, 1991). The workability index of asphaltic concrete made with the smallest size (40's) rubber was high enough for all rubber concentrations (2,5,10 and 20%) for both penetration grade bitumen 50 and

100, the WI values are under 6 for the bigger sizes (12's, 12/16's and 3's) for both penetration grade bitumen when the rubber content exceeds 5%.

6.11. Summary

In this investigation, a laboratory work conducted on the engineering properties of asphaltic concrete made with waste shredded tire rubber modified (50 and 100 pen. grade base) bitumen. The experimental design procedure showed that the difficulties arose during the compaction of the material. At high rubber content the elasticity on the binder increases leading to a rubber like behavior during compaction, i.e. release of the pressure caused a large amount rebound of the compacted mixture. In the previous chapter, it was said that the rubber particles are not digested completely. At the time of compaction, the temperature of the sample is still around 130 °C and the viscosity of bitumen is still low and it do not withstand to keep the form of sample, this leads an increase on the porosity of the compacted specimen. To reduce this, a modification of the compaction method. is necessarily

In spite of increasing the porosity, the investigated properties of the rubberised asphaltic concrete are still in the acceptable range in accordance with the related specifications (BS 594, 1992; Asphalt Institute, 1988).

Properties	50PBB1	2401	5401	10401	20401	2261	5261	10261	20261
Optimum Binder Content (%)	5.91	6.09	6.19	6.32	6.47	6.28	6.65	6.38	6.28
Marshall Stability (kN)	9.8	9.6	9.4	9.6	9.4	9	8.7	6.8	7.9
Marshall Flow (mm)	4.1	4.06	4.0	3.9	3.4	4.65	4.8	4.7	3.3
Marshall Quotient (kN/mm)	2.39	2.36	2.35	2.46	2.76	1.94	1.8	1.45	2.39
Density (gr/cc)	2.42	2.415	2.41	2.39	2.33	2.38	2.38	2.23	2.20
Compacted Agg. Density(gr/cc)	2.28	2.27	2.265	2.24	2.18	2.25	2.22	2.12	2.06
Voids in Mixed Aggregates (%)	17.2	17.5	17.8	19.0	21.5	18.4	19.4	24.6	25.1
Porosity(%)(calc. fr. geometry)	3.6	3.6	3.7	4.4	6.6	4.0	5.7	11.5	13.9
Porosity(%)(calc. in air & water)	3.4	3.2	3.3	4.3	6.3	3.5	4.0	9.5	12.0
Porosity (%)(Vacuum sat.)	1.8	1.4	1.7	2.5	4.4	1.4	2.0	7.3	10.9
Workability Index	7.25	7.6	7.3	7.2	6.3	6.8	6.75	4.8	4.3
Creep Stiffness (MPa)	10.0	22.0	15.0	11.0	12.0	9.0	6.0	3.0	5.0
Coeff. of Permeability (cm/s x10^{-5})	0.5	0.7	0.1	5.0	9.0	1.5	10.0	115.0	72.0

Properties	50PBB1	2121	5121	10121	20121	231	531	1031	2031
Optimum Binder Content (%)	5.91	6.58	6.62	6.06	6.43	6.01	6.65	6.50	6.53
Marshall Stability (kN)	9.8	8.4	8.6	6.2	6.0	10.5	8.5	6.3	NA
Marshall Flow (mm)	4.1	5.1	5.2	5.6	5.0	4.1	4.8	5.0	NA
Marshall Quotient (kN/mm)	2.39	1.65	1.65	1.11	1.2	2.56	1.77	1.26	NA
Density (gr/cc)	2.42	2.37	2.34	2.20	2.05	2.41	2.37	2.24	2.10
Compacted Agg. Density(gr/cc)	2.28	2.23	2.2	2.12	2.0	2.28	2.22	2.13	2.02
Voids in Mixed Aggregates (%)	17.2	18.3	20.6	24.1	27.5	17.5	19.7	22.9	26.6
Porosity(%)(calc. fr. geometry)	3.6	4.6	5.5	13.3	18.5	4.3	4.4	12	19.1
Porosity(%)(calc. in air & water)	3.4	3.8	4.5	10.6	15.8	3.4	3.9	10	17.5
Porosity (%)	1.8	2.2	3.1	10.2	14.2	1.9	2.1	8.6	13.2
Workability Index	7.25	7.3	6.8	4.7	3.8	7.4	6.7	4.7	3.7
Creep Stiffness (MPa)	10.0	8.0	3.0	2.0	2.0	11.5	4.0	2.0	NA
Coeff. of Permeability (cm/s x10^{-5})	0.5	8.0	23.0	200.0	270.0	1.0	3.0	66.0	316.0

Table 6.1. Engineering Properties of Asphaltic Concrete Made With 50 pen. Base Bitumen Plus Shredded Rubber.

Properties	100PBB3	2403	5403	10403	20403	2263	5263	10263	20263
Optimum Binder Content (%)	5.84	6.28	6.24	6.35	6.41	6.42	6.59	6.62	6.78
Marshall Stability (kN)	9.6	7.7	8.0	8.0	7.5	7.9	6.7	5.2	5.7
Marshall Flow (mm)	3.5	4.25	4.0	3.7	2.8	3.8	4.1	4.0	3.4
Marshall Quotient (kN/mm)	2.74	1.81	2.0	2.16	2.68	2.08	1.63	1.3	1.68
Density (gr/cc)	2.43	2.39	2.39	2.35	2.29	2.40	2.35	2.26	2.16
Compacted Agg. Density(gr/cc)	2.29	2.24	2.24	2.21	2.15	2.25	2.21	2.12	2.03
Voids in Mixed Aggregates (%)	17.5	18.8	19.0	20.2	22.7	18.3	20.0	23.5	26.5
Porosity(%)(calc. fr. geometry)	3.0	4.2	4.2	5.5	8.2	4.1	5.4	9.7	13.4
Porosity(%)(calc. in air & water)	3.0	3.8	3.9	5.5	8.1	3.4	4.6	8.5	12.6
Porosity (%)(vacuum sat.)	2.0	3.2	2.5	4.2	7.4	2.4	3.9	7.7	12.3
Workability Index	7.4	6.75	6.75	6.1	5.75	6.95	6.15	5.1	4.4
Creep Stiffness (MPa)	21.5	12.0	12.0	10.0	16.0	9.5	6.5	NA	NA
Coeff. of Permeability (cm/s x10^{-5})	2.5	22.0	20.0	7.5	12.5	17.0	16.0	38.0	85.0

Properties	100PBB3	2123	5123	10123	20123	233	533	1033	2033
Optimum Binder Content (%)	5.84	5.98	6.5	6.36	5.97	6.07	6.10	6.25	6.47
Marshall Stability (kN)	9.6	8.7	7.0	6.3	5.0	8.0	7.3	6.7	NA
Marshall Flow (mm)	3.5	3.2	3.9	3.7	2.6	3.7	3.7	3.5	NA
Marshall Quotient (kN/mm)	2.74	2.72	1.79	1.70	1.92	2.16	1.97	1.91	NA
Density (gr/cc)	2.43	2.42	2.35	2.25	2.16	2.41	2.36	2.30	2.20
Compacted Agg. Density(gr/cc)	2.29	2.28	2.21	2.13	2.05	2.27	2.25	2.17	2.10
Voids in Mixed Aggregates (%)	17.5	17.4	20.0	22.8	25.5	18.1	19.7	21.2	24.3
Porosity(%)(calc. fr. geometry)	3.0	3.5	5.1	10.3	14.7	4.5	5.5	8.3	13.6
Porosity(%)(calc. in air & water)	3.0	3.3	4.6	9.1	12.6	3.5	4.9	7.3	11.1
Porosity (%)(vacuum sat.)	2.0	1.9	3.4	8.8	14.3	2.6	4.2	7.0	13.2
Workability Index	7.4	7.0	6.4	5.0	4.1	6.75	6.1	5.4	4.4
Creep Stiffness (MPa)	21.5	10.8	5.2	NA	NA	NA	NA	NA	NA
Coeff. of Permeability (cm/s x10^{-5})	2.5	8.5	35.0	68.0	275.0	35.0	37.0	38.0	150.0

Table 6.2. Engineering Properties of Asphaltic Concrete Made With 100 pen. Base Bitumen Plus Shredded Rubber.

CHAPTER SEVEN

THE MEASUREMENT OF THE POROSITY OF ASPHALTIC CONCRETE MADE WITH WASTE THREAD TIRE RUBBER MODIFIED BINDERS

7.1. Introduction

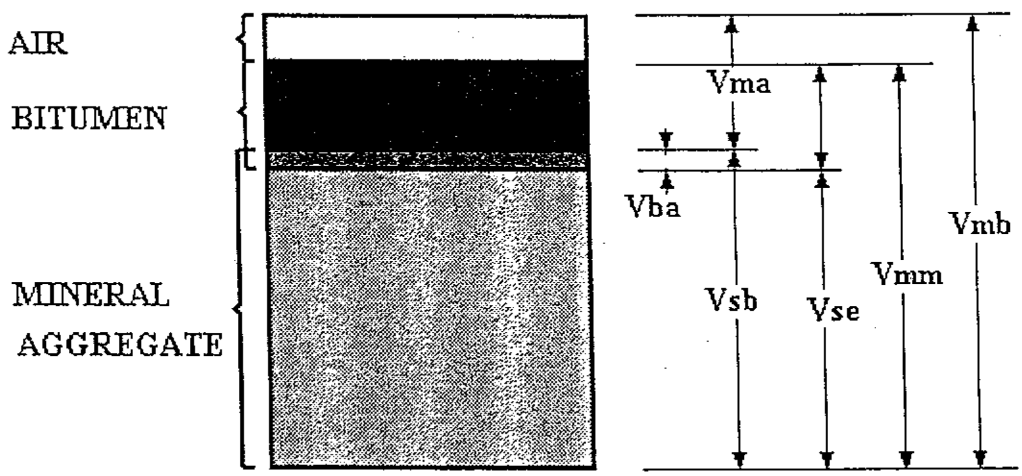
Most researchers agree that high porosity, high permeability should be avoided when designing and constructing bituminous pavements. Limiting the porosity of bituminous mixture is one of the main considerations when determining the optimum binder content (O'Flaherty, 1988).

Porosity is the sum of the pores in the bituminous mixture. It is expressed as a percentage of the total volume of the specimen and it represents the volume that is not occupied by either bitumen or aggregates. VMA consist of the inter-granular void spaces between the aggregate particles that hold bitumen in a compacted mixture. VMA is equal to the volume of binder plus the porosity. It is also expressed as a percentage of the total volume of the specimen and represents the volume of the compacted mix that is not occupied by aggregate.

Porosity is one of the most important factor effecting the long term performance and durability of bituminous mixture. Basically, the porosity must be high enough to allow for expansion of binder and entrained air under hot weather condition, to provide the mixture with a certain amount of elasticity and to provide safety space for further compaction of the surfacing under heavy traffic. When the porosity is too low, the pavement is likely to flush or bleed. The porosity should not be too high such that it will allow ingress of air and water to further aggravate disintegration.

7.2. Porosity Calculation

The fundamentals involved in the calculations of porosity of bituminous mixtures are shown in Figures 7.1. and 7.2. (Asphalt Institute, 1988).



Vma, Volume of voids in mineral aggregate,
Vmb, Bulk volume of compacted mix,
Vmm, Voidless volume of paving mix,
Va, Volume of air,
Vb, Volume of bitumen,
Vba, Volume of absorbed bitumen,
Vsb, Volume of mineral aggregate (by bulk specific gravity),
Vse, Volume of mineral aggregate (by effective specific gravity).

Figure 7.1. Representative of volumes in compacted bituminous mixture (Asphalt Institute, 1988).

The porosity expressed as a percentage in the compacted specimen is obtained by subtracting the bulk density from the theoretical or actual voidless density (specific gravity or relative density of the mix) and expressing the difference as a percentage of the specific gravity of the mix.

$$P = \frac{SG_{mix} - D}{SG_{mix}} \times 100 \qquad (7.1.)$$

where;

P, porosity (%),

Sg_{mix}, Specific Gravity of Mix (gr/cc),

D, bulk density of the specimen (gr/cc).

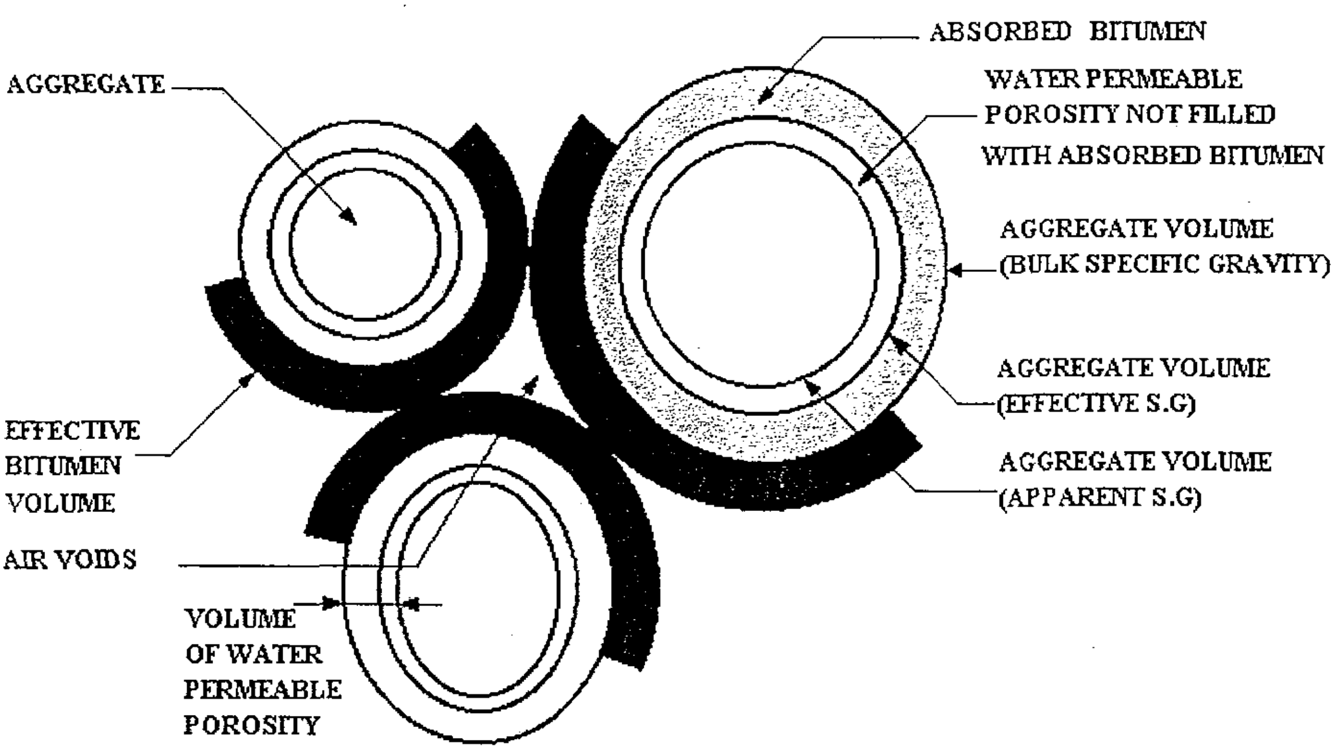


Figure 7.2. VMA, Porosity and Effective Binder Content in compacted bituminous paving mixture (Asphalt Institute, 1988).

Because the determination of porosity in bituminous mixtures is wholly based on calculated value which are dependent on the accuracy of the mix composition, it is difficult to use porosity as a crucial parameter for acceptance of constructed bituminous pavements. In fact porosity measurements are often the source of dispute and disagreement between contractor and engineer.

For this reason, it is important to develop methods for the direct measurement of porosity in the laboratory so that verification in the field can be made independently of the compaction of the mix.

This Chapter presents the results of the method of measurement based on saturating specimens in water under vacuum. The measured values are compared with the values calculated from the known mix composition and the differences are discussed in the context of performance and acceptability of bituminous mixtures.

7.3. Vacuum Porosity Measurement

The accurate measurement of the porosity of compacted bituminous mixtures is relevant in the context of their long term performance and durability. Direct measurements of porosity was made using the Leeds Vacuum Porosimeter developed by Cabrera (1985) for the measurement of cement concrete. Using this apparatus as a model a larger Vacuum Porosimeter was built specially for this work. A schematic diagram and the picture of the Leeds Vacuum Porosimeter is given in Figure 7.3. and Plate 7.1. respectively.

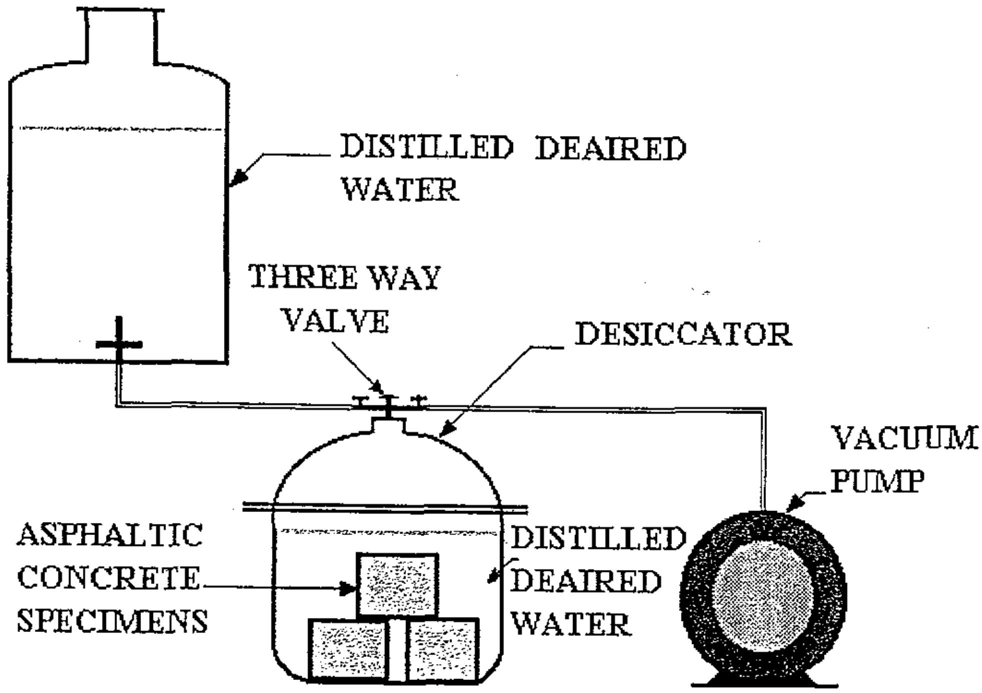


Figure 7.3. A schematic diagram of the Leeds Vacuum Porosimeter.

Basically, this apparatus consist of a container (desiccator) to place the specimens, a vacuum pump and a reservoir to hold distilled deaired water and a three way valve to connect the sample container to the pump, the reservoir and the environment.

7.3.1. Description of Saturation Process

The samples are placed the desiccator, which can hold up to six specimens. The desiccator is then connected to the vacuum pump and the specimens subjected to a pressure of -1 atm. for three hours. Then the valve connected to the water reservoir is open and water is allowed to cover the specimen (at least 3 cm. of water above the surface of the specimens). The specimens are kept immersed under vacuum for another 3 hours or until all air bubbles have been released. At the end of this period the container is connected to the atmosphere so that the pressure on the water surface is 1 atm. and inside the specimen

-1 atm.. The specimens are kept immersed overnight, then they are taken out of the desiccator, surface dried and weighed in air and in water.

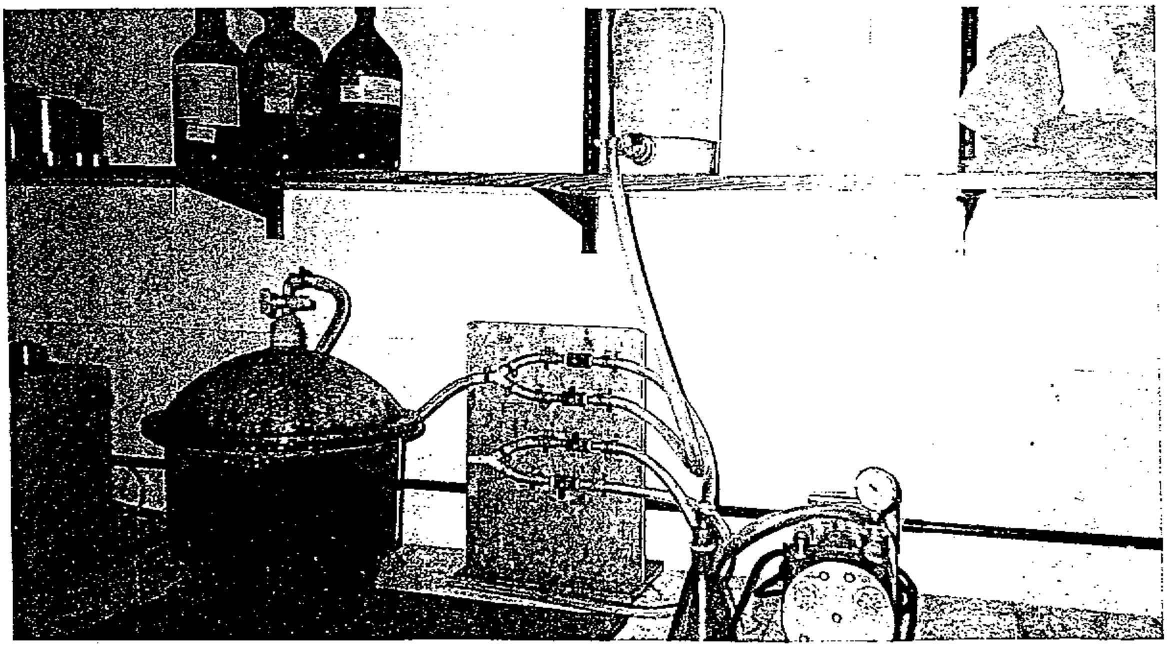


Plate 7.1. Leeds Vacuum Porosimeter (LVP).

The weight of water, which fills the void spaces inside the specimen expressed as a percentage of the total volume of the specimen, is the "water accessible porosity". It is calculated as shown in equation (7.2.);

$$Pv = \frac{Ws - Wa}{Ws - Ww} \times 100 \qquad (7.2.)$$

where;

Pv, porosity measured by vacuum saturation (%),

Ws, weight of sample in air (vacuum saturated) (gr),

Wa, weight of dry sample in air (gr),

Ww, weight of sample in water (vacuum saturated) (gr).

7.4. Results and Discussion

The calculated porosities, obtained either by using geometry of the specimen (Pg) or by using the volume of specimen obtained by water displacement (Paw) are plotted together with the measured vacuum porosity values for the range of binder contents of the asphaltic concrete made with 50 pen. and 100 pen. grade base bitumens. These are given in Figure 7.4. and 7.5.. An exponential curve can be fitted to the porosity vs. binder content function. The correlation coefficient was over 88% for the three functions within the investigated binder content range. The form of the equation found was as follows;

$$P = a \times 10^{b \times BC} \tag{7.3.}$$

where;

P, porosity of the compacted specimen (%),

BC, binder content (in weight) (%),

a and b, constant.

Their values are given in Figures 7.4., 7.5., 7.6. and 7.7..

7.4.1. The Effect of Rubber

Figure 7.6., Figure 7.7. (a) and (b) show the effect of 40's size shredded rubber concentration on the porosity of asphaltic concrete made with 50 and 100 pen. grade base bitumen respectively. It is clear that the porosity increases with increasing rubber content. This is attributed to increase on the elastic behavior of the binder. In Chapter 5, it was said that the viscosity and the elastic components of the binder increase with increasing rubber content. During the compaction process rubberised binder behaves more like an elastic

material, i.e. under pressure exhibits large deformation, once the pressure is released the instant rebound also large. This in addition to the decrease in WI leads to higher porosity.

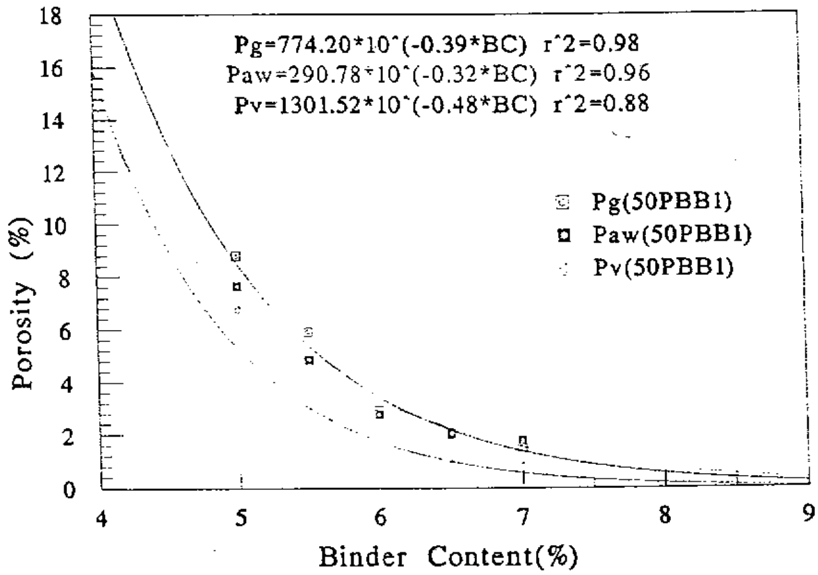


Figure 7.4. Porosity vs. Binder Content of asphaltic concrete made with 50 pen grade base bitumen.

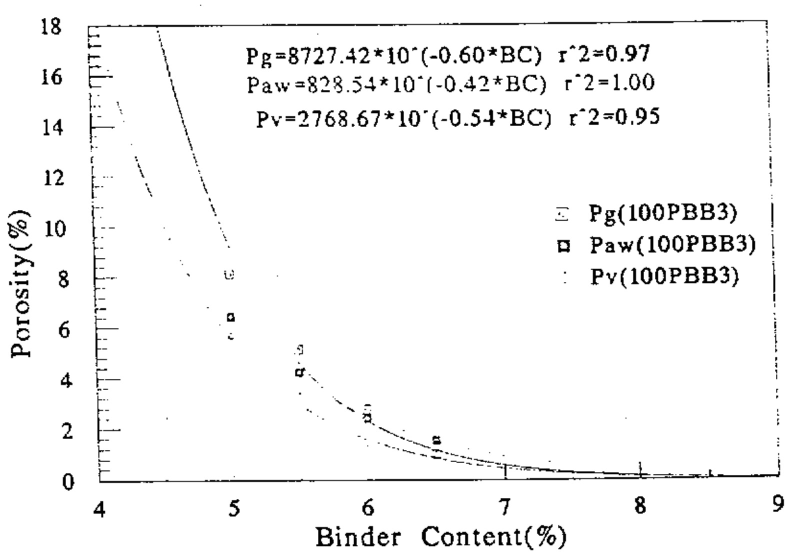


Figure 7.5. Porosity vs. Binder Content of asphaltic concrete made with 100 pen grade base bitumen.

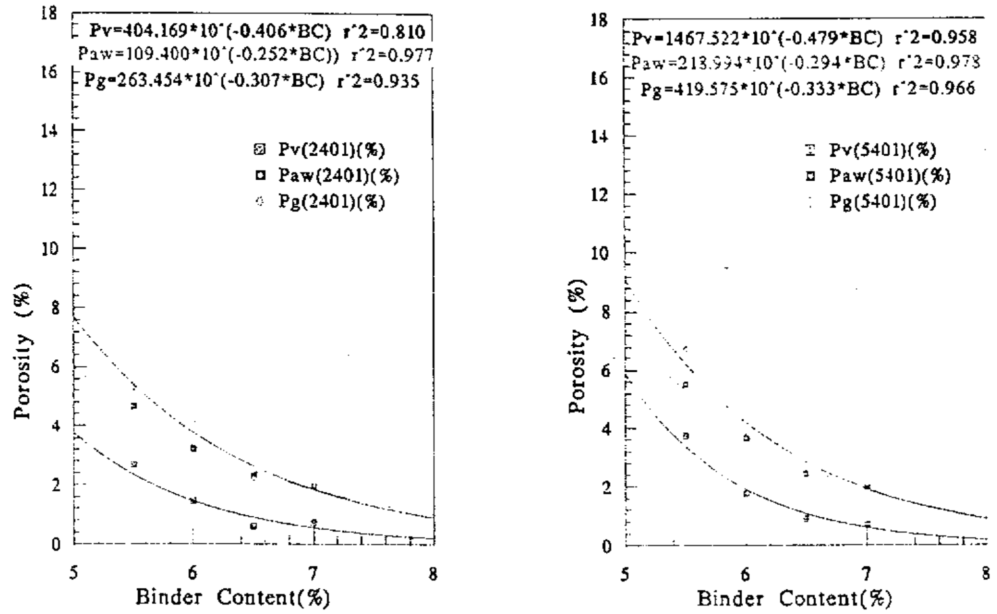


Figure 7.6. (a). Porosity vs. Binder Content of rubberised asphaltic concrete made with 50 pen base bitumen plus 2% (left) and 5% (right) 40's size rubber.

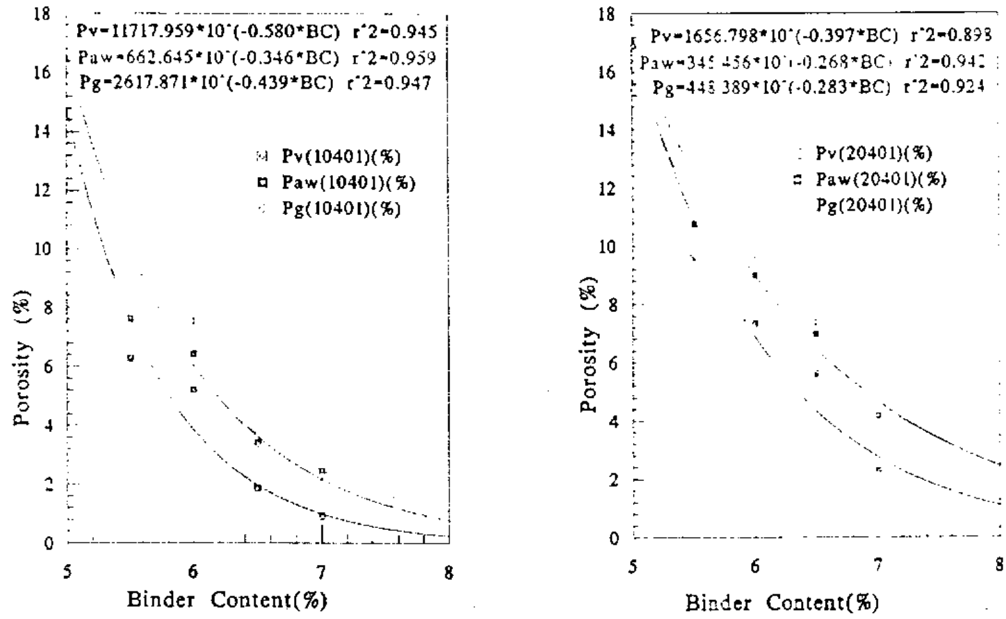


Figure 7.6. (b). Porosity vs. Binder Content of rubberised asphaltic concrete made with 50 pen. base bitumen plus 10% (left) and 20% (right) 40's size rubber.

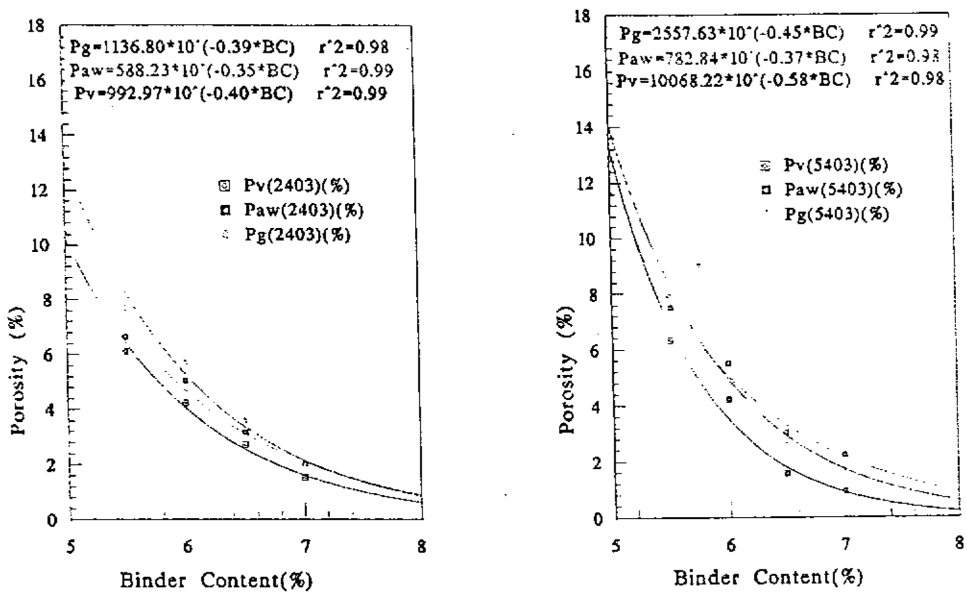


Figure 7.7. (a). Porosity vs. Binder Content of rubberised asphaltic concrete made with 100 pen. base bitumen plus 2% (left) and 5% (right) 40's size rubber.

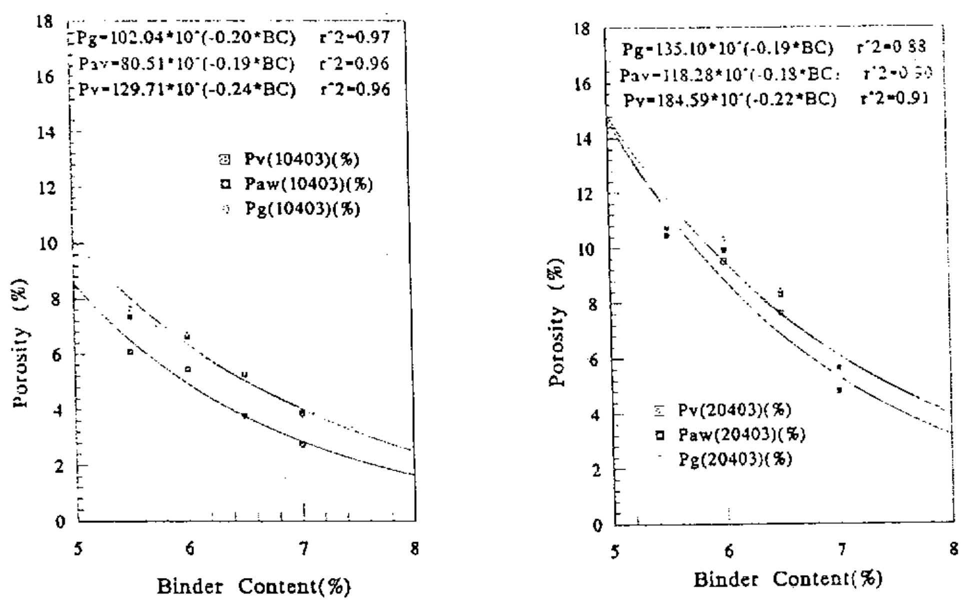


Figure 7.7. (b). Porosity vs. Binder Content of rubberised asphaltic concrete made with 100 pen. base bitumen plus 10% (left) and 20% (right) 40's size rubber.

7.4.2. The Effect of Porosity Measurement Method on The Porosity Values of The Mixture Studied

The relationship between the three porosity values will give a better idea of the porosity and pore structure characteristics of the bituminous mixtures. The three porosity values obtained are;

- Vacuum Porosity (Pv) measured as explained using by Leeds Vacuum Porosimeter,
- Geometric Porosity (Pg) calculated from the geometry of the specimen and its specific gravity,
- Air Water Porosity (Paw) using the calculated specific gravity and calculated density weighing the sample in air and in water.

Figure 7.8. shows the comparison between porosities obtained by vacuum saturation and calculated from the geometry of sample (Pg) (left) and by weighing the sample in air in water (Paw) (right) for the samples made 50 pen. base bitumen and 50 pen. base bitumen plus different size of shredded tire rubber. A linear relation with the correlation coefficient of 96% express the relationship between Pg and Pv (Figure 7.8.(left)). The equation of this line gives a slope which is quite close to 45° (constant b = 1.022). The constant of the equation (2.29) then can be taken as the percentage of non-water permeable voids or in other words the unconnected porous in the compacted mixture.

The calculated porosity (Paw) versus the measured vacuum porosity (Pv) (Figure 7.8. (right)) gives a relationship where the slope of the equation (b = 0.74) less than 1 with the result of that at low values of porosity the measured porosity is lower than the calculated one, while at high values of porosity the measured values are higher than the calculated ones and therefore using the constant of this relationship as an indication of the volume of porous which are not interconnected is not justified. However, there is a good statistical relation between Paw and Pg and as expected the values of Paw are smaller than the values of Pg. This is probably because on immersion a small volume of water penetrates the surface porous and because the volume measured is generally smaller than the calculated one.

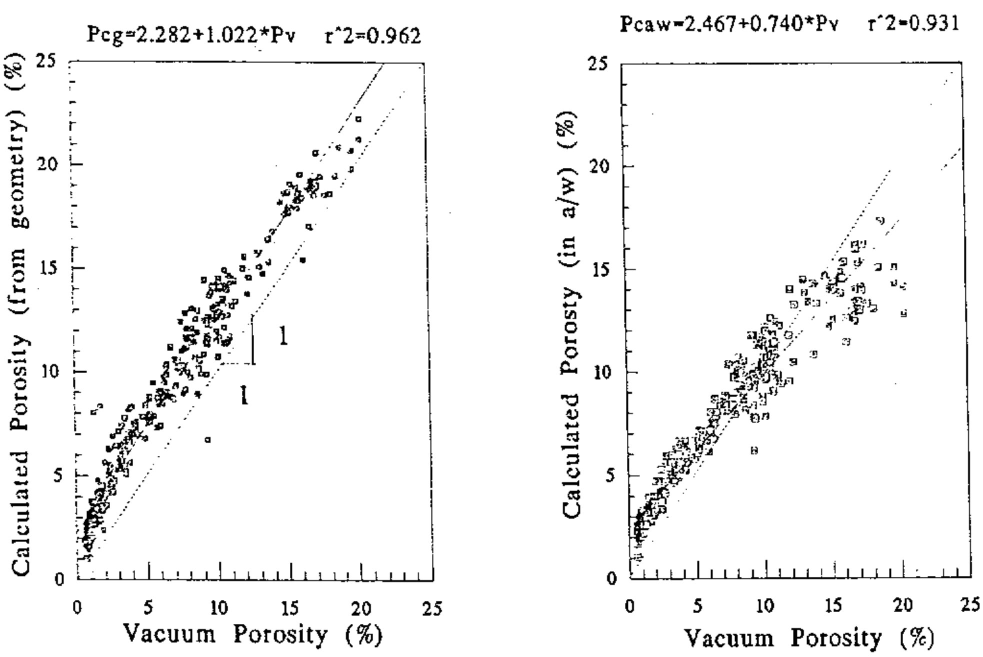


Figure 7.8. Vacuum Porosity (measured) vs. Calculated Porosity of asphaltic concrete made with 50 pen base bitumen plus different size of shredded tire rubber in different concentration Pg (left) and Paw (right).

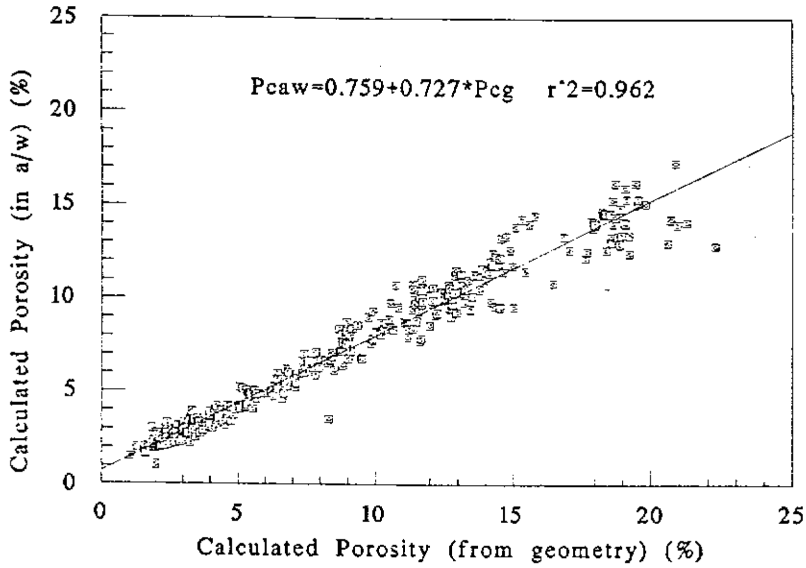


Figure 7.9. Calculated porosity values of rubberised asphaltic concrete (50PBB plus different size of shredded tire rubber in different concentration).

The porosity values for asphaltic concrete made with 100 pen. base bitumen and 100 pen base bitumen plus different shredded rubber sizes are given in Figure 7.10.. The results showed also that the constant b is very close to 1 (b = 0.963) in the equation relating Pg and Pv (Figure 7.10. (left)). The unreachable pore volume was found to be lower than for the specimen made with 50 pen. base bitumen (a = 1.599). This value is very close to that found for the relationship between Paw and Pv (a = 1.606) (Figure 7.10. (right)). But the constant b is quite low (b = 0.827) due to the same reason as explained.

Figure 7.11. shows the relationship between the calculated porosities Paw and Pg for the asphaltic concrete made with 100 pen base bitumen and its modification with the shredded rubber. In both cases, 50 and 100 pen, the Paw values were found to be higher than the Pg values. The porosity determination using the method of immersion (Paw) does not appear to be reliable particularly for the samples having higher porosity.

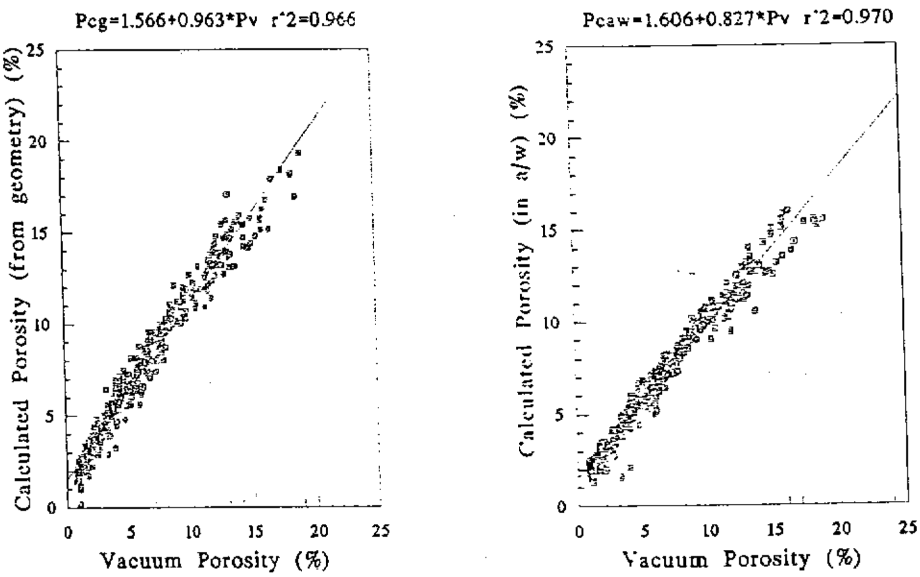


Figure 7.10. Vacuum Porosity vs. Calculated Porosity of asphaltic concrete made with 100 pen base bitumen plus different size of shredded tire rubber Pg (left), Paw (right).

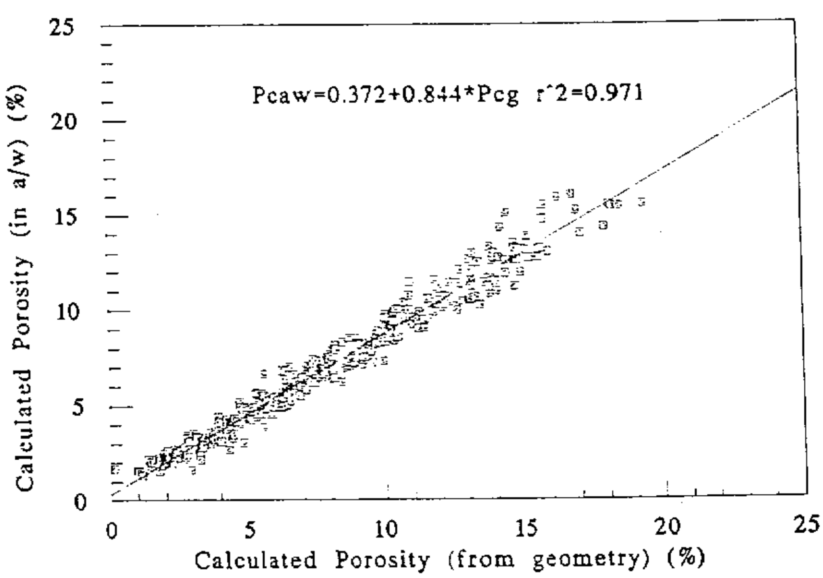


Figure 7.11. Calculated porosity values of rubberised asphaltic concrete (100PBB plus different size of shredded tire rubber in different concentration).

7.5. Summary

Porosity is one of the most important factor effecting the long term performance and durability of bituminous mixture. Basically, the porosity must be high enough to allow for expansion of binder and entrained air under hot weather condition, to provide the mixture with a certain amount of elasticity and to provide safety space for further compaction of the surfacing under heavy traffic. When the porosity is too low, the pavement is likely to flush or bleed. The porosity should not be too high such that it will allow ingress of air and water to further aggravate disintegration.

The accurate measurement of the porosity of compacted bituminous mixtures is relevant in the context of their long term performance and durability. Direct measurements of porosity was made using the Leeds Vacuum Porosimeter. The calculated porosities, obtained either by using geometry of the specimen (Pg) or by using the volume of specimen obtained by water displacement (Paw) are compared together with the measured vacuum porosity (Pv) values for the range of binder contents of the asphaltic concrete made with 50 pen. and 100 pen. grade base bitumens. The relationship between the three porosity values will give a better idea of the porosity and pore structure characteristics of the bituminous mixtures. The unconnected porous in the compacted mixture found was 2.3 % for the asphaltic concrete made with 50 pen. base bitumen plus different size of shredded rubber in different concentration. This value was low for the specimen made with 100 pen. base bitumen 1.6 %.

CHAPTER EIGHT

CREEP BEHAVIOR OF ASPHALTIC CONCRETE MADE WITH WASTE THREAD TIRE RUBBER MODIFIED BINDERS

8.1. Introduction

Permanent deformation or rutting is a problem that may result in the reduction of service life through disintegration of a pavement structure. Most investigators concentrate on developing mix design methods which permit optimization of the stability of bituminous mixtures. Although they recognize that different requirements are needed depending on the conditions (pavement construction, climate, traffic), hardly any method deals with this aspect in a quantifiable and rational way.

In 1977, Shell presented a paper "Asphalt Pavement Design" (Claessen, *et.al.*, 1977), in which the estimation of permanent deformation in the bituminous layers was incorporated as an integral part of the design procedure. Since the part dealing with permanent deformation is rather unconventional, the development of the procedure took several years, in which a series of publication appeared (Hills, 1973; Hills *et al.*, 1974; Van de Loo, 1974; Uge *et al.*, 1974; Van de Loo, 1976; Barkdale, 1977; Hilster *et al.*, 1977; Van de Loo, 1978; Monismith *et al.*, 1991).

In 1973 Hills presented a physical deformation model for bituminous mixtures. It was assumed that any deformation in the mix is the result of sliding displacements between adjacent mineral particles, separated by thin film of bitumen (Figure 8.1.). In this model, the rate of strain was dependent on the magnitude of the stress, the thickness of the bitumen and bitumen properties.

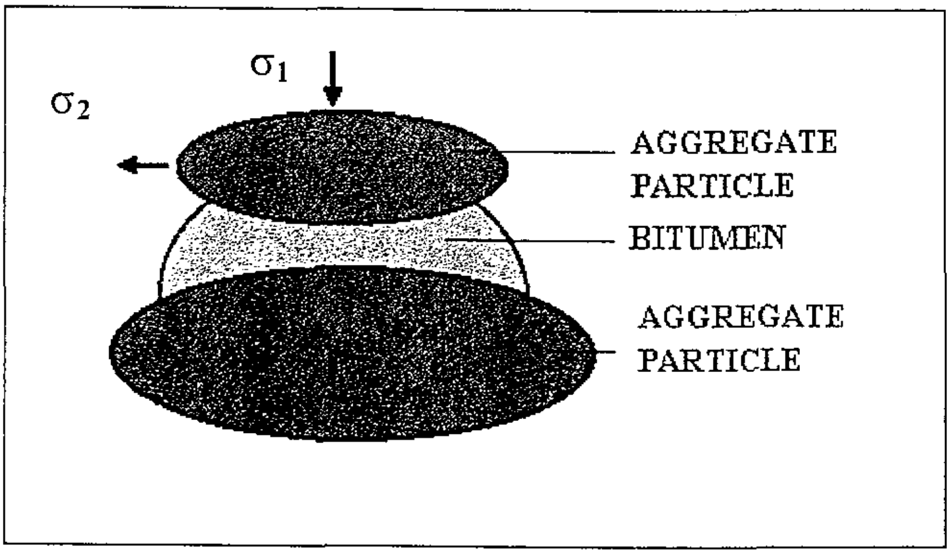


Figure 8.1. Two particles separated by a film of bitumen (Hills, 1974).

Under laboratory test track conditions (single wheel load no lateral distribution) the rut depth calculated by:

$$\textbf{rut depth} = \mathbf{h} \times \frac{\sigma_{av}}{S_{mix}} \qquad (8.1.)$$

where;

h, thickness of the layer,

σ_{av}, average stress under the moving wheel.

On the basis of this approach a large number of bituminous mixtures were tested in static creep test and in various laboratory test tracks, measured and calculated values were compared (Hills *et al.*, 1974). Good agreement was found, although the measured values were somewhat higher than those calculated. The phenomenon was explained by the difference between static and dynamic loading (Van de Loo, 1974). It was recommended that instead of following the more complex dynamic mode of testing to use the simple static creep test and introduce an empirical correction factor *Cm*, which depends on the type of mixture (see Table 8.1).

$$\textbf{rut depth} = \mathbf{C_m} \times \mathbf{h} \times \frac{\sigma_{av}}{\mathbf{S_{mix}}} \qquad \textbf{(8.2.)}$$

Table 8.1. Correction Factors for Dynamic Effects (Van de Loo, 1974).

Mix Type		Cm
Open	Sand sheet and lean sand mixtures Lean open Asphaltic Concrete	1.6-2.0
↑	Lean Bitumen Macadam	1.5-1.8
↓	Asphaltic Concrete Gravel Sand Asphalt Dense Bitumen Macadam	1.2-1.6
Dense	Mastic Types Gussasphalt Hot Rolled Asphalt	1.0-1.3

Barksdale (1977) defined permanent deformation as the progressive accumulation of plastic strains in each layer of the pavement system that occurs under each load repetition.

According to Monismith *et al.* (1991) permanent deformation in a bituminous layer may be caused by densification or volume change and shear deformation which is plastic deformation without any change in volume.

According to Dawlew *et al.* (1990), rutting of bituminous pavement can be classified into three categories;

-Wear rutting which is caused by the combined influence of environment and traffic. In this type of rutting, coated aggregates are lost from the pavement surface,

-Structural rutting which is caused by a repeated traffic loading. Traffic loading cause a permanent vertical deformation in the pavement structure,

-Instability rutting which is caused by inadequate resistance of the structure to resist the stresses imposed upon it, particularly by frequent repetition of high axle loading.

Permanent deformation may occur in all different layers. The wearing course layer is of main concern in this investigation. The creep test has been developed into a practical method in which the resistance to permanent deformation of different bituminous mixtures can be assessed and compared. Based on the above classification, the results obtained from the creep test can be used to estimate the structural rut depth (second type) of a pavement. In this study, the static creep test was performed on standard 101.6 mm. ∅ 63.5 mm. height specimens using the CANIK-UL creep testing machine. Detailed information on testing procedures can be found in reference Cabrera and Nikolaides (1987). A brief description regarding the apparatus and testing procedure has been given in Chapter 4.

8.2. Test Conditions

The measurements of permanent deformation on the different mixes were carried out by application of static load on the CANIK-UL creep testing machine. The CANIK-UL creep testing machine is developed in the University of Leeds. It is shown in Plate 8.1..

At the International Colloquium on plastic deformability of bituminous mixtures held at Zurich (Colloquium 77, 1977), creep testing conditions were standardised at an axial stress of 0.1 MPa for 60 minutes at a temperature of 40 °C.

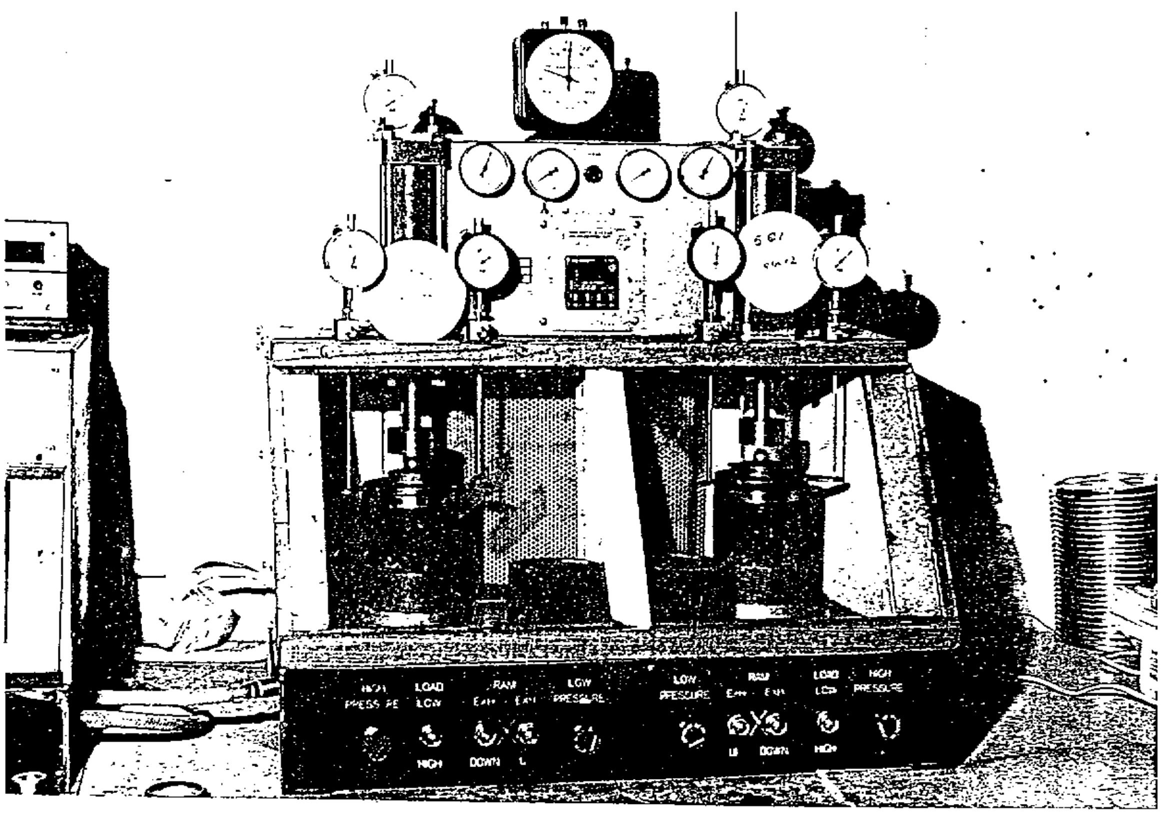


Plate 8.1. CANIK-UL Creep Testing Machine.

8.3. Sample Preparation

In this investigation, resistance to permanent deformation was measured by the application of static creep load on cylindrical standard 101.6 mm. ∅ 63.5 mm. height specimens. These were prepared as described in Chapter 3. The aggregate gradation used in the mixture was DCT result. Four standard specimen were produced for each mix using the Gyratory Testing Machine (GTM). Two of them were used for the measurement of creep characteristics of waste shredded rubber modified asphaltic concrete and the other two for the Marshall stability and flow.

As it was mentioned earlier, the samples containing biggest rubber sizes were very weak. Therefore, the creep test was carried out on the samples made with fine size of rubber (40's) in concentration of 2, 5, 10 and 20%, in 50 and 100 pen. grade base bitumen, and middle sizes (12/16's and 12's) in concentration of 2 and 5%, in 50 and 100 pen. grade base bitumen. The creep stiffness of the mixture made with 12's size rubber in concentration of 10 and 20%, in 50 pen. base bitumen are also measured to predict the creep behavior of the mixture with increasing rubber sizes.

8.4. Test Results

The results of creep test presented in this section are the average values obtained from two specimens. The main variables investigated on the creep properties of asphaltic concrete are the effect of rubberised binder made with 50 and 100 pen. base bitumen and rubber size and concentration.

Figure 8.2. shows the relationship between strain and loading time for the asphaltic concrete, made with 50 pen. base bitumen and 50 pen. base bitumen plus 2% 40's size shredded rubber for the aggregate type of DCT at different binder content.

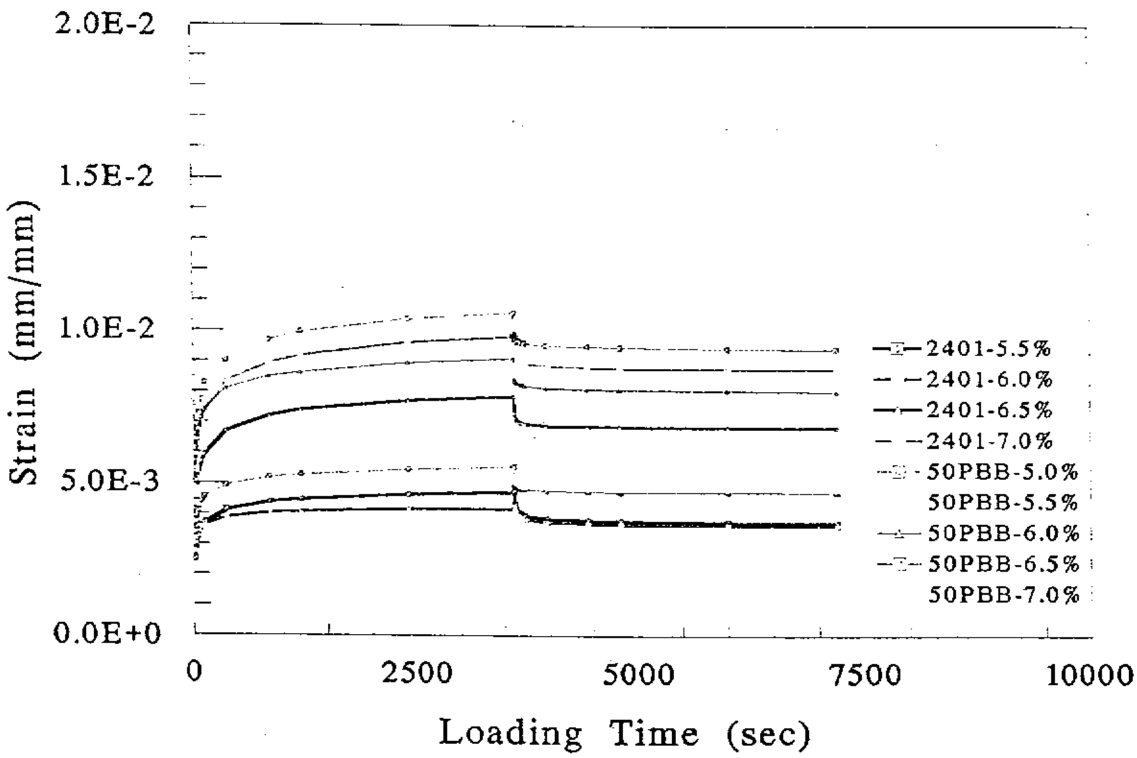


Figure 8.2. Strain vs. Loading Time of asphaltic concrete made with 50 pen. grade base bitumen and 50 pen. base bitumen plus 2% 40's size shredded rubber.

The effect of shredded tire rubber on the creep properties of asphaltic concrete may also be assessed by a graphical double logarithmic plot of Mix Stiffness against Bitumen Stiffness.

In the Leeds Design Method (Cabrera, 1992), this log-log relationship is used to characterize resistance to permanent deformation of bituminous mixtures by using the slope of curve vs. binder content or the maximum mix stiffness at 1 hour loading vs. binder content The minimum slope value or the maximum stiffness value at 1 hour loading time are decided the mixture will perform better. This manner of characterizing a mix is based on the fact that more resistant mixtures have stiffness that are larger and decrease less rapidly with increasing time.

Figure 8.3 and 8.4 show Mix Stiffness vs. Bitumen Stiffness of the asphaltic concrete made with 50 and 100 pen base bitumen respectively. A linear regression line can be fitted to the results of all the mixes with the correlation coefficient of over 88%.

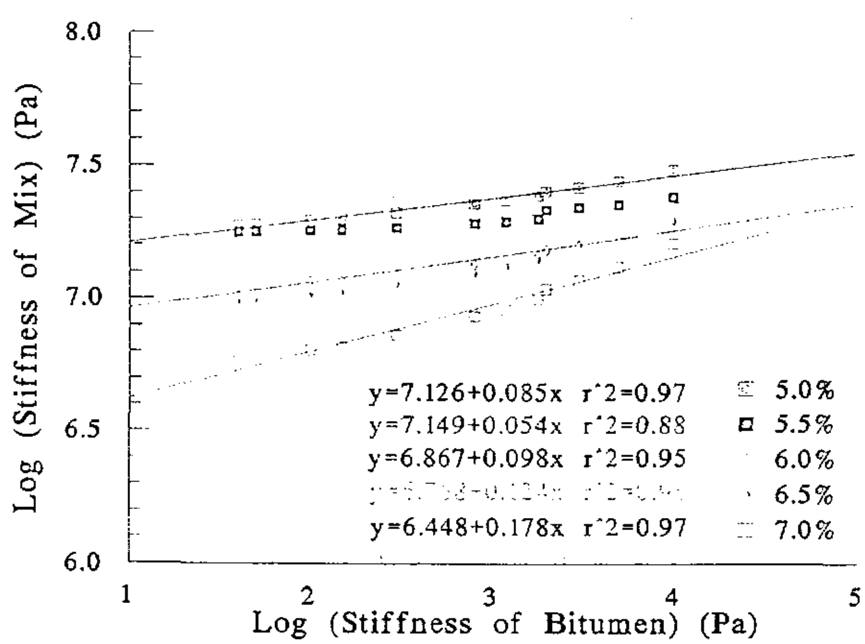


Figure 8.3. Stiffness of Mix vs. Stiffens of Bitumen of asphaltic concrete made with 50 pen. grade base bitumen for the aggregate type of DCT results.

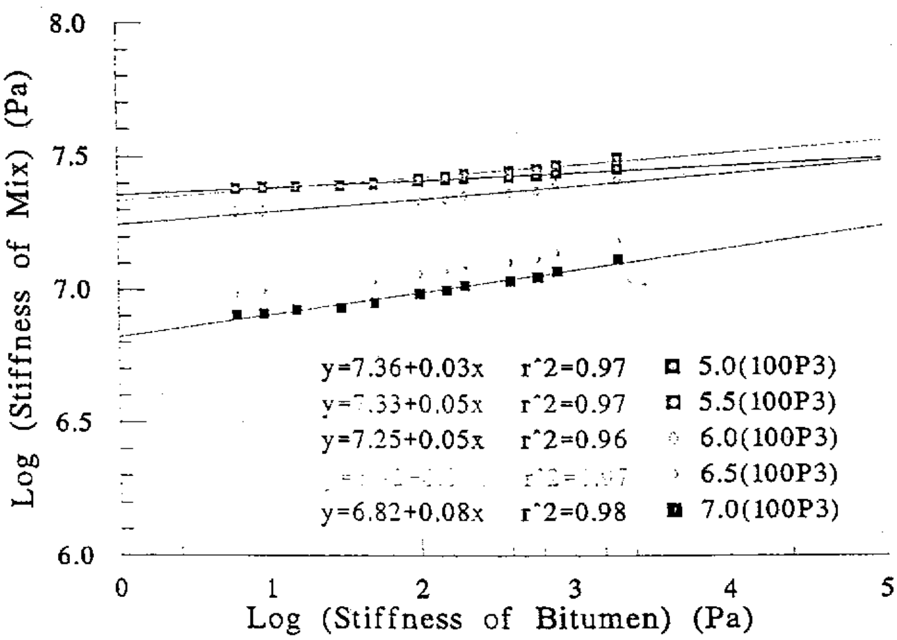


Figure 8.4. Stiffness of Mix vs. Stiffens of Bitumen of asphaltic concrete made with 100 pen. grade base bitumen for the aggregate type of DCT results.

The stiffness of the base bitumen was predicted by using Van Der Poel nomograph. The Van der Poel nomograph is given in Figure 8.6.. Detailed information to predict the bitumen stiffness using Van der Poel nomograph can be found elsewhere (Shell, 1990). Although, it is not known whether the nomograph can be reliably used for the prediction of modified bitumen stiffness, most researchers still use it. In this study, a comparison is made between the values of binders stiffness predicted from the nomograph and values obtained directly from the oscillatory shear analysis, which were discussed in Chapter 5. The bitumen stiffness values for 50 and 100 pen. grade base bitumen obtained from the Van der Poel nomograph are used to select the closest values of frequency on the Frequency vs. Stiffness of Bitumen graph (Figure 8.5).

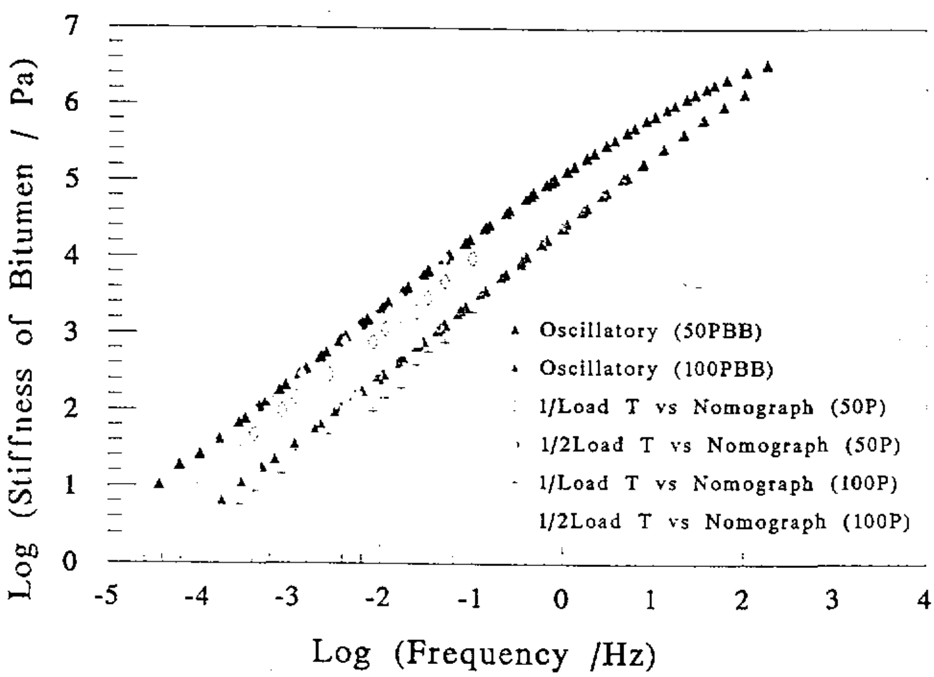


Figure 8.5. Stiffness of Bitumen measured with Oscillatory Shear Analysis and Stiffness of Bitumen obtained from Van Der Poel Nomograph vs. Frequency (for 50 and 100 pen. grade base bitumen).

Table 8.2. Equivalency of Loading Time to Frequency.

load time (sec)	1/Load T. (Hz)	1/Load T. (Hz)(log)	50 pen. (obtained)	100 pen (obtained)	1/2 Load T. (Hz)(log)
10	0.1	-1	-1.314	-1.144	-1.301
20	0.05	-1.301	-1.655	-1.554	-1.602
30	0.033	-1.477	-1.904	-1.678	-1.778
40	0.025	-1.602	-2.098	-1.856	-1.903
60	0.0167	-1.778	-2.148	-2.160	-2.079
90	0.0111	-1.954	-2.338	-2.286	-2.255
120	0.0083	-2.079	-2.526	-2.462	-2.380
360	0.00278	-2.556	-2.971	-2.763	-2.857
840	0.00119	-2.924	-3.279	-2.982	-3.225
1200	0.000833	-3.0791	-3.457	-3.276	-3.380
2400	0.000417	-3.380	-3.755	-3.502	-3.681241
3600	0.000278	-3.556	-3.853	-3.669	-3.857332

Table 8.2. shows the obtained equivalence of loading time to the frequency. The logarithmic frequency values, at certain loading time, was calculated from the best curve fit equation, which was obtained during oscillatory shear analysis. Figure 8.5. and Table 8.3. show the stiffness values for 50 and 100 pen. grade base bitumen in comparison with the obtained values from oscillatory shear analysis and the nomograph. 1/2 loading time is assumed as best approach for frequency values to get the stiffness values from the oscillatory shear analysis.

Table 8.3. Stiffness of Bitumen obtained from Oscillatory Shear Analysis for different loading time.

	Stiffness of Bitumen (Pa)					
time (sec)	50 PBB 1/Load T.	50 PBB 1/ 2 Lo. T.	50 PBB nomograf	100PBB 1/Load T.	100PBB 1/ 2 Lo. T.	100PBB nomograf
10	18620.87	10266.11	10000	2754.23	1403.12	2000
20	10266.11	5589.52	5000	1403.12	711.83	800
30	7204.73	3894.09	3000	943.88	477.68	600
40	5589.52	3005.43	2000	711.83	359.62	400
60	3894.09	2078.53	1800	477.68	240.74	200
90	2701.32	1431.35	1200	320.10	160.93	150
120	2078.53	1095.65	800	240.74	120.82	100
360	748.998	387.06	300	80.57	40.17	50
840	333.64	169.80	150	34.39	17.06	30
1200	236.05	119.36	100	23.99	11.87	15
2400	119.36	59.60	50	11.87	5.85	9
3600	79.63	39.48	40	7.85	3.86	6

From the results obtained it can be clearly seen that the loading type is half a sin wave in creep testing and in the field and the loading time used in oscillatory shear analysis is full sin wave.

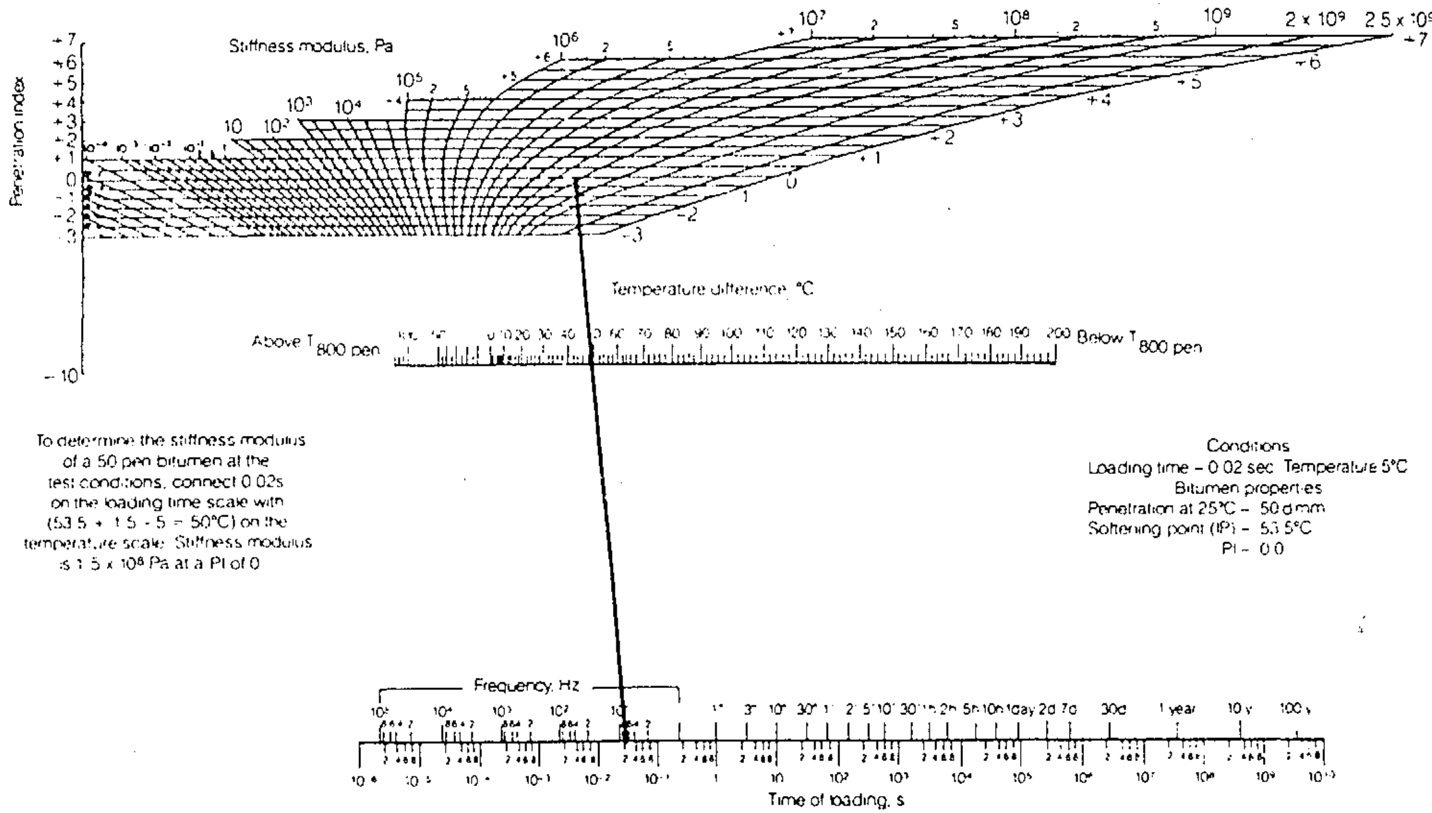


Figure 8.6. Nomograph for determining the Stiffness of Bitumen (Shell, 1990).

8.4.1. The Effect of Rubber

The shredded rubber modified bitumen used to evaluate the effect on the static creep test was limited to the following composition;

1. 50 pen. base bitumen + 2, 5, 10 and 20% 40's size shredded rubber,
2. 50 pen. base bitumen + 2 and 5% 12/16's size shredded rubber,
3. 50 pen. base bitumen + 2, 5, 10 and 20% 12's size shredded rubber,
4. 100 pen. base bitumen + 2, 5, 10 and 20% 40's size shredded rubber,
5. 100 pen base bitumen + 2 and 5% 12/16's size shredded rubber,
6. 100 pen. base bitumen + 2 and 5% 12's size shredded rubber.

The mixtures made with biggest size rubber (3's) were eliminated because of the weakness. Figures 8.7. (a), 8.7. (b), 8.8. (a), 8.8. (b), 8.9., 8.10., 8.11.(a), 8.11.(b) and 8.12. show the effect of rubberised binder concentration made with other three sizes (40's, 12/16's and 12's) in different rubber content on the creep properties of asphaltic concrete tested at 40 °C. It can be seen on the graphs (8.7.(a) and 8.7.(b)), the stiffness of mix increases with the increasing stiffness of binder in a logarithmic relation. Modification of the base bitumen with shredded tire rubber increases the stiffness of binder, however it is not proportional to the stiffness of mix. For instance, the 1 hour stiffness of binder found was 39.5 Pa. for the mixture made with 50 pen. base bitumen and the stiffness of mix was 11 MPa. (Figure 6.13.) at 6.0% binder content. The stiffness of binder decreased from 39.5 Pa. to 36.88 Pa. with the modification of 2% 40's size rubber as the 1 hour stiffness of mix increased 11 MPa. to 23.87 MPa. (Figure 6.13), this increase is over 100%, binder stiffness increases with further increase on rubber concentration and the mix stiffness reduces. For example, for 5% rubber concentration the 1 hour binder stiffness is 53.66 Pa. as the mix stiffness is 15.2 MPa. (Figure 6.13.), for 10% rubber concentration the binder stiffness is 79.71 Pa. as the mix stiffness is 11.3 MPa. (Figure 6.13.) and for 20% rubber concentration the binder stiffness is 382.66 Pa. as the mix stiffness is 12.03 MPa. (Figure 6.13.). However, the value obtained for asphaltic concrete made with 20% 40's size rubber modified binder is even higher than the value obtained for asphaltic concrete made with base bitumen (11 MPa <12.03 MPa).

These values for the asphaltic concrete made with 100 pen base bitumen and rubberized with 40's size rubber at 5.5% binder content are as follows when compared with the rubberised asphaltic concrete made with 50 pen. grade base bitumen. The stiffness of asphaltic concrete made 100 pen. base bitumen modified with 2% 40's size decreased about to half of the mix stiffness values obtained for the asphaltic concrete made with 100 pen. base bitumen. The 1 hour binder stiffness obtained for the 100 pen. base bitumen was 6 Pa. as the mix stiffness was 24.1 MPa. (Figure 6.13.), for the binder made with 2% 40's size rubber the binder stiffness increased to the value of 6.26 Pa. as the mix stiffness halved to the 13.7 MPa. (Figure 6.13.). These values are 8.41 Pa. and 13.8 MPa. (Figure 6.13.) for the 5% 40's rubber, 8.75 Pa. and 18.0 MPa. (Figure 6.13.) for the 10% 40's rubber and 32.16 Pa. and 23.1 MPa. (Figure 6.13.) for the 20% 40's rubber respectively for 5.5% binder content. Similar findings were obtained when the creep test carried out on the asphaltic concrete made with larger sizes.

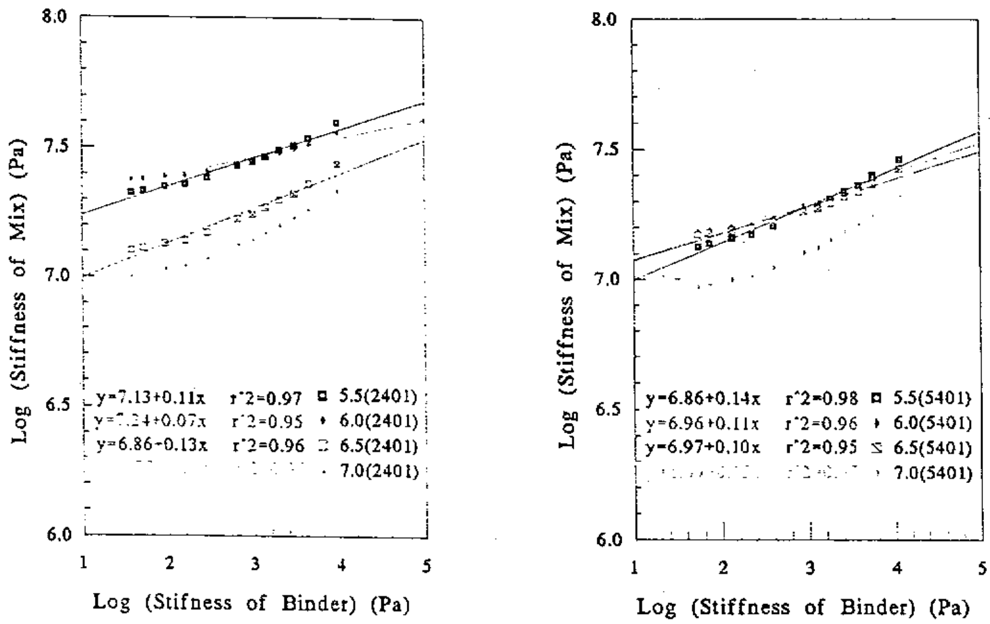


Figure 8.7. (a). Stiffness of Binder vs. Stiffness of Mix for asphaltic concrete made with 50 pen. base bitumen plus 40's size shredded tire rubber 2% (left), 5% (right).

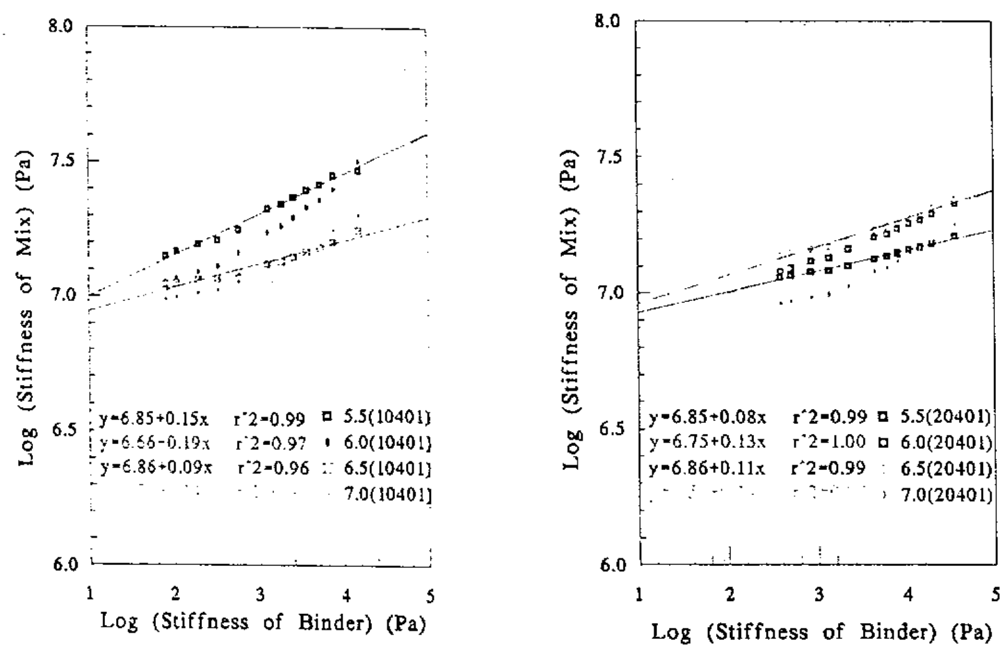


Figure 8.7. (b). Stiffness of Binder vs. Stiffness of Mix for asphaltic concrete made with 50 pen. base bitumen plus 40's size shredded tire rubber 10% (left), 20% (right).

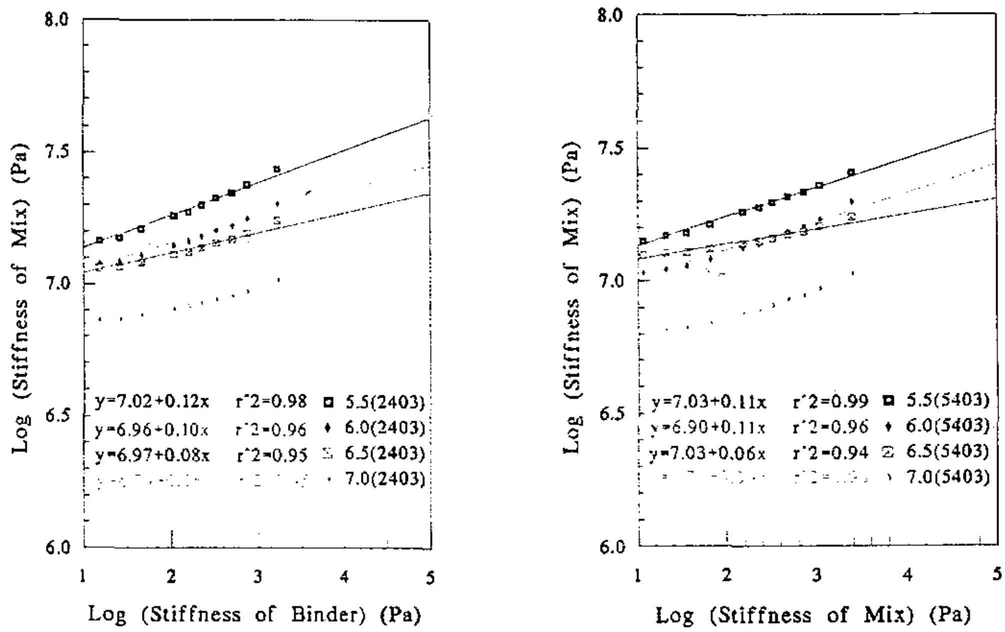


Figure 8.8. (a). Stiffness of Binder vs. Stiffness of Mix for asphaltic concrete made with 100 pen. base bitumen plus 40's size shredded tire rubber 2% (left), 5% (right).

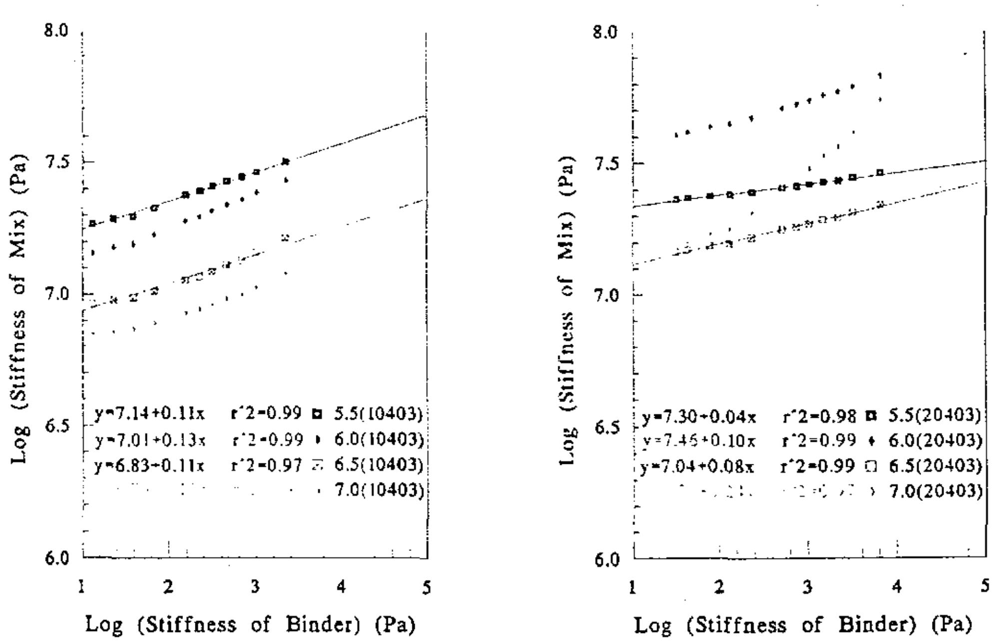


Figure 8.8. (b). Stiffness of Binder vs. Stiffness of Mix for asphaltic concrete made with 100 pen. base bitumen plus 40's size shredded tire rubber 10% (left), 20% (right).

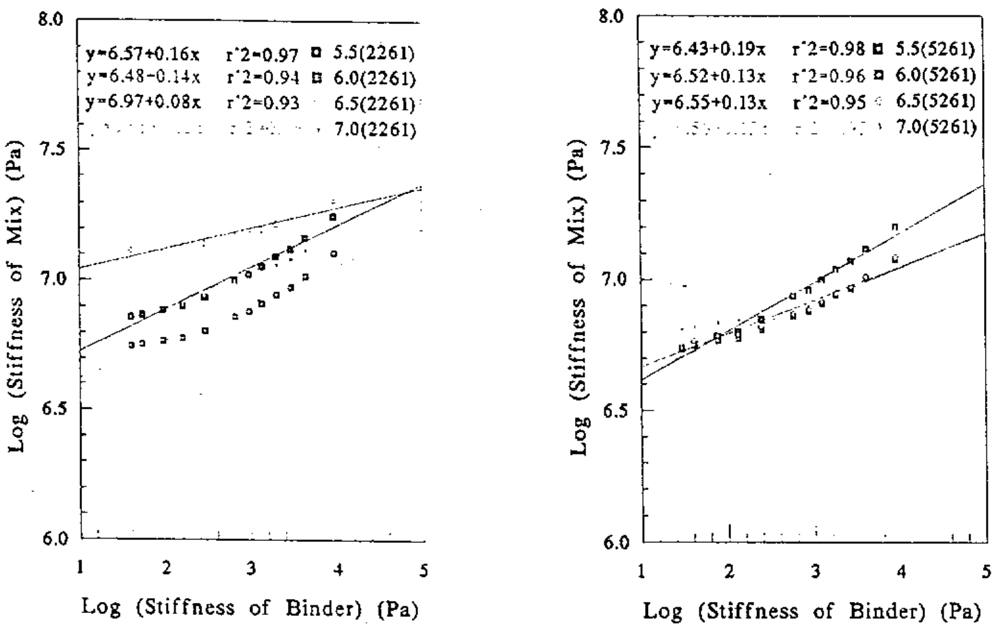


Figure 8.9. Stiffness of Binder vs. Stiffness of Mix for asphaltic concrete made with 50 pen. base bitumen plus 12/16's size shredded tire rubber 2% (left), 5% (right).

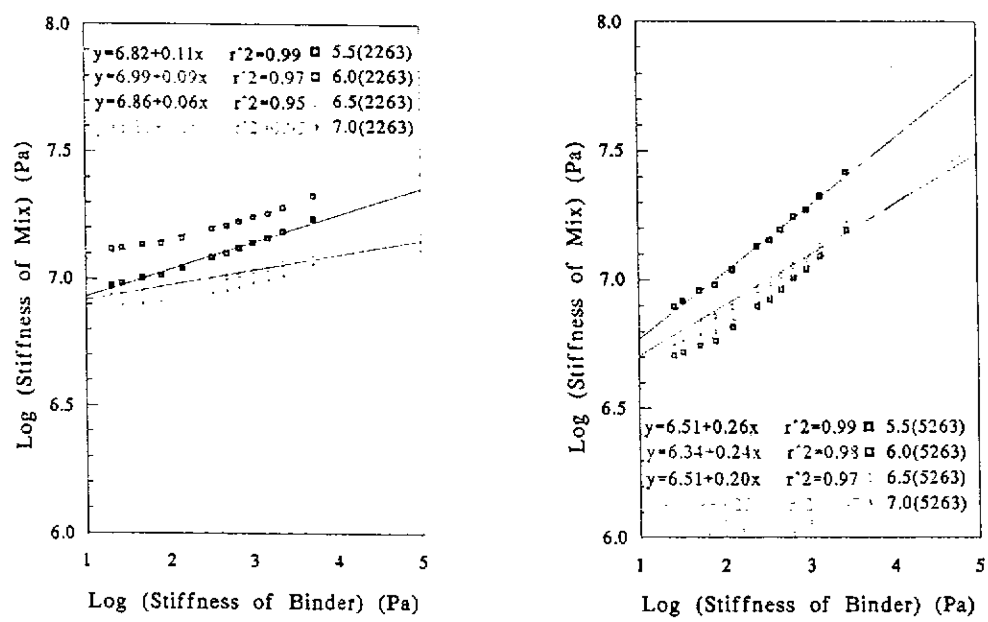


Figure 8.10. Stiffness of Binder vs. Stiffness of Mix for asphaltic concrete made with 100 pen. base bitumen plus 12/16's size shredded tire rubber 2% (left), 5% (right).

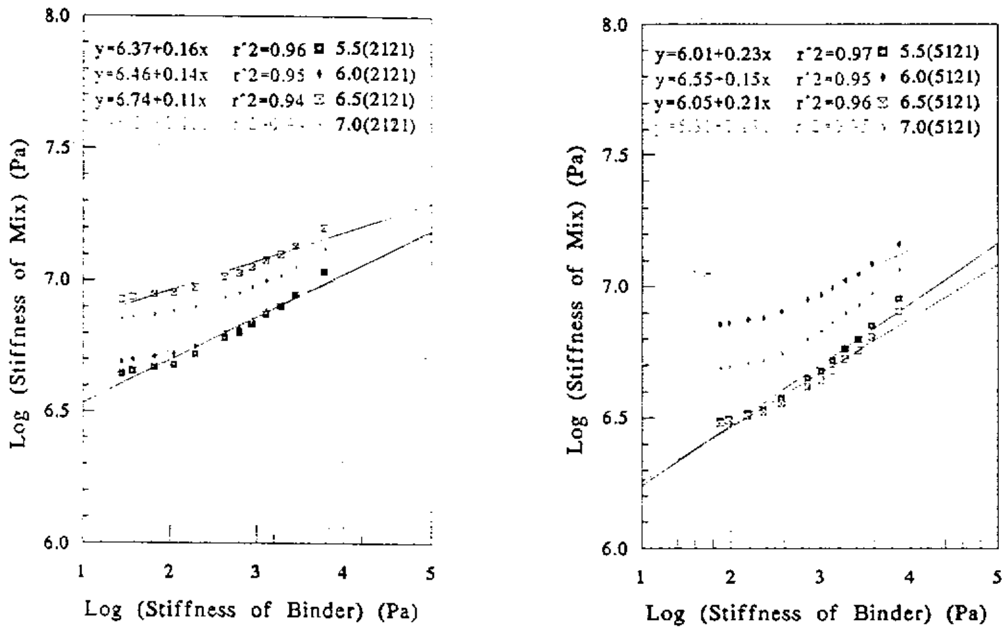


Figure 8.11. (a). Stiffness of Binder vs. Stiffness of Mix for asphaltic concrete made with 50 pen. base bitumen plus 12's size shredded tire rubber 2% (left), 5% (right).

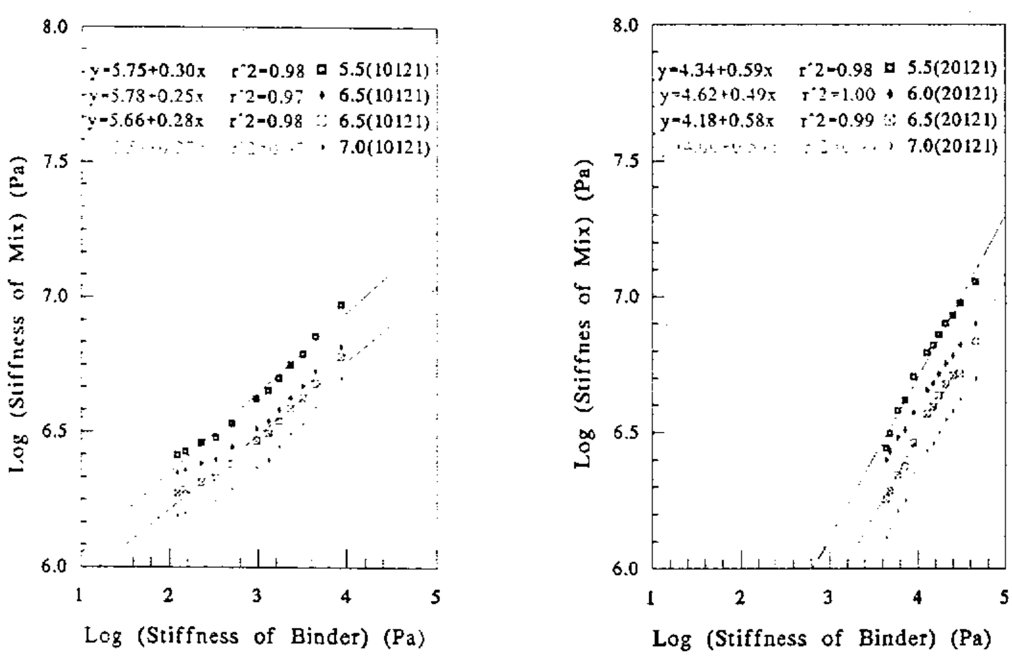


Figure 8.11. (b). Stiffness of Binder vs. Stiffness of Mix for asphaltic concrete made with 50 pen. base bitumen plus 12's size shredded tire rubber 10% (left), 20% (right).

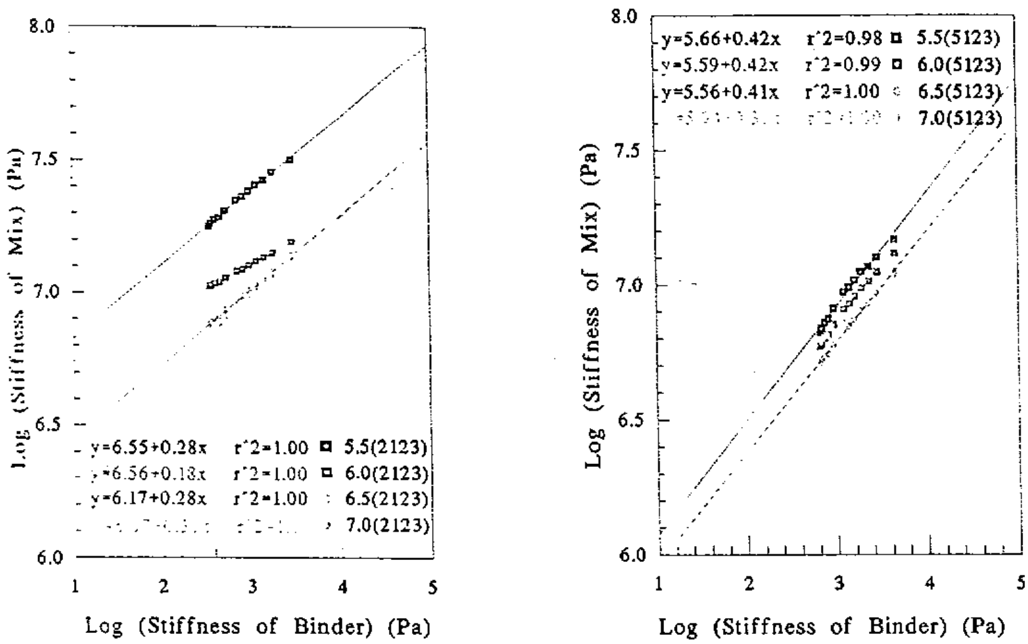


Figure 8.12. Stiffness of Binder vs. Stiffness of Mix for asphaltic concrete made with 100 pen. base bitumen plus 12's size shredded tire rubber 2% (left), 5% (right).

The maximum 1 hour creep stiffness against binder content of the mixtures investigated are given in Chapter 6. A detailed information was also given for the determination of optimum binder content in same Chapter.

Another approach for the determination of creep behavior of the bituminous mixture as mentioned earlier is the slope of the graph in log-log scale drown by Bitumen Stiffness against Mix Stiffness. The slope of the Stiffness of Mix against Stiffness of Binder also indicates the mix susceptibility to the loading time, that is, smaller slope indicates less susceptibility. Nicolaides (1994) named the slope of line as Creep Stiffness Coefficient

(CSC) and expressed as sensitivity of the mixtures to permanent deformation. The lower the CSC is the lesser the sensitivity of the mixture to permanent deformation. The results indicate that the mixes containing high concentration rubber additives binder are high susceptible to the time of loading in comparison to the control mixes. The CSC values are given in the following Figures for the asphaltic concrete made with 40's size rubber in concentration of 2, 5, 10 and 20% with 50 pen. (Figure 8.13.) and 100 pen. (Figure 8.14.) base bitumen, 12's size rubber 2, 5, 10 and 20% with 50 pen. base bitumen (Figure 8.15.) and 12/16's size rubber 2,5% with 50 pen. and 100 pen. and 12's size with 100 pen. base bitumen (Figure 8.16.).

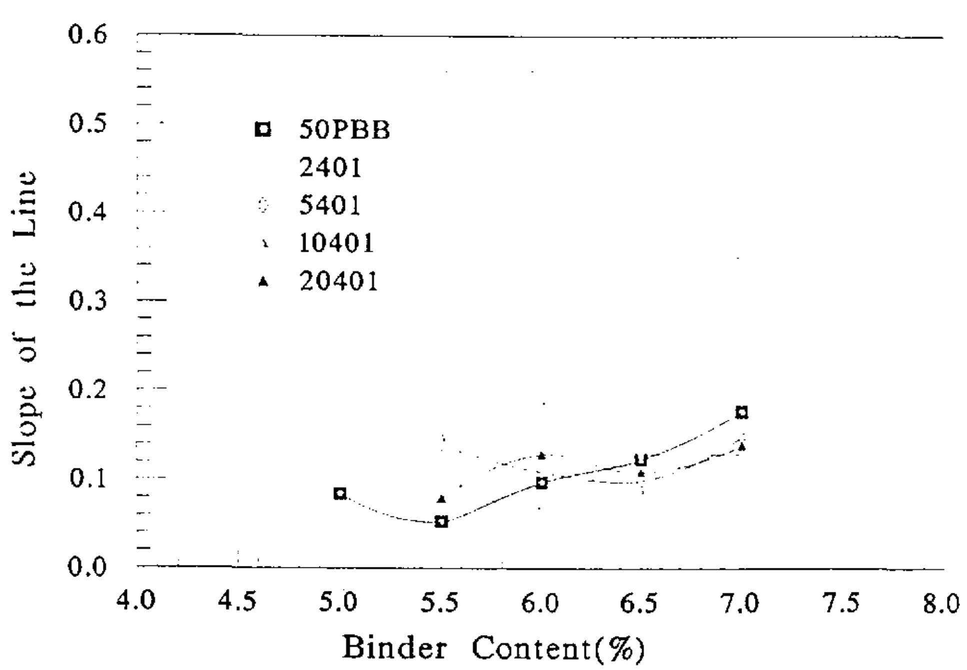


Figure 8.13. Slope of the Line vs. Binder Content for asphaltic concrete made with 50 pen. base bitumen plus 40's size shredded tire rubber and aggregate type DCT results.

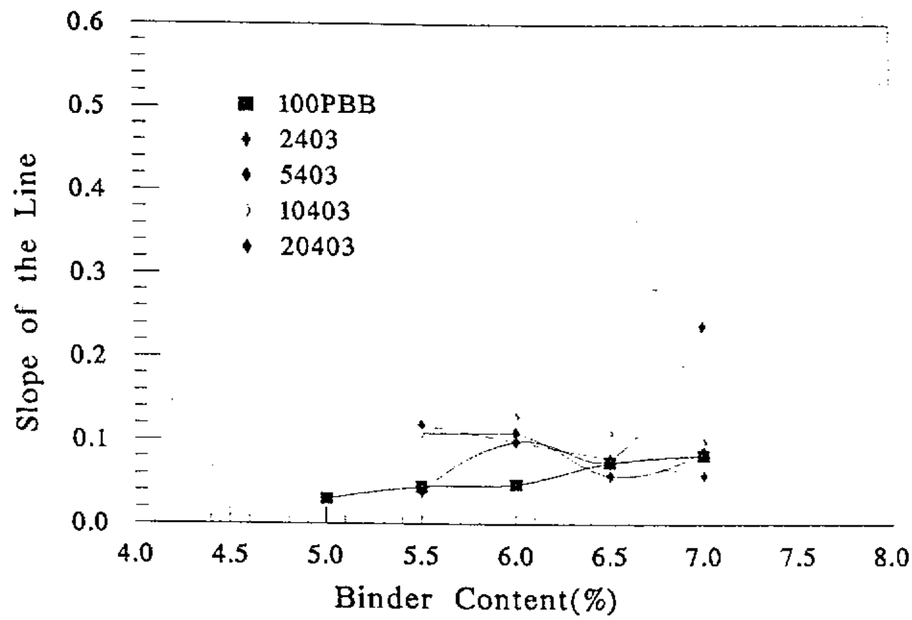


Figure 8.14. Slope of the Line vs. Binder Content for asphaltic concrete made with 100 pen. base bitumen plus 40's size shredded tire rubber and aggregate type DCT results.

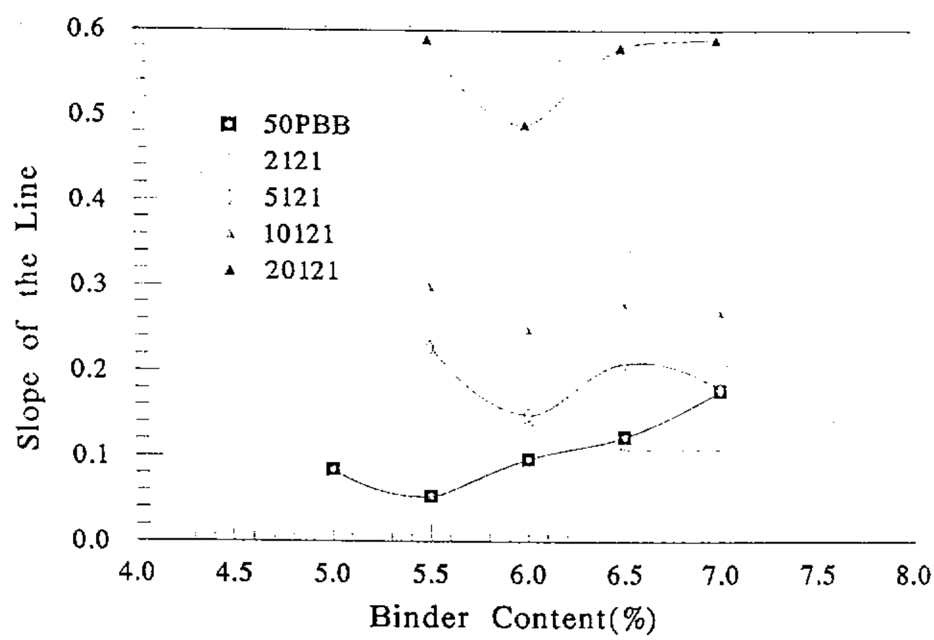


Figure 8.15. Slope of the Line vs. Binder Content for Asphaltic Concrete made with 50 pen. base bitumen plus 12's size shredded tire rubber and aggregate type DCT results.

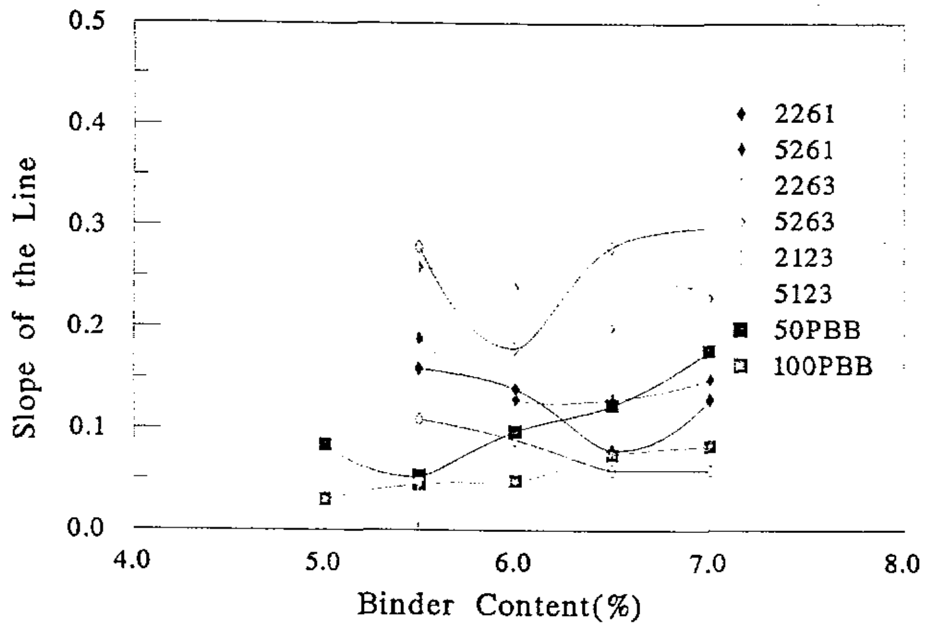


Figure 8.16. Slope of the Line vs. Binder Content for asphaltic concrete made with 12/16's size in 50, 12's and 12/16's size in 100 pen. and aggregate gradation type DCT.

8.5. Summary

A series of creep test were conducted on the shredded rubber modified asphaltic concrete. Analysis of result show that the mixture stiffness increases as bitumen content increases until a maximum value is reached after which the stiffness decreases with increasing binder content. It was noted also that the type of binder greatly affects the stiffness of bitumen. The creep stiffness of the mixture made with 100 pen grade base bitumen was higher than 50 pen. grade base bitumen especially at lower binder content. That may happen due to compaction temperature. Both mixes were compacted at 130 °C. Due to same compaction temperature binders will have different viscosity and the bitumen film thickness on the surface of aggregate particles may be different. Thicker bitumen film around the aggregates will have higher creep deformation. Indeed, when small amount of fine rubber (40's rubber 2% by weight) is incorporated into the bitumen it has been observed that the creep stiffness of the mixture increases over 100%. The addition of high content of rubber, on the other hand, reduces the creep stiffness of the mixture due to high porosity.

CHAPTER NINE

FATIGUE BEHAVIOR OF ASPHALTIC CONCRETE MADE WITH WASTE THREAD TIRE RUBBER MODIFIED BINDERS

9.1. Introduction

Apart from permanent deformation, pavement failure can also be manifested by excessive cracking due to fatigue. Highway pavements are subjected to a repeated passage of wheel loading of varying magnitude and intensity. Fatigue has been defined as the phenomenon of fracture under repeated or fluctuating stress having a maximum value generally less than the tensile strength of the material (Pell, 1971).

Hveem (1955) was the first researcher to indicate that the failure of a pavement could occur due to fatigue. Since then, the matter has been a continuos subject of extensive investigation during the last three decades. A bituminous material subjected to stresses smaller than the breaking strength fractures after subjected to large number of load repetitions. In real life, flexible pavement layers are subjected to continuos flexing under traffic loading. The magnitude of stress is dependent on the overall stiffness and nature of the pavement construction but tensile strains in order of 50 to 200 microstrains have been recorded (Pell, 1986).

Throughout the world, extensive laboratory fatigue studies on bituminous mixtures have been carried out. Experimental works on fatigue properties of bituminous mixtures were performed by Pell (1962), Pell (1971), Epps *et al.* (1972), Vila and Terrel (1975), Pell *et al.* (1975), Van Dijk (1975), Van Dijk *et al.* (1977), Irwin (1977), Ruth *et al.* (1977), Barksdale (1978), Noureldin *et al.* (1978), Bonnaure *et al.* (1980) and Dukatz *et al.* (1980), Gerritsen *et al.* (1988). Meanwhile the studies on the fatigue properties of modified bituminous materials have been reported by Verga *et al.* (1975)., Rostler *et al.* (1977), Meyer *et al.* (1977), Piggot *et al.* (1977), Button *et al.* (1987), Goodrich (1988), Brule *et al.* (1988), Khosla *et al.* (1989), Freeman *et al.* (1989), Nahas *et al.* (1990) and Napiah (1993). However, structured studies on the fatigue behavior of asphaltic concrete made with waste shredded tire rubber modified bitumen is not available. The unique work which performed by Esch (1982) has indicated that the fatigue life of PlusRide pavements may be more than 10 times greater than that of normal mixes (Figure 2.15. in Chapter 2).

Cracks generally initiate at the bottom of pavement layers where tensile strains are highest and propagate towards the surface. Fatigue cracking due to repeated traffic loads in bituminous pavement is dependent upon many factors including the loading patterns, relief or rest periods between applied loads, climatic and environmental factors and mix characteristics. The mix variables affecting the fatigue resistance of bituminous mixture are (Zoorob, 1995);

1-aggregate gradation,

2-binder content,

3-degree of compaction (porosity),

4-mix stiffness.

This chapter examines a laboratory characterisation of the fatigue behavior of rubberised asphaltic concrete and assesses the effect of waste shredded rubber on its fatigue properties.

9.2. Test Methods

Epps *et al.* (1972) divided the variables affecting the fatigue response into three categories, namely load, environmental and mixture variables. Furthermore, Irwin *et al.* (1974) proved that laboratory test method also significantly affects fatigue test results. Baburamani (1992) listed the following laboratory test methods to evaluate fatigue characteristics of bituminous materials;

1-Bending; -third point loading of flexural beam or two point trapezoidal or rotating (a combination of torque and cantilever loading of waisted cylindrical specimens) or wheel tracking,

2- Direct-axial tension-compression or push-pull,

3-Indirect-diametrical.

The above fatigue test methods differ in terms of loading configuration, load pulse wave form, stress distribution within the specimen and specimen shape.

Fatigue test on bituminous specimen are performed either under controlled stress (load) or controlled strain (displacement). It has been suggested (Barksdale, 1978; Texas Institute, 1987) that constant stress mode of loading is applicable to thick bituminous pavements

(>150 mm), whilst the constant strain tests are applicable to thinner bituminous pavements (≤5 mm). It is also widely known that the mode of testing has an influence on the laboratory test results because the specimen response varies according to the input constraint (load or displacement).

In controlled stress testing, a constant load is applied (stress amplitude is constant) and the displacement (deflection) on the specimen is monitored in order that the strain resulting from the applied load can be calculated. In this mode of testing, crack propagation time is very rapid because the strain amplitude increases as the number of cycles of constant load increases and the specimen fracture is not long after fatigue crack initiation. The end point of test is definite, i.e. complete fracture of the specimen. Therefore, in the case of fundamental investigation of mix variables there is some advantage in the controlled stress type of test, which is the one used in this investigation.

The strain amplitude remains constant in the constant strain test (Barksdale, 1978). The required stress gradually decreases to maintain the initial strain amplitude. In this mode of testing, when the crack initiates, to maintain the same strain level, the stress level is quite low. 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 mode of testing requires much more time than controlled stress mode of testing to predict the fatigue life of the bituminous mixtures.

The design criteria for fatigue is generally the maximum tensile strain. Laboratory fatigue tests are based on applying a constant load or displacement for a specific number of repetitions. Ideally, the test may be continued until the specimen actually fails. However, many researchers have defined fatigue failure by monitoring changes in the stiffness modulus (Sharp *et al.*, 1991), with the failure indicated when the stiffness of the sample drops to 50% of the initial value (measured after 100-200 load repetition) (Texas Institute, 1987). In controlled strain mode the fatigue life of a mix also has been defined as the number of repetition at which the force has decreased to half of its initial value (Van Dijk *et al.*, 1977; Moutier, 1988; Moutier *et al.*, 1990).

The load frequency has an effect on the fatigue life of bituminous mixture. Taylor (1968) carried out a fatigue test with a *rotating cantilever testing machine* over a range of frequency between 1.3 and 41.7 Hz.. The results showed that fatigue life is significantly affected at the frequency below 3.3 Hz.. However, Pell *et al.* (1969) indicated that the largest change in fatigue life occurred at frequencies below 32 Hz.. Another work conducted by Deacon and Monismith (1967) showed that frequencies of 0.5 to 1.7 Hz. caused a significant reduction in fatigue life. These differences may arise due to different test method employed by each researcher.

Environmental variables such as temperature, moisture and the alteration of material properties during the service life of the pavement also affect the fatigue performance of bituminous mixtures. The influence of temperature on fatigue properties of a basecourse mix tested in a controlled stress mode has been presented by Pell *et al.* (1969) as shown in Figure 9.1.. In general, if the test was conducted in a controlled stress mode, fatigue life increased as the temperature decreased. In controlled strain mode, the fatigue life increased as the temperature increased.

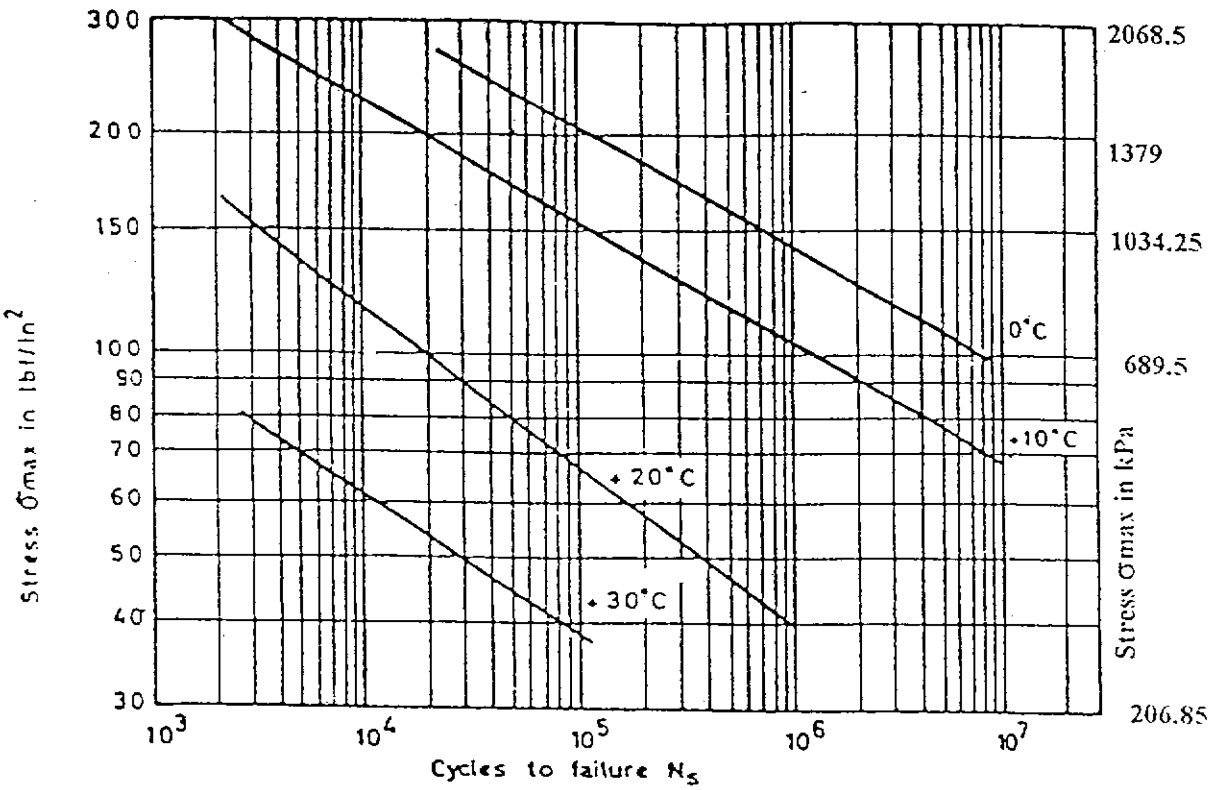


Figure 9.1. Fatigue results of base course mix G specimens at various temperatures (Pell *et al.*. 1969).

Another important loading pattern affecting the fatigue life is the rest periods. Many researchers (Bonnaure *et al.*, 1980; Raithby *et al.*, 1970 and Van Dijk *et al.*, 1977) attempted to quantify the effect of rest period into fatigue testing programmes. Rest periods are a simulation of the loading pattern on a pavement structure under actual traffic conditions. The beneficial effects of rest periods in prolonging fatigue life was confirmed, for instance, a mere one second rest period increased the number of cycles to failure by a factor of 25 when compared with the life under continuos sinusoidal loading (Raithby *et al.*, 1970). However, there is a limit to which the rest period can be increased, above which an additional increase in rest period does not influence the fatigue life. This limit was found to be dependent on test temperature (Raithby *et al.*, 1970).

9.3. Analysis of Fatigue of Bituminous Mixtures

The fatigue resistance of bituminous mixtures is generally defined as the ability of the mixture to respond to repeated traffic loads without significant cracking (Baburamani, 1992). Fatigue failure is often taken as an arbitrary point at which the pavement can no longer perform satisfactorily as a load carrying under repeated load. It is therefore critical that the condition at failure be clearly defined for the particular test effort selected in a laboratory investigation. For instance, the fatigue failure point may correspond to the point at which a certain property of the specimen, such as stiffness modulus, has reduced by a specified proportion, or the point at which crack first occurs and sudden increase in strain value registered. On the other hand, fatigue failure in the field is evidenced by cracking.

It was usually found that a reasonable linear relationship existed between applied stress and cycles to failure when plotted on a log-log basis (Pell *et al.*, 1977 and Cooper *et al.*, 1975). This relationship can be expressed as (Pell, 1962);

$$N = K\left(\frac{1}{\sigma}\right)^{n} \qquad \textbf{(9.1.)}$$

where;

N, the number of cycles to failure obtained at a particular stress level,

σ, maximum amplitude of the applied stress,

K, a constant which determines the position of the line,

n, slope factor of the fatigue line.

When stress is converted to the strain;

$$\varepsilon = \frac{\sigma}{S} \quad (9.2.)$$

where:

σ and ε, maximum amplitude of applied stress and strain respectively,

S, initial dynamic stiffness at a particular stress, temperature and loading time.

It may also be written as a relationship between fatigue life and applied strain which could be consider linear over the range investigated, and be expressed as;

$$N = K_1\left(\frac{1}{\varepsilon}\right)^m \quad (9.3)$$

where;

N, the number of cycles to failure obtained at a particular strain level,

ε, maximum amplitude of the applied strain,

K_1, a constant which determines the position of the line,

m, slope factor of the fatigue line.

The term service life (Ns) and fracture life (Nf) are often related with the definition of fatigue failure point. Service life which is also known as fatigue life, is defined as the number of cycles at which the stiffness modulus of the material has decreased to half of its initial value (Bonnaure *et al.*, 1982). Therefore, with constant strain mode of loading, it is accepted that fatigue life reaches to the end when the stress amplitude has decreased by half, and the strain amplitude has doubled with constant stress mode of loading (Figure 9.2). On the other hand, fracture life is defined as the number of load application required to completely fracture the specimen. Therefore, when the point of failure is defined as a complete fracture, then the term of service life (Ns) is synonymous to fracture life (Nf).

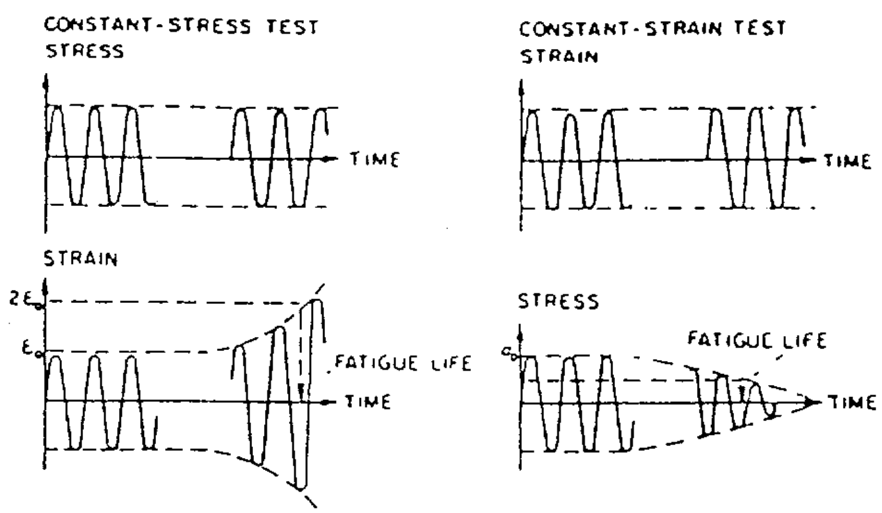


Figure 9.2. Loading signals and response on fatigue testing (Bonnaure *et al.*, 1982).

9.4. Fatigue Behavior of Rubberised Asphaltic Concrete

This section describes the laboratory investigation carried out to investigate the fatigue properties of the waste shredded tire rubber modified asphaltic concrete. To achieve this, fatigue properties are measured in a rectangular beam specimen supported on an elastic rubber base. A strain gauge bonded to the beam was used to measure tensile strain in the beam. The experimental set-up used was developed at the University of Leeds during research carried out by Napiah (1993) and Zoorob (1995). Plate 9.1. shows the fatigue testing set-up used to assess the fatigue performance of the waste shredded rubber modified asphaltic concrete. The constant stress mode of testing is chosen to evaluate the fatigue life of beam specimens. Within this framework of knowledge on the methodology for evaluating the fatigue resistance of asphaltic concrete, the objectives of the test programme are as follows:

1- to determine the effect of waste shredded rubber modified bitumen on the fatigue behavior of asphaltic concrete beam specimens,
2- to evaluate the effect of varying binder content (0.5 per cent above and below the optimum binder content) on the fatigue life of asphaltic concrete.

9.4.1. Sample Preparation

Beam specimens made with 50 and 100 pen. grade base bitumen were prepared to serve as control mixes for two types of aggregate gradation investigated in this study (sea Chapter 3.5.1.)). The shredded rubber modified bitumens used to evaluate the effect on fatigue behavior of asphaltic concrete were limited to the following compositions:

1- 50 penetration grade base bitumen + 2% 40's size shredded rubber,
2- 50 penetration grade base bitumen + 5% 40's size shredded rubber,
3- 100 penetration grade base bitumen + 2% 40's size shredded rubber,
4- 100 penetration grade base bitumen + 5% 40's size shredded rubber.

The materials used in this investigation have been discussed in Chapter 3. Beam specimens were cast in a steel mould with an inner dimension of 100 × 100 × 500 mm. When a beam is tested for fatigue, the presence of a notch serves as a deliberate weakness that enhances the formation and propagation of fatigue cracking. In the Leeds fatigue testing set-up for dense mixes, a notch was engraved underneath each beam using either a lathe machine or hacksaw (Napiah, 1993; Salah, 1995; Ali, 1996). Hamzah (1995) developed a method to give a V shaped notch during casting.

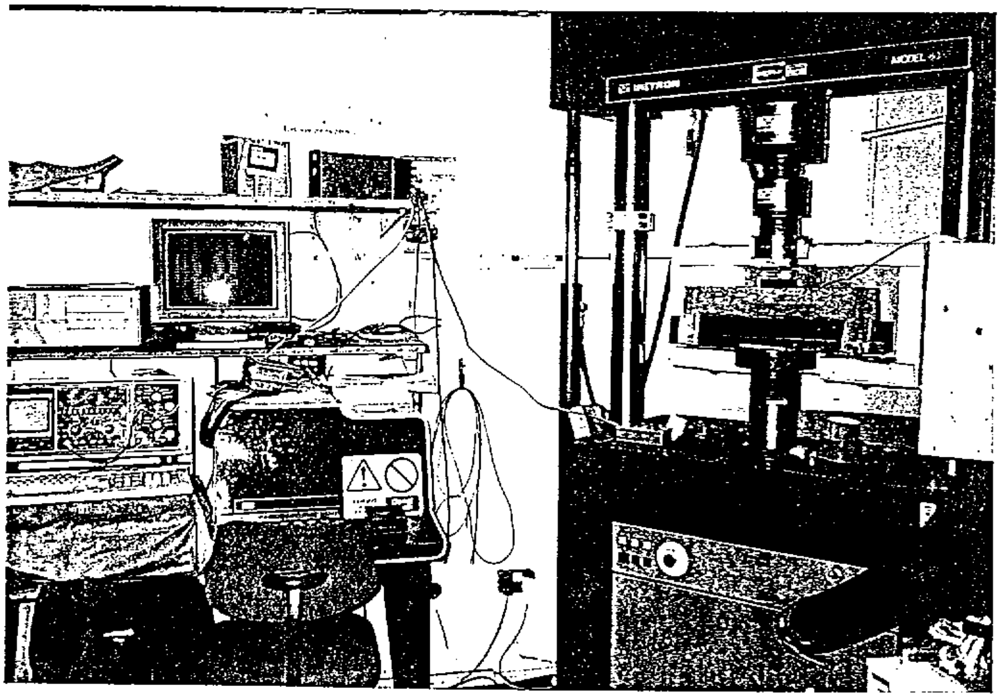


Plate 9.1. INSTRON Fatigue Testing Machine.

A Kango hammer was used with an attached special tamping foot to compact the fatigue beams. A kneading action produced by the tamping foot is applied to give the specimen having aggregate orientation similar to that developed by the GTM in the laboratory and under the action of a roller compaction in the field. Plate 9.2. shows the Kango hammer with tamping foot attached and compaction mould used to prepare the fatigue beams.

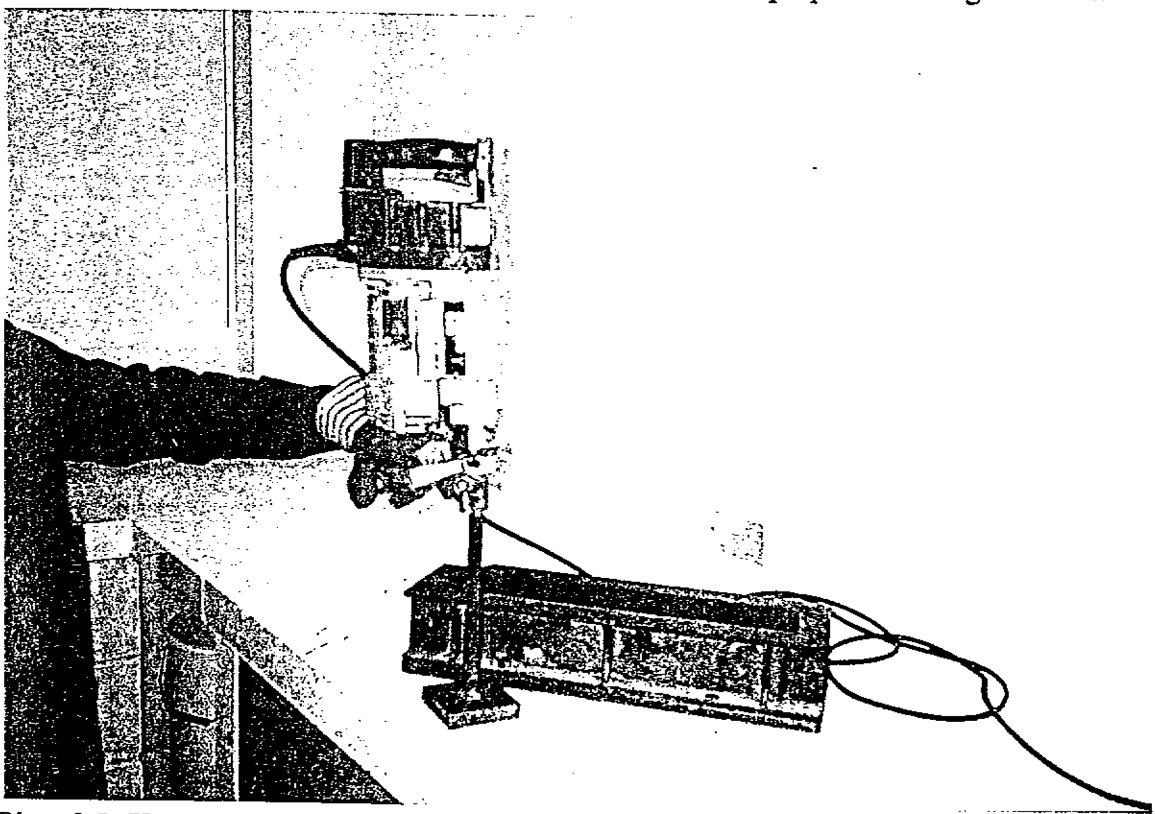


Plate 9.2. Kango Hammer with Tamping Foot Attached and Compaction Mould.

An adequate quantity of material was used to get an equivalent density as that corresponding to the standard specimen compacted by GTM. Each specimen was compacted in four layers. The compacted specimen was then sliced into two identical 40 × 100 × 500 mm. rectangular beams and labeled. Table 9.1. shows the density of the compacted fatigue beams in comparison with the specimens compacted by the GTM.

A PI-60 mm strain gauge (120 Ω) was used to monitor the strains. The strain gauge must be glued on to a smooth flat surface. A quick set epoxy glue was used to make smooth the surface of the sample. The strain gauge was glued 10 mm above the bottom and at the center of the beam above the notch. The beams were then placed on a flat clean surface free of stone particles to avoid inducing tensile strains prior to testing.

Table 9.1. Density of The Compacted Fatigue Beams in Comparison With Standard Specimen.

	50PBB1			50PBB2			2401			5401		
BC (1) (%)	5.4	5.9	6.4	5.1	5.6	6.1	5.6	6.1	6.6	5.7	6.2	6.7
D_1 (gr/cc) (2)	2.34	2.38	2.43	2.36	2.40	2.40	2.39	2.41	2.42	2.38	2.40	2.41
D_2 (gr/cc) (3)	2.37	2.42	2.42	2.39	2.43	2.42	2.38	2.42	2.42	2.38	2.42	2.42

	100PBB3			100PBB4			2403			5403		
BC (1) (%)	5.3	5.8	6.3	5.1	5.6	6.1	5.8	6.3	6.8	5.8	6.3	6.8
D_1 (gr/cc) (2)	2.38	2.40	2.36	2.33	2.37	2.40	2.40	2.41	2.42	2.40	2.40	2.41
D_2 (gr/cc) (3)	2.37	2.43	2.44	2.37	2.39	2.41	2.36	2.39	2.41	2.36	2.39	2.41

(1) Binder Content (as a weight)
(2) Density of the fatigue beams (gr/cm^3)
(3) Density of the GTM compaction (gr/cm^3)

9.4.2. Fatigue Testing Set Up

The experimental testing set-up is shown in Plate 9.1.. A half sine shaped repeated loading was applied to the top of the beam. Each beam was loaded at a rate of 3 Hertz with no rest periods and tested at 5 °C (±1). The constant load level varied from 1500 to 2000 Newtons. The beam specimen was supported on a rubber foundation. The beam-rubber interface was greased to eliminate shear forces. The rubber foundation was resting on a steel plate which was secured to the base of the load frame. Loading was applied at the center over the width of the beam via a 60 mm. wide rubber padding. The strain gauge was connected to the strains amplifier unit. Both ends of the beams were strapped to the rubber foundation to prevent uplifting of both ends after prolonged loading. A thermocouple was fixed onto the beam's surface to monitor beam temperature. The whole assembly was than kept enclosed and sealed in an insulated environmental chamber. The chamber has an air inlet and an air outlet to facilitate circulation of cold air supplied by a freezer unit.

Fatigue testing commenced by slowly raising the assembly so that the loading head touched to the steel loading block. Having selected the desired loading level and wave form, the beam was then subjected to repeated loading under controlled stress.

9.5. Results and Discussion

A total of 72 beams from eight different combination of mixes at three different binder content (optimum binder content ± 0.5%) were tested in this investigation. Each mix was tested at three constant stress levels. In all test Bjorklund's (1985) definition of failure was adopted, i.e. in a constant stress loading test, the material behaves more elastically with less permanent displacement after an initial period of comparatively large vertical displacement. An inflection point is eventually reached which is evidenced that the materials resistance to fatigue loading has reduced and micro cracking has developed. The fatigue life is defined as a number of cycles corresponding to this point.

Fatigue test results can be presented in terms of number of cycles to failure as a function of both the load applied to the beam (in constant strain mode of loading) or the maximum tensile strain (in constant stress mode of loading). In a plot relating tensile strain and number of cycles, a linear part of the curve which represents an initial period of large strain and the part that represents a nearly constant rate of strain amplitude was extrapolated. Strain value corresponding to the intersection point of these two extrapolations is defined as the initial strain (Napiah, 1993). Figure 9.3. shows the procedure attempted to determine the initial strain on each beam. However, this procedure was not used in this investigation. Instead of that the following procedure was used:

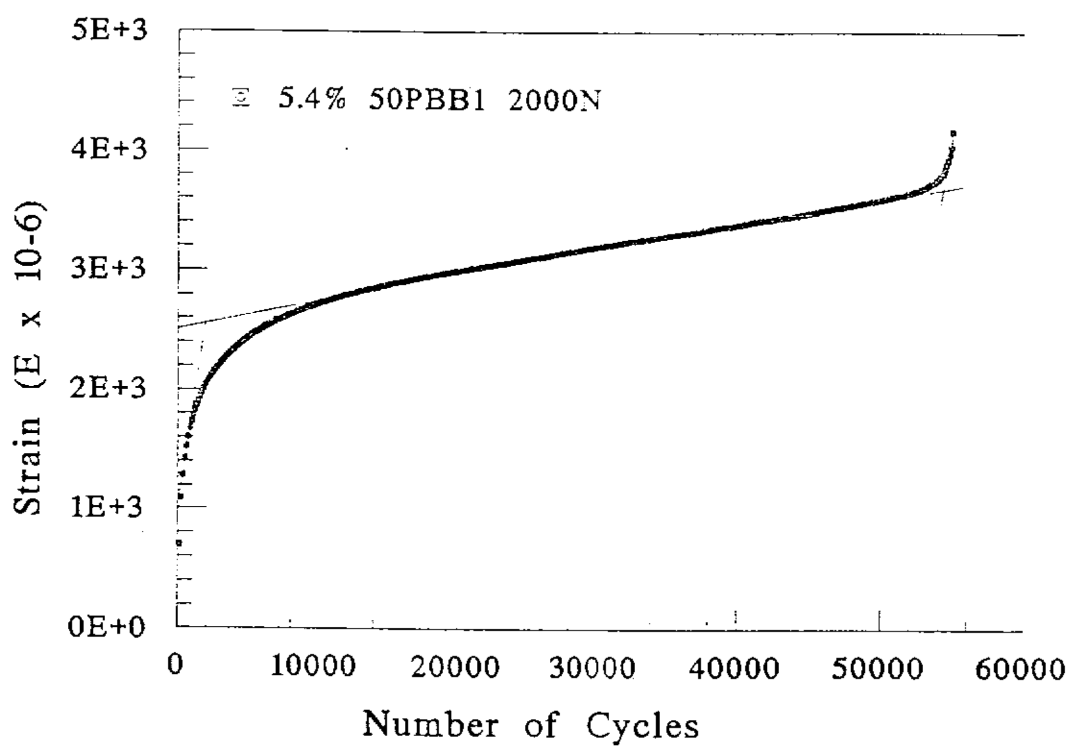


Figure 9.3. Strain vs. Number of Cycles for failure definition.

The initial stiffness of the beam was predicted by using the BANDS computer program (Shell Research BV, 1990) and the initial strain was then calculated dividing the applied stress by the stiffness calculated for the compacted mixture. In the fatigue test used, strain values were captured at 40 second intervals. At 3 Hz. loading frequency readings were obtained at 120 cycles intervals, hence, the initial strain measurement was not available on the fatigue testing set-up. The stress amplitude σ (Pa) is calculated from the equation given below;

$$\sigma = \frac{M \times y}{I} \tag{9.4.}$$

where;

M, bending moment, calculated from Equation (9.5.)(Nm),

y, vertical distance of the strain gauge location from the beam central point (m),

I, moment of inertia of rectangular beam ($I=(1/12)\ bd^3$) (m^4).

For the case of a beam on an elastic rubber foundation being acted upon by a concentrated load P, the bending moment M is calculated from the equation (Hetenyi, 1946) given below;

$$M = \frac{P}{4\left(\dfrac{k}{4.E \times I}\right)^{0.25}} \tag{9.5.}$$

where;

k, modulus of rubber foundation (=4.3 MPa),

E, beam stiffness modulus (MPa),

I, moment of inertia of rectangular beam (m^4).

Beam stiffness modulus was calculated with the aid of the BANDS computer programme (Shell Research BV, 1990) by keying the values of bitumen stiffness, volume of binder and volume of aggregates. The bitumen-rubber stiffness values were calculated by using the oscillatory shear analysis at testing temperature (5 °C) and time of loading (3 Hz). The values used are given in Table 9.2 and in Appendix C. The stiffness of bitumens are normally obtained from the Van der Pool nomograph or by using the first part of the BANDS computer programme (Shell Research BV, 1990). The nomograph and the programme assume that the relationship between viscosity or penetration and temperature is linear at least up to the softening point temperature but as shown in Figure 8.5 the relationship is not linear when dealing with rubber modified binders. The values of binders stiffness calculated from BANDS programme were higher than the that of measured from the oscillatory shear analysis. This difference was as large as 35 MPa, it shows inclusively that the Van der Pool nomograph should not be used when rubber modified bitumen are involved.

As indicated before the initial strain values were calculated dividing the applied stress by the initial stiffness value. During the controlled stress mode of testing, the strain amplitude increases with increasing number of cycles and as a result that the stiffness of the beam decreases. It was decided that the prediction of initial strain obtained from the calculated initial stiffness of the beam was more logic than the predicted values used by early researchers in Leeds (Napiah, 1993; Hamzah, 1995). The fatigue life determination made in this investigation was based on the microcracking. In a plot correlating tensile strain and number of cycles (Figure 9.3), after the initial strain, a constant rate of strain continues up to the number of load repetitions which microcracking produces. This is the point which the rate of strain increases noticeably. The number of cycles corresponding to the intersection point of these two rates of strains is defined as the number of cycles to failure. A summary of the fatigue test results of the asphaltic concrete are given in Table 9.3. Figure 9.4 illustrates the result of fatigue curves of the asphaltic concrete made with 50 pen. base bitumen for the aggregate gradation type DCT investigated in this research.

Table 9.2 Mix Factors Used in the Fatigue Analysis.

Test series	Binder Content	Vagg (%)	Vb (%)	Va (%)	Sbit (MPa)	Smix (MPa)
50PBB1	5.4	81.77	11.58	6.65	214.98	11500
	5.9	82.85	12.98	4.17	214.98	12500
	6.4	82.77	14.20	3.03	214.98	12500
50PBB2	5.1	83.93	11.88	4.19	214.98	13700
	5.6	84.24	13.18	2.58	214.98	14000
	6.1	83.46	14.32	2.22	214.98	13200
100PBB3	5.3	81.83	12.12	6.05	114.37	8390
	5.8	82.15	13.58	4.27	114.37	8640
	6.3	82.29	14.84	2.87	114.37	9250
100PBB4	5.1	81.97	11.64	6.39	114.37	8500
	5.6	83.40	11.95	4.65	114.37	9690
	6.1	82.68	14.24	3.08	114.37	9080
2401	5.6	82.40	12.92	4.68	302.97	14300
	6.1	82.55	14.20	3.25	302.97	14500
	6.6	82.24	15.41	2.35	302.97	14200
5401	5.7	81.90	13.51	4.59	315.5	14100
	6.2	82.20	14.35	3.45	315.5	14500
	6.7	81.98	16.00	2.12	315.5	14300
2403	5.8	80.71	13.54	5.75	156.7	8960
	6.3	81.27	14.75	3.98	156.7	9400
	6.8	81.39	16.02	2.59	156.7	9510
5403	5.8	80.35	13.11	6.54	200.59	9910
	6.3	81.12	14.92	3.96	200.59	10600
	6.8	81.44	15.96	2.60	200.59	10900

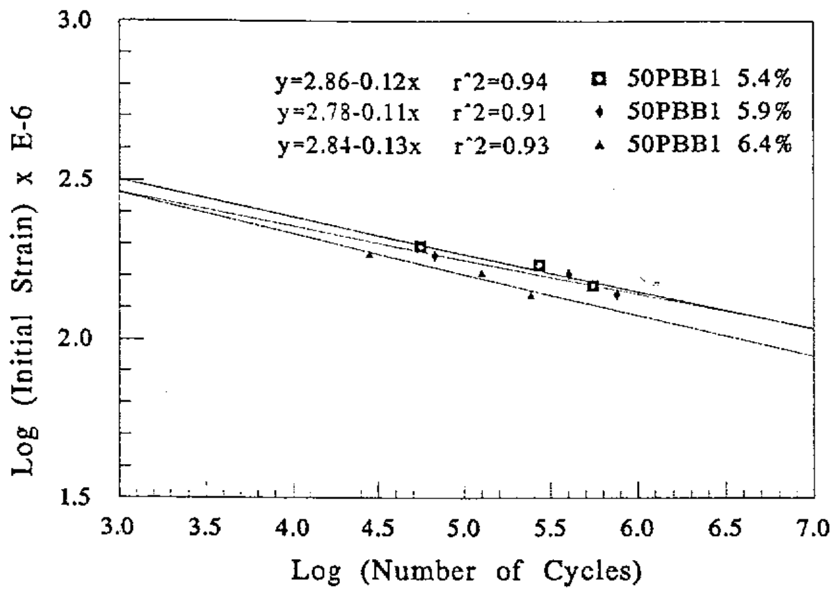


Figure 9.4. Initial Strain vs. Fatigue Life of asphaltic concrete made with 50 pen. base bitumen for the aggregate gradation type DCT result.

A linear relationship exist between the initial strain and fatigue life when plotted in log-log basis. This relationship is in the form of;

$$\log\varepsilon = a - b \log Nf \qquad (9.6.)$$

It can be expressed also as below;

$$N_f = K_1 \left(\frac{1}{\varepsilon}\right)^m \qquad (9.7.)$$

where;

Nf, fatigue life of the beam (number of cycles),

K_1, a constant which determines the position of the line ($=10^{a/b}$),

m, slope factor of the fatigue line (=1/b),

ε, initial strain (μs).

The rest of the fatigue test result curves of the asphaltic concrete are given in Figures 9.5., 9.6., 9.7., 9.8., 9.9., 9.10. and 9.11..

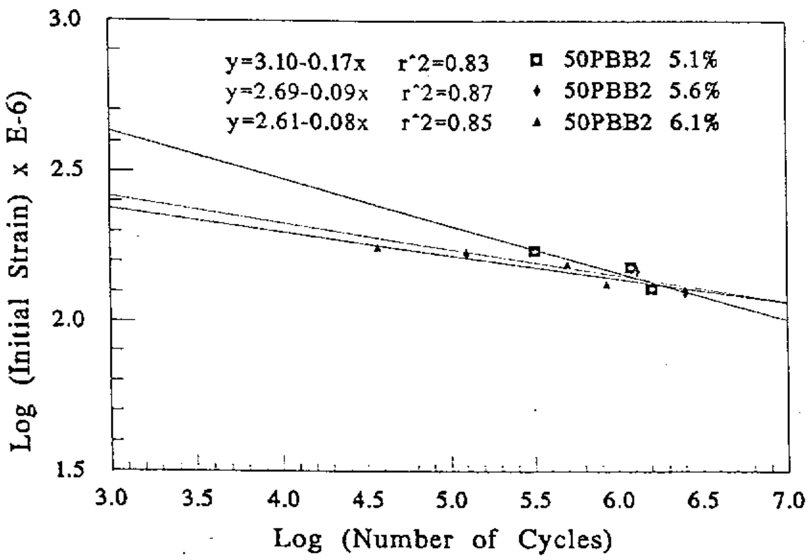


Figure 9.5. Initial Strain vs. Fatigue Life of asphaltic concrete made with 50 pen. base bitumen for the aggregate gradation type equation (3.1.)(n=0.5).

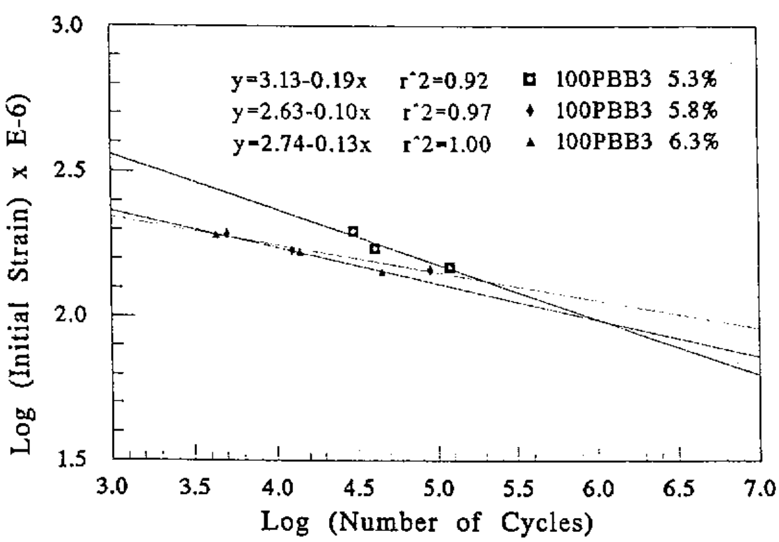


Figure 9.6. Initial Strain vs. Fatigue Life of asphaltic concrete made with 100 pen. base bitumen for the aggregate gradation type of DCT result.

Table 9.3. A Summary of Fatigue Test Results.

Test series	Binder Content	Stress (MPa)	Initial Strain $\times(10^{-6})$	Initial Stiffness (GPa)	Fatigue Life (Nf)
50PBB1	5.4%	1.64	148	11.1	550000
	5.9%	1.68	139	12.1	750000
	6.4%	1.67	139	12.0	242000
	5.4%	1.91	172	11.1	270000
	5.9%	1.95	161	12.1	400000
	6.4%	1.95	163	12.0	125000
	5.4%	2.19	197	11.1	55000
	5.9%	2.23	184	12.1	66000
	6.4%	2.23	186	12.0	28000
50PBB2	5.1%	1.71	129	13.2	1600000
	5.6%	1.72	126	13.6	2500000
	6.1%	1.70	134	12.7	860000
	5.1%	2.00	152	13.2	1200000
	5.6%	2.01	148	13.6	1300000
	6.1%	1.98	156	12.7	500000
	5.1%	2.28	173	13.2	320000
	5.6%	2.30	169	13.6	125000
	6.1%	2.26	178	12.7	37000
100PBB3	5.3%	1.64	148	11.1	120000
	5.8%	1.65	145	11.4	90000
	6.3%	1.65	143	11.5	45000
	5.3%	1.91	172	11.1	41000
	5.8%	1.93	169	11.4	12500
	6.3%	1.93	168	11.5	14000
	5.3%	2.19	197	11.1	30000
	5.8%	2.20	193	11.4	5000
	6.3%	2.21	192	11.5	4300
100PBB4	5.1%	1.64	146	11.2	92000
	5.6%	1.66	142	11.7	122000
	6.1%	1.67	139	12.0	35000
	5.1%	1.92	171	11.2	55000
	5.6%	1.94	166	11.7	90000
	6.1%	1.95	163	12.0	20000
	5.1%	2.19	196	11.2	12000
	5.6%	2.21	189	11.7	42000
	6.1%	2.23	186	12.0	13500

Table 9.3. continued

2401	5.6%	1.75	122	14.3	872000
	6.1%	1.75	121	14.5	950000
	6.6%	1.74	123	14.2	250000
	5.6%	2.04	143	14.3	520000
	6.1%	2.04	141	14.5	500000
	6.6%	2.03	143	14.2	130000
	5.6%	2.33	163	14.3	155000
	6.1%	2.34	161	14.5	226000
	6.6%	2.32	163	14.2	35000
5401	5.7%	1.76	118	14.9	1100000
	6.2%	1.77	116	15.2	1300000
	6.7%	1.77	118	15.0	760000
	5.7%	2.06	138	14.9	450000
	6.2%	2.07	136	15.2	850000
	6.7%	2.06	137	15.0	325000
	5.7%	2.35	158	14.9	33000
	6.2%	2.36	155	15.2	132000
	6.7%	2.36	157	15.0	125000
2403	5.8	1.54	176	8.75	280000
	6.3	1.56	170	9.19	244000
	6.8	1.57	169	9.30	32000
	5.8	1.80	206	8.75	155000
	6.3	1.82	198	9.19	126000
	6.8	1.83	197	9.30	6600
	5.8	2.06	235	8.75	53000
	6.3	2.09	227	9.19	12000
	6.8	2.09	225	9.30	3360
5403	5.8	1.57	168	9.35	1200000
	6.3	1.60	160	9.98	2091000
	6.8	1.61	156	10.30	200000
	5.8	1.83	196	9.35	310000
	6.3	1.86	186	9.98	183200
	6.8	1.88	182	10.30	51000
	5.8	2.09	224	9.35	6800
	6.3	2.13	213	9.98	55000
	6.8	2.15	209	10.30	23800

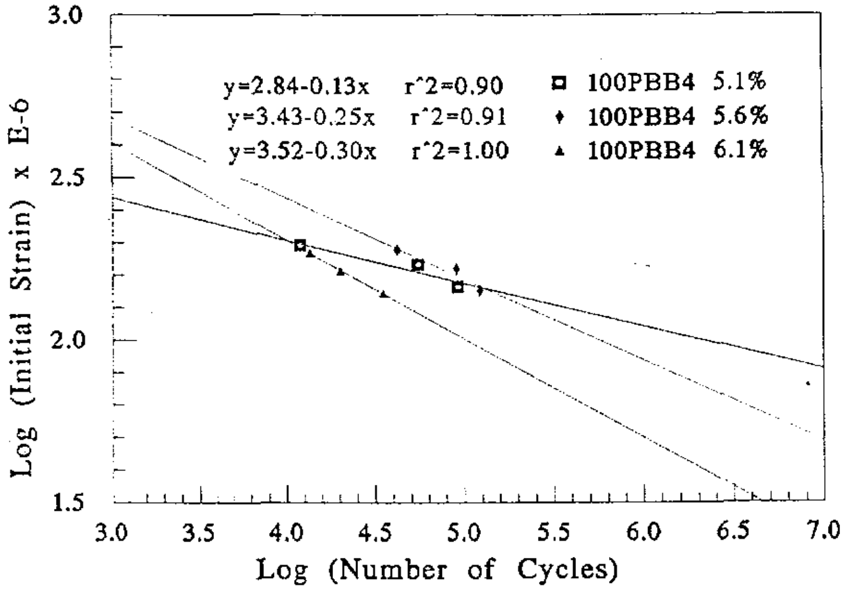


Figure 9.7. Initial Strain vs. Fatigue Life of Asphaltic Concrete Made with 100 pen. Base Bitumen for the aggregate gradation type equation (3.1., n=0.5).

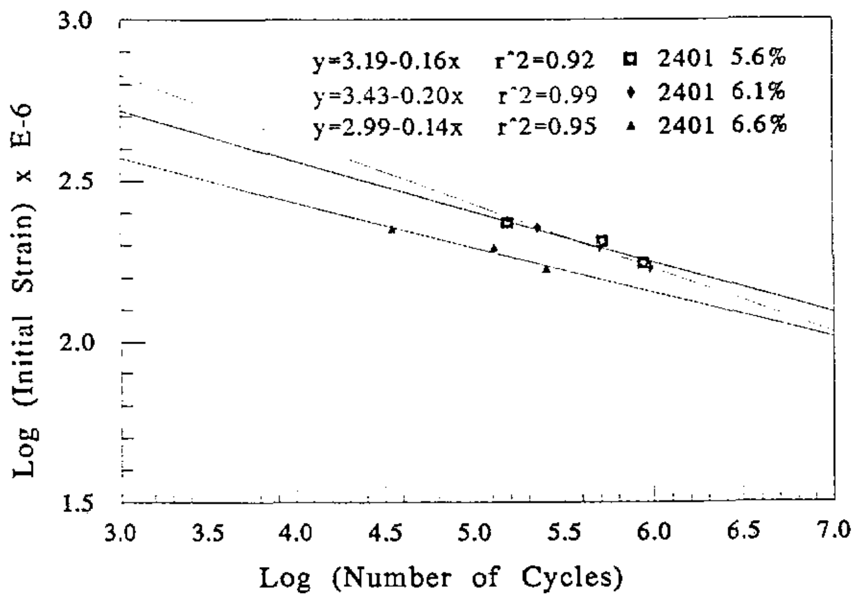


Figure 9.8. Initial Strain vs. Fatigue Life of asphaltic concrete made with 50 pen. base bitumen plus 2% 40's size shredded tire rubber for aggregate gradation type DCT result.

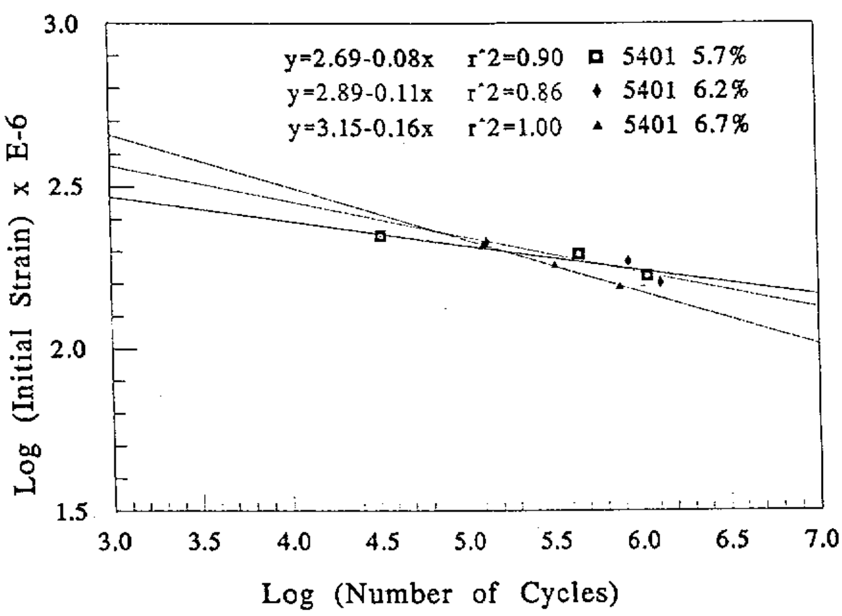


Figure 9.9. Initial Strain vs. Fatigue Life of asphaltic concrete made with 50 pen. base bitumen plus 5% 40's size shredded tire rubber for aggregate gradation type DCT result.

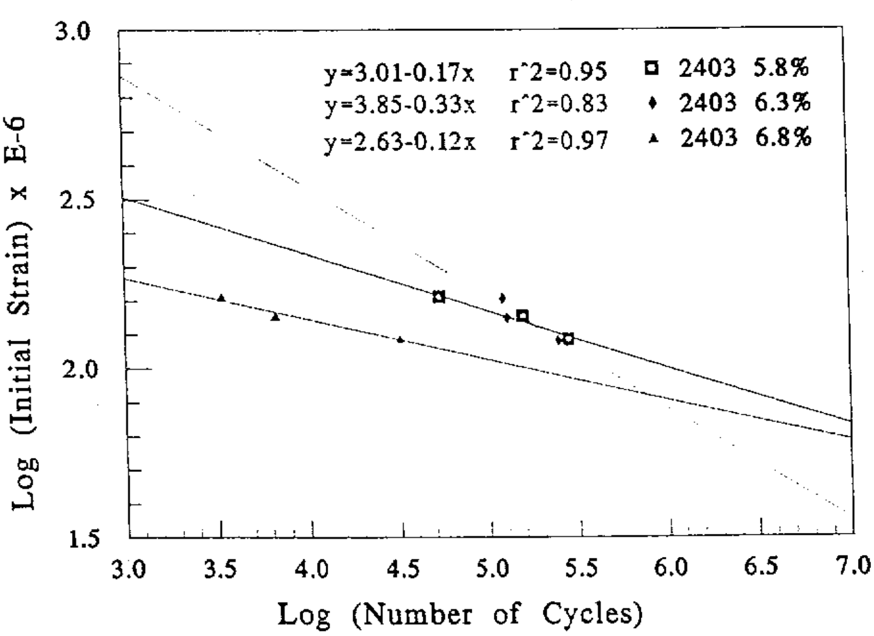


Figure 9.10. Initial Strains vs. Fatigue Life of asphaltic concrete made with 100 pen. base bitumen plus 2% 40's size shredded tire rubber for aggregate gradation type DCT result.

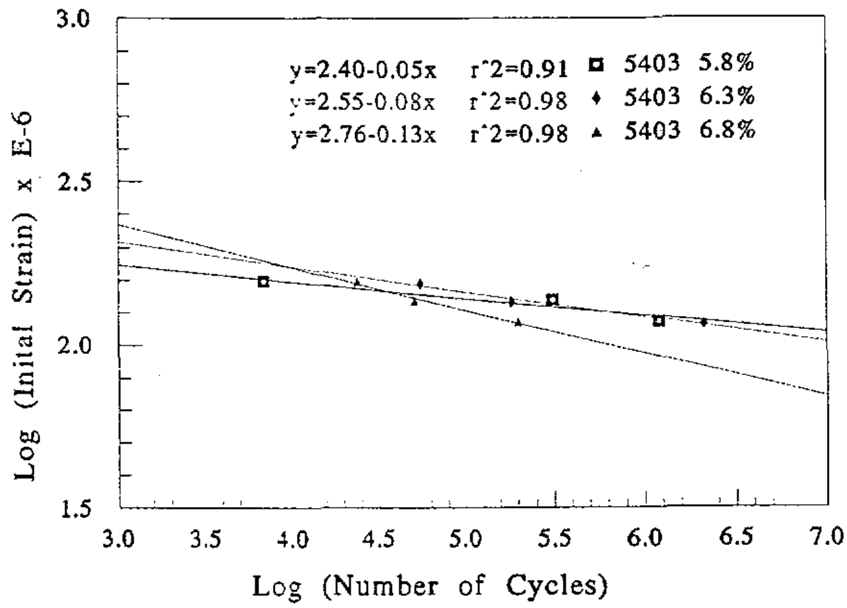


Figure 9.11. Initial Strain vs. Fatigue Life of asphaltic concrete made with 100 pen. base bitumen plus 5% 40's size shredded tire rubber for aggregate gradation type DCT result.

9.6. Summary

Both investigated aggregate gradations either obtained from dry compaction test in the laboratory or obtained from the equation (3.1. for n=0.5) were compacted with two different base bitumen (50 pen and 100 pen.) to evaluate the fatigue properties of asphaltic concrete. In addition, to predict the effect of rubber on the fatigue properties of asphaltic concrete, both gradation were mixed with the 40's size shredded rubber modified bitumen in 2 and 5% by weight. The test conducted on the optimum binder content, which was obtained from the Leeds Design Method, and ± 0.5%.

The fatigue lines obtained from this investigation are unique for the test method adopted and the fatigue failure criterion. It could not be favorably compared with the test results obtained other work undertaken elsewhere.

The addition of the rubber is highly significant on the fatigue life of asphaltic concrete. A small quantity of rubber addition (5%) increased the fatigue life of the mixture twofold for the modified binder made with the 50 pen base bitumen and 23 times for the binder made with the 100 pen. base bitumen. A summary of the fatigue results obtained in this research are also given in Figure 9.12. at optimum binder content.

The fatigue results showed that the type of bitumen greatly affected the fatigue life of the asphaltic concrete. The stiffer binder leads the stiffer mixture. This is in line with the observations of Epps *et al.*(1969). Pell (1985) has noted similar findings and concluded that an improvement in the fatigue life may simply be obtained by increasing the stiffness. On the other hand, at the fatigue test temperature (5 °C) both base bitumens had approximately the same stiffness (200 MPa). This can be attributed to the film thickness around the aggregate. The mixing temperature was kept constant in this investigation (150 °C). Hence, the viscosity of the 50 pen. and 100 pen. base bitumen was different. The softer bitumen, with low viscosity, covered the aggregate with less film thickness while the harder bitumen probably provided a thicker film around the aggregates. This can lead early cracking on the fatigue life of the mixture made with softer binnder comparing with the hard one.

It was seen also that the aggregate gradation has an effect on the fatigue life of the mixture. The aggregate gradation, which was obtained by equation 3.1.and n=0.5, was showed longer fatigue life than the gradation obtained in the laboratory on the basis of minimum porosity.

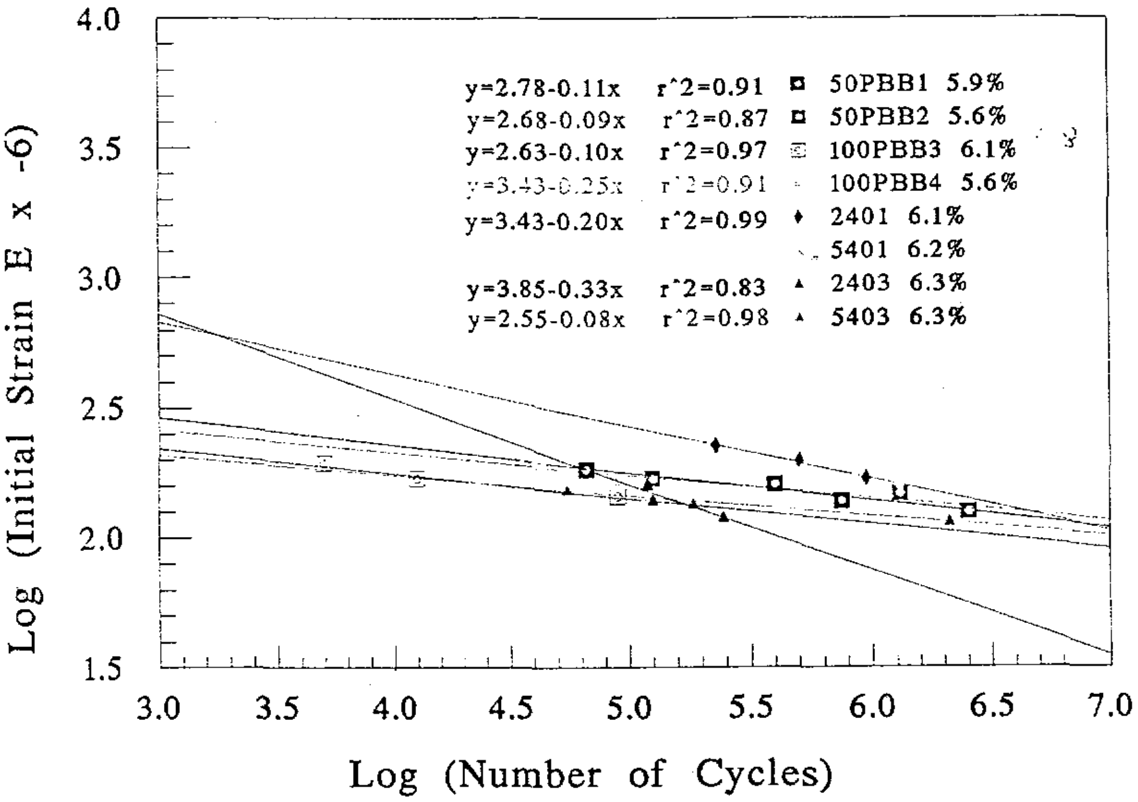


Figure 9.12. Initial Strain vs. Fatigue Life of asphaltic concrete investigated in this research (at optimum binder content).

CHAPTER TEN

CONCLUSIONS AND RECOMMENDATIONS

10.1. General Conclusions

The most important general conclusion of this study can be outlined as follows:

1- Waste shredded rubber can be used in bituminous mixtures as an effective modifier of the binder and as an improver of the properties of bituminous mixtures.

2- The rheological properties of waste shredded tire rubber modified binders were different from the properties of the base bitumen. Generally, the modified binders gave higher values of viscosity and stiffness when compared with the base bitumen.

3- The porosity of the asphaltic concrete was significantly influenced by the modification of bitumen with shredded tire rubber. High volume of rubber or/and large particle size of shredded tire rubber increased the porosity and gave higher values of permeability, and in general resulted in unsatisfactory mixes. Limiting the amount of rubber to ≤ 5% and using the smallest size gave improved properties to the asphaltic concrete mixes studied.

4- A new method of measuring porosity was developed. This method has the advantage of measuring only the interconnected porosity.

5- Small quantity of fine shredded tire rubber incorporated into bitumen resulted on asphaltic concrete with improved fatigue resistance.

6- The specific gravity of shredded rubber modified bitumen was found to increase with increasing rubber content.

10.2. Detailed Conclusions

The specific conclusions obtained from this investigation and valid only for the materials and conditions of the study are given in the following paragraphs.

10.2.1. Properties of Rubber-Bitumen Blends

- The rheological properties of shredded rubber modified binders studied using controlled stress oscillatory rheometer and other empirical tests resulted on the following conclusions;

- The rubber modified binders have higher viscosity and stiffness than the base bitumen.

- The hardness of the shredded rubber modified bitumen as measured by the penetration test show that the penetration value increases with the addition of rubber. The change in penetration resulting from addition of shredded rubber can be estimated using a statistical equation. The mixing time of shredded rubber and base bitumen is important to produce a homogeneous mix where the rubber particles are digested in the hot bitumen. The degree of dispersion and digestion of the rubber particles is seen by the reduction of penetration value with increasing time of mixing. A mixing time of 30 minutes was found to be adequate to obtain consistent behavior of the rubber-bitumen blends.

- The softening point test shows similarity to the penetration test. A linear increase was also observed on the softening point value with increasing shredded rubber concentration. The softening point increases with increasing mixing time.

- A relationship between penetration and viscosity was developed during the study. The numerical equation quantifying this relation is different from the equation previously proposed for pure bitumen.

- The elastic recovery of shredded rubber modified bitumen as measured by modified ductility test increases with the addition of rubber. This is a desirable property for the binder used in bituminous mixture. However, this improvement gives in some instance undesirable results such as an increase on the porosity.

10.2.2. Aggregate Gradation

- The concept of maximum aggregate packing or minimum porosity used to develop a particle size distribution resulted in a gradation close to the gradation proposed by Cooper *et al.* (1985), but with enough variation as to produce a lower VMA.

10.2.3. Porosity Measurements

- Increasing rubber content increases the porosity of shredded rubber modified asphaltic concrete.

- Linear statistical relationships were found between measured vacuum porosity using by Leeds Vacuum Porosimeter and the traditional methods of calculated porosities. In the equations, the slope of the line found very close to 45° for both cases when the vacuum porosity value Pv, compared with the calculated porosity value of Pg and Paw. The values of interconnected voids were found depends on the base bitumen used in the asphaltic concrete. These values found were 2.3% for 50 pen. base bitumen and 1.6% for the 100 pen. base bitumen used in rubber-bitumen blends for rubberised asphaltic concrete.

10.2.4. The Properties of Asphaltic Concrete made with Shredded Tire Rubber Modified Bitumen

- The workability index decreased with increasing rubber content.

- The addition of shredded rubber decreased the density of compacted asphaltic concrete.

- The Compacted Aggregate Density (CAD) of the asphaltic concrete decreased with the addition of shredded rubber or Voids in Mixed Aggregate (VMA) increased with the in creasing of shredded rubber in mixture.

- The addition of small proportion (up to 5%) of fine rubber increased the Marshall stability of the mixtures made with low penetration grade base bitumen (100 pen.). The stability values decreased with further increase on rubber content and rubber sizes.

-The addition of rubber decreases the Marshall flow. A large decrease was observed at higher rubber content and bigger rubber sizes.

- The air permeability of asphaltic concrete is significantly increased by the addition rubber.

- The 1 hour creep stiffness of the mixture increased by about 85%, when small amount of fine rubber (40's rubber 2% by weight) was incorporated into the 50 pen grade base bitumen. The addition of high rubber content reduced the creep stiffness of the mixture particularly the mixture made with 100 pen. base bitumen.

- The addition of rubber increases the optimum binder content.

10.2.5. Fatigue Behavior of Asphaltic Concrete made with Shredded Rubber Modified Bitumen

- Controlled stress fatigue testing was found to be an adequate method to study the fatigue behavior of asphaltic concrete made with shredded rubber modified bitumen.

- The addition of rubber chances significantly the fatigue life of the asphaltic concrete. A small quantity of rubber addition (40's size 5%) doubles the fatigue life of the mixture made with 50 pen. base bitumen. Mixes made with 100 pen. base bitumen increased their fatigue life by a factor of 20.

10.3. Recommendation for Further Study

1- This investigation concentrated on the study of the effect of shredded tire rubber particles on the rheological properties of rubber-bitumen blends and their influence on the engineering properties and fatigue behavior of asphaltic concrete. An extension to this investigation is desirable to study the effects of incorporating the shredded rubber particles directly into the aggregate binder mixture.

2- A series of porosity measurements was made using by Leeds Vacuum Saturation method. More data is required to correlate the relationship between measured porosities and calculated porosities.

3- The rheological measurements showed that the elasticity of the shredded rubber bitumen greatly increases with the addition of rubber. These increases should improve the properties of the mixture. However, increased elasticity caused an increase on the porosity of the mixtures. During compaction of the sample using GTM, when the compaction pressure was lifted, the rubber particles caused rebound due to the elasticity of the binder.

It was not expected on this stage of the investigation. New compaction techniques should be investigated to avoid this problem.

4- In this investigation mixing and compaction temperature were kept constant. Future research should investigate the effect of compaction and mixing temperature.

5- The studies conducted in the laboratory should be extended to field evaluations preferably by constructing trial sections of a pavement where the surfacing should be the new asphaltic concrete developed in this investigation.

CHAPTER TEN

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- The rheological properties of shredded rubber modified binders studied using controlled stress oscillatory rheometer and other empirical tests resulted on the following conclusions;

- The rubber modified binders have higher viscosity and stiffness than the base bitumen.

- The hardness of the shredded rubber modified bitumen as measured by the penetration test show that the penetration value increases with the addition of rubber. The change in penetration resulting from addition of shredded rubber can be estimated using a statistical equation. The mixing time of shredded rubber and base bitumen is important to produce a homogeneous mix where the rubber particles are digested in the hot bitumen. The degree of dispersion and digestion of the rubber particles is seen by the reduction of penetration value with increasing time of mixing. A mixing time of 30 minutes was found to be adequate to obtain consistent behavior of the rubber-bitumen blends.

- The softening point test shows similarity to the penetration test. A linear increase was also observed on the softening point value with increasing shredded rubber concentration. The softening point increases with increasing mixing time.

- A relationship between penetration and viscosity was developed during the study. The numerical equation quantifying this relation is different from the equation previously proposed for pure bitumen.

- The elastic recovery of shredded rubber modified bitumen as measured by modified ductility test increases with the addition of rubber. This is a desirable property for the binder used in bituminous mixture. However, this improvement gives in some instance undesirable results such as an increase on the porosity.

10.2.2. Aggregate Gradation

- The concept of maximum aggregate packing or minimum porosity used to develop a particle size distribution resulted in a gradation close to the gradation proposed by Cooper *et al.* (1985), but with enough variation as to produce a lower VMA.

10.2.3. Porosity Measurements

- Increasing rubber content increases the porosity of shredded rubber modified asphaltic concrete.

- Linear statistical relationships were found between measured vacuum porosity using by Leeds Vacuum Porosimeter and the traditional methods of calculated porosities. In the equations, the slope of the line found very close to 45° for both cases when the vacuum porosity value Pv, compared with the calculated porosity value of Pg and Paw. The values of interconnected voids were found depends on the base bitumen used in the asphaltic concrete. These values found were 2.3% for 50 pen. base bitumen and 1.6% for the 100 pen. base bitumen used in rubber-bitumen blends for rubberised asphaltic concrete.

10.2.4. The Properties of Asphaltic Concrete made with Shredded Tire Rubber Modified Bitumen

- The workability index decreased with increasing rubber content.

- The addition of shredded rubber decreased the density of compacted asphaltic concrete.

- The Compacted Aggregate Density (CAD) of the asphaltic concrete decreased with the addition of shredded rubber or Voids in Mixed Aggregate (VMA) increased with the in creasing of shredded rubber in mixture.

- The addition of small proportion (up to 5%) of fine rubber increased the Marshall stability of the mixtures made with low penetration grade base bitumen (100 pen.). The stability values decreased with further increase on rubber content and rubber sizes.

-The addition of rubber decreases the Marshall flow. A large decrease was observed at higher rubber content and bigger rubber sizes.

- The air permeability of asphaltic concrete is significantly increased by the addition rubber.

- The 1 hour creep stiffness of the mixture increased by about 85%, when small amount of fine rubber (40's rubber 2% by weight) was incorporated into the 50 pen grade base bitumen. The addition of high rubber content reduced the creep stiffness of the mixture particularly the mixture made with 100 pen. base bitumen.

- The addition of rubber increases the optimum binder content.

10.2.5. Fatigue Behavior of Asphaltic Concrete made with Shredded Rubber Modified Bitumen

- Controlled stress fatigue testing was found to be an adequate method to study the fatigue behavior of asphaltic concrete made with shredded rubber modified bitumen.

- The addition of rubber chances significantly the fatigue life of the asphaltic concrete. A small quantity of rubber addition (40's size 5%) doubles the fatigue life of the mixture made with 50 pen. base bitumen. Mixes made with 100 pen. base bitumen increased their fatigue life by a factor of 20.

10.3. Recommendation for Further Study

1- This investigation concentrated on the study of the effect of shredded tire rubber particles on the rheological properties of rubber-bitumen blends and their influence on the engineering properties and fatigue behavior of asphaltic concrete. An extension to this investigation is desirable to study the effects of incorporating the shredded rubber particles directly into the aggregate binder mixture.

2- A series of porosity measurements was made using by Leeds Vacuum Saturation method. More data is required to correlate the relationship between measured porosities and calculated porosities.

3- The rheological measurements showed that the elasticity of the shredded rubber bitumen greatly increases with the addition of rubber. These increases should improve the properties of the mixture. However, increased elasticity caused an increase on the porosity of the mixtures. During compaction of the sample using GTM, when the compaction pressure was lifted, the rubber particles caused rebound due to the elasticity of the binder.

It was not expected on this stage of the investigation. New compaction techniques should be investigated to avoid this problem.

4- In this investigation mixing and compaction temperature were kept constant. Future research should investigate the effect of compaction and mixing temperature.

5- The studies conducted in the laboratory should be extended to field evaluations preferably by constructing trial sections of a pavement where the surfacing should be the new asphaltic concrete developed in this investigation.

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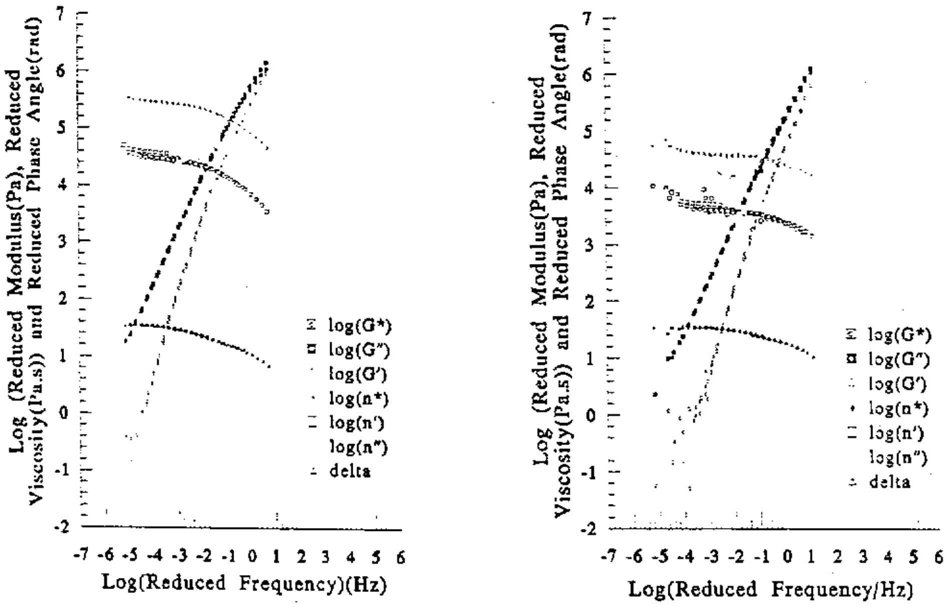
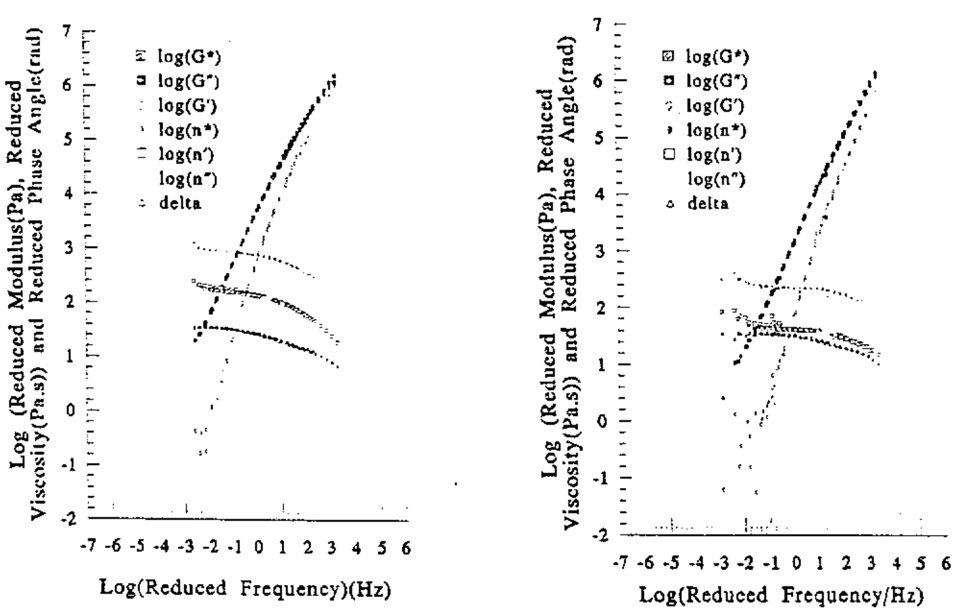
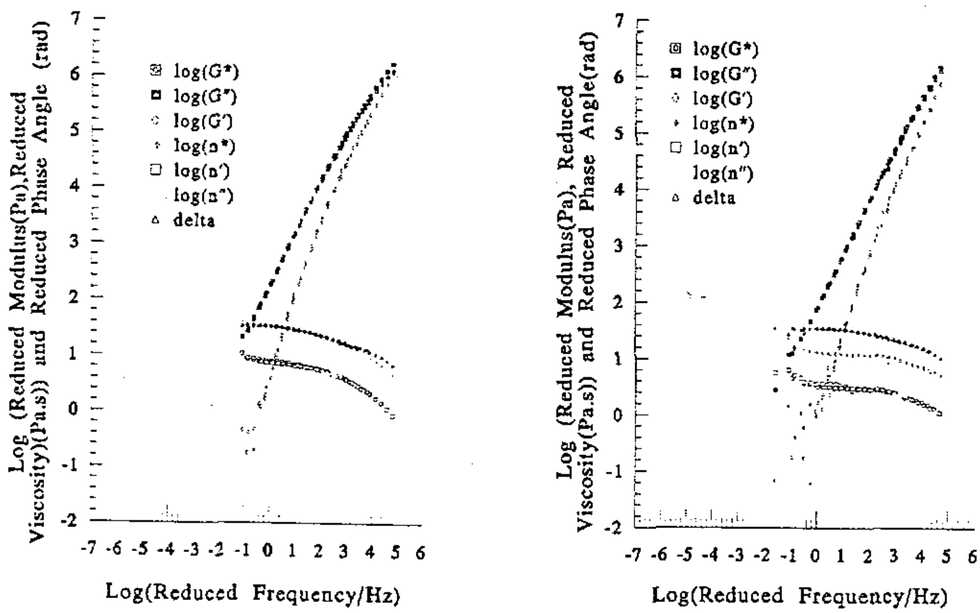
APPENDIX A



A.1. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 2% 40's Size Rubber at 30 °C (blended at 160 °C for 1/2 hour).



A.2. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 2% 40's Size Rubber at 60 °C (blended at 160 °C for 1/2 hour).



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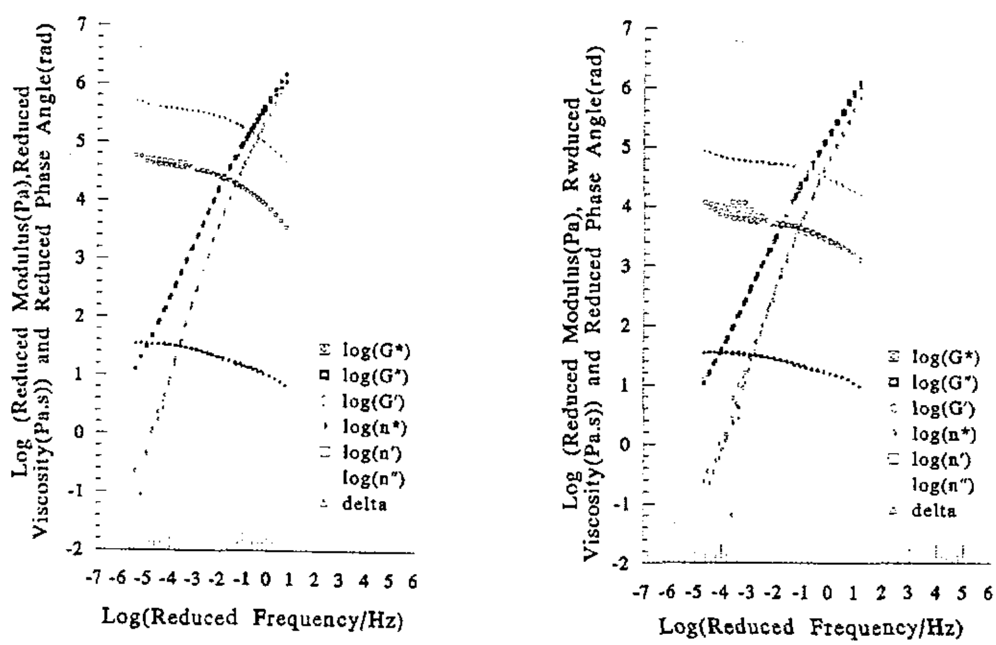
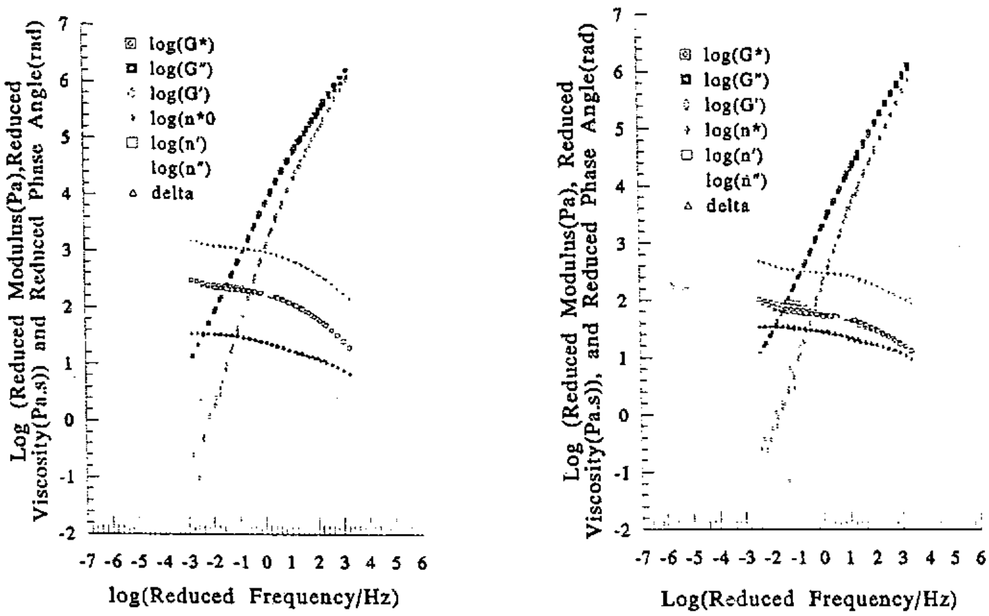
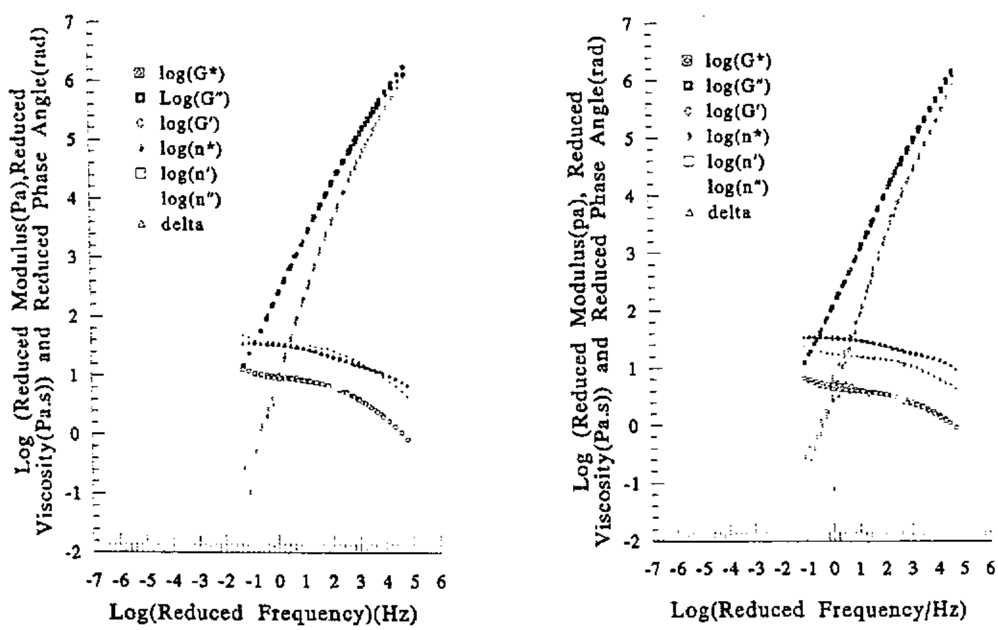


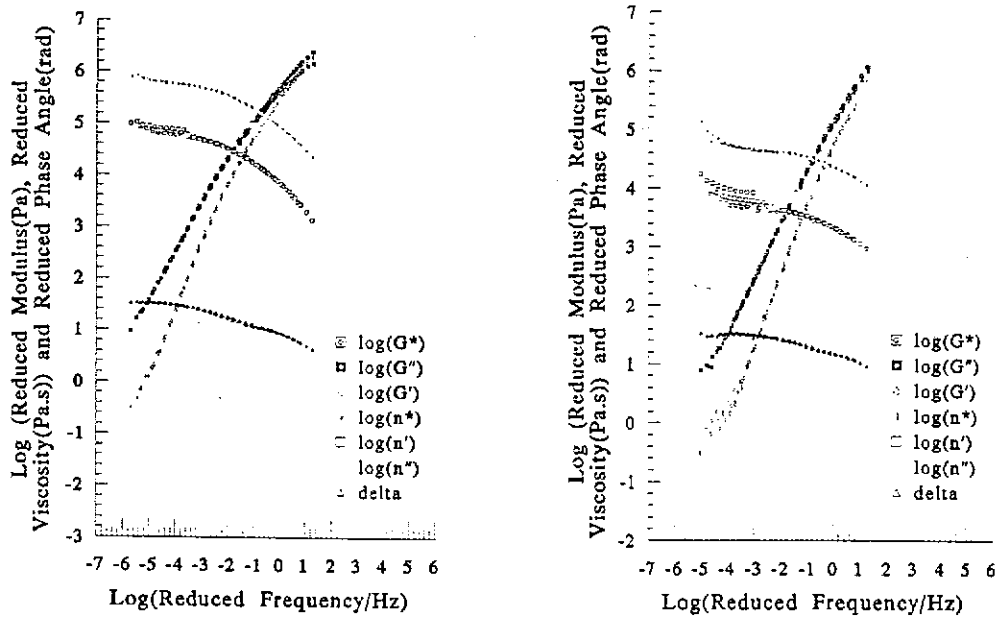
A.4.Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 5% 40's Size Rubber at 30 °C (blended at 160 °C for 1/2 hour).



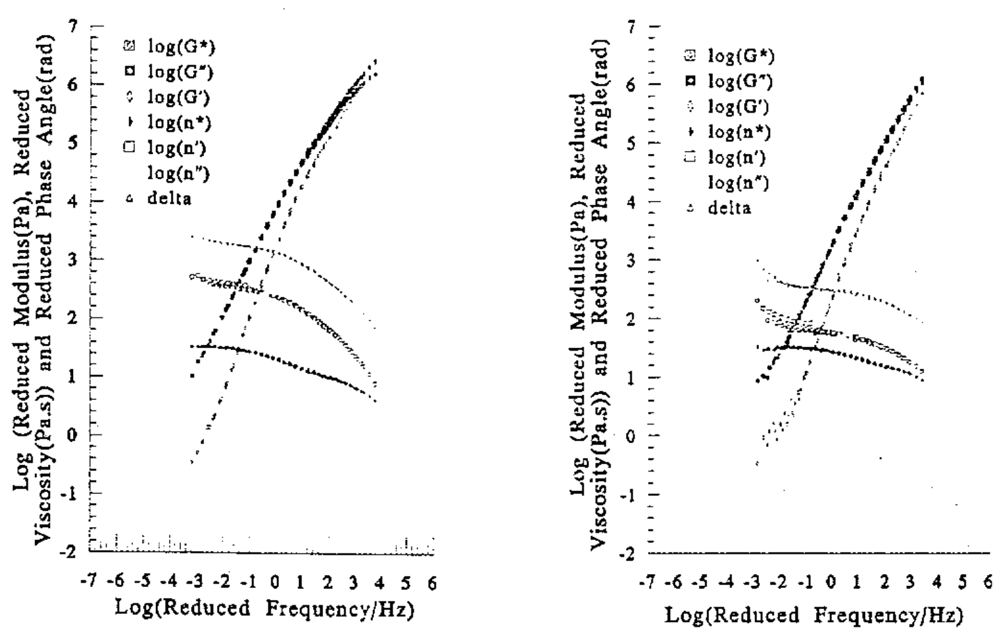
A.5. Rheological Parameters Rubber Bitumen Mix Made With of 50 (left) and 100 (right) pen. Base Bitumen plus 5% 40's Size Rubber at 60 °C (blended at 160 °C for 1/2 hour).



A.6. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 5% 40's Size Rubber at 90 °C (blended at 160 °C for 1/2 hour).



A.7. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 10% 40's Size Rubber at 30 °C (blended at 160 °C for 1/2 hour).



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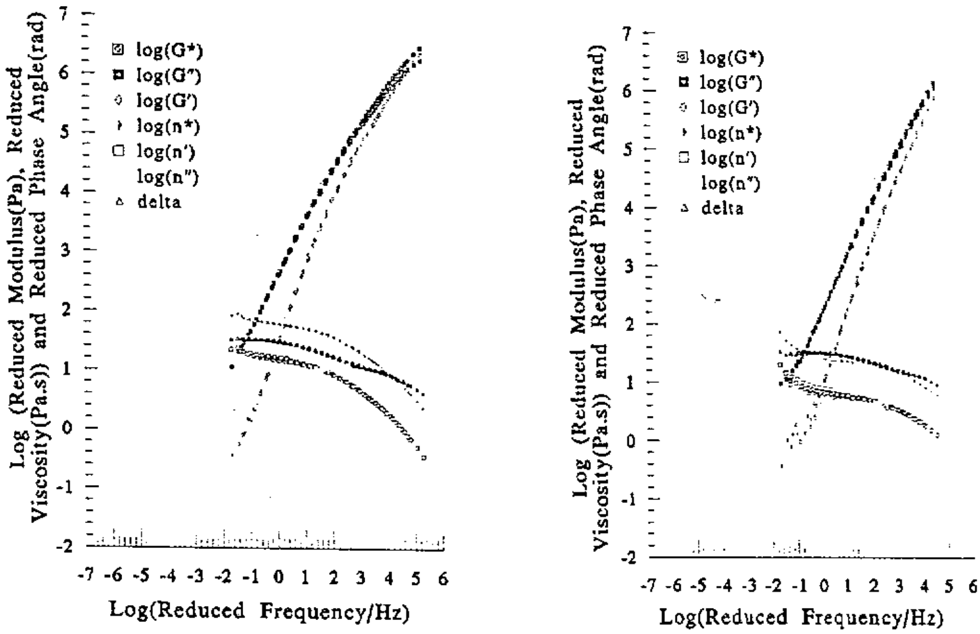

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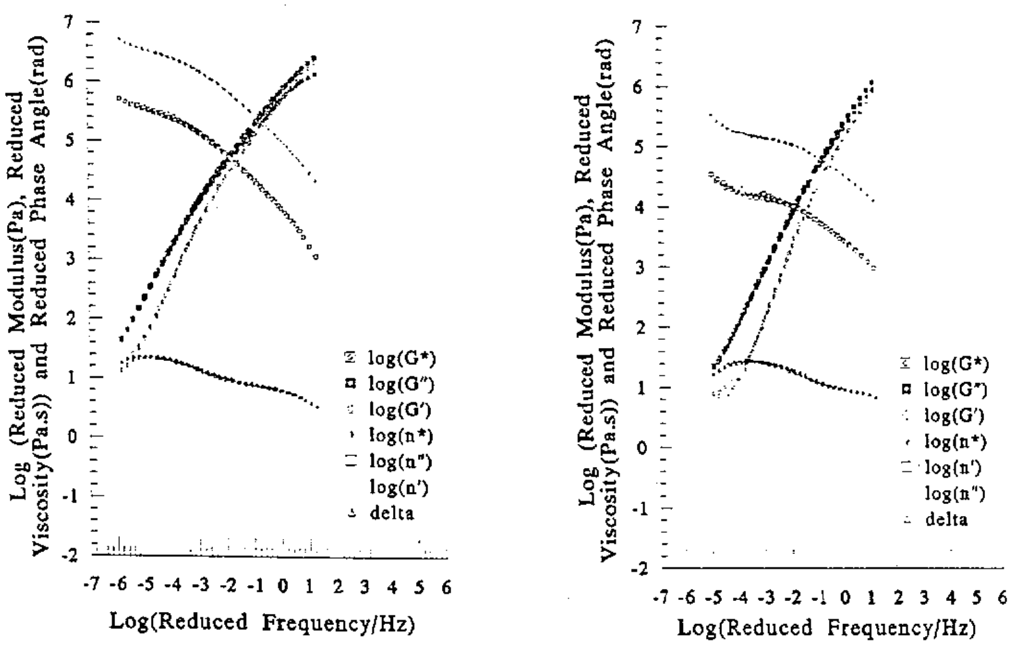
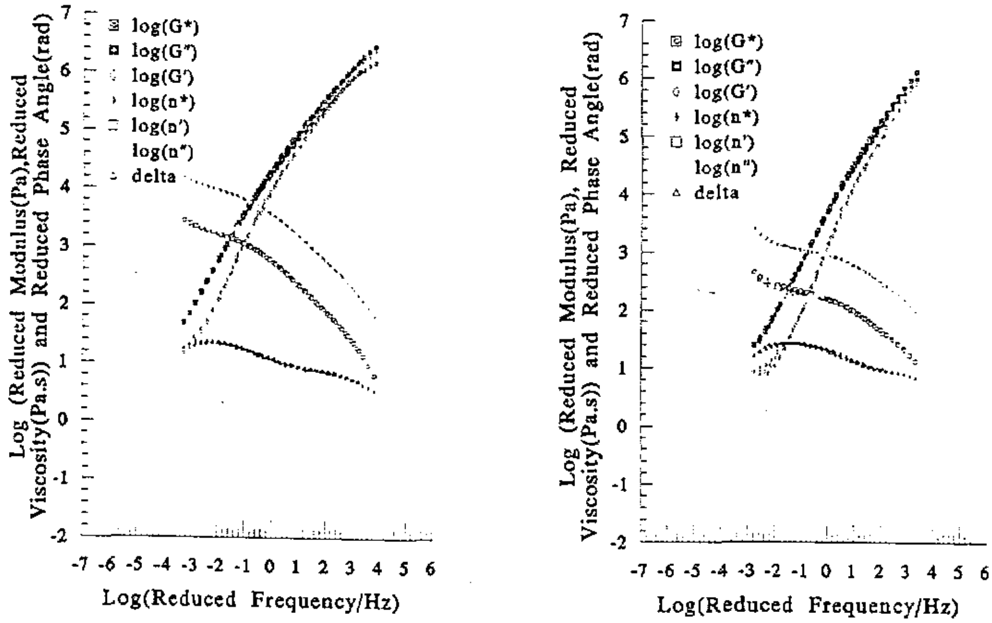


A.10. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Bitumen plus 20% 40's Size Rubber at 30 °C (blended at 160 °C for 1/2 hour).



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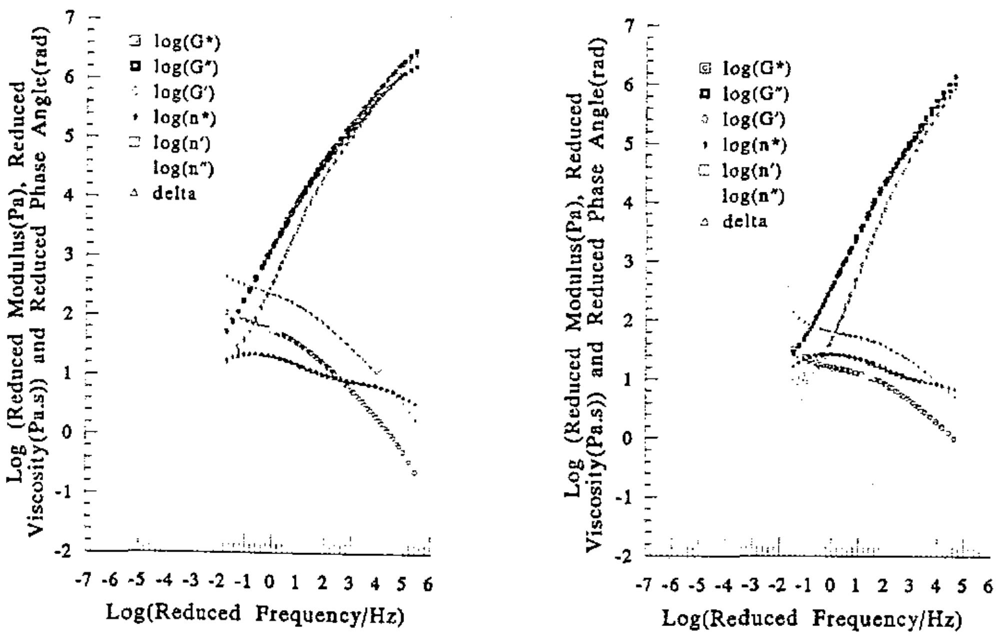

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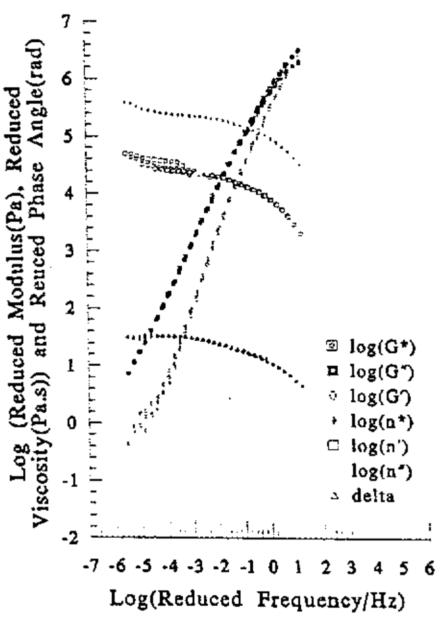
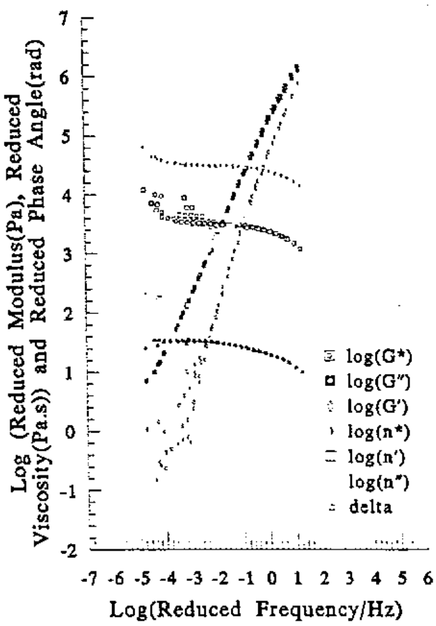
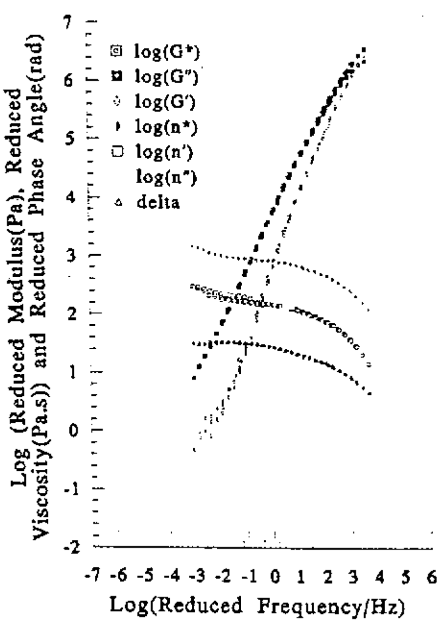
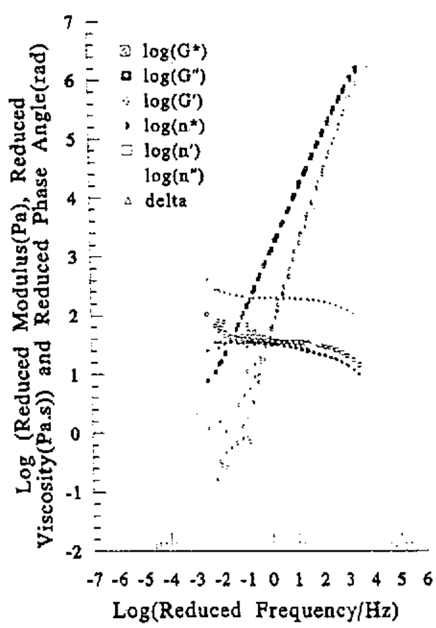


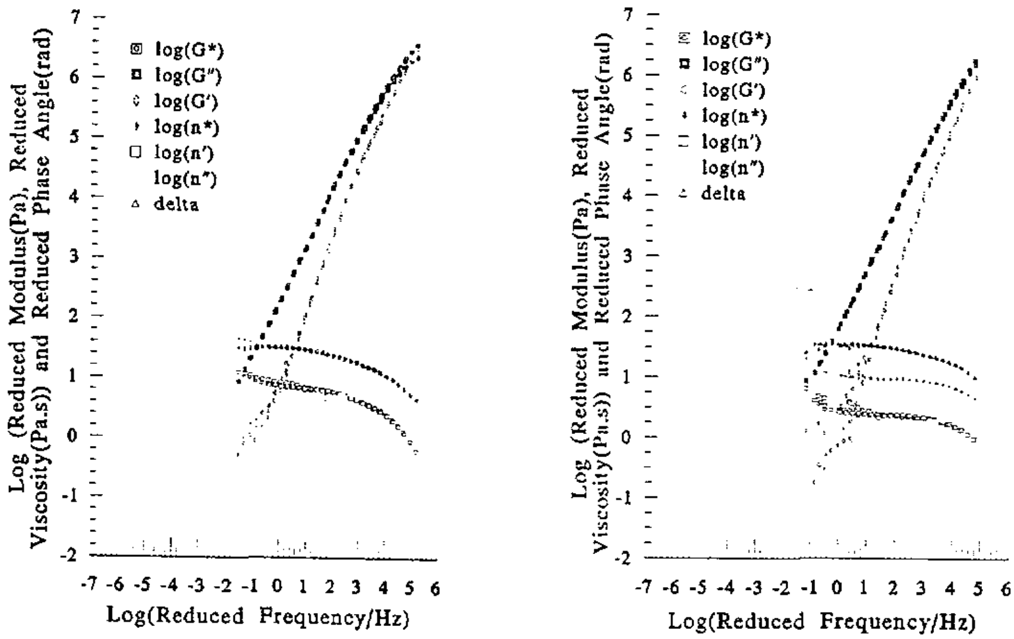


A.13. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Bitumen plus 2% 12/16's Rubber at 30 °C (blended at 160 °C for 1/2 hour).

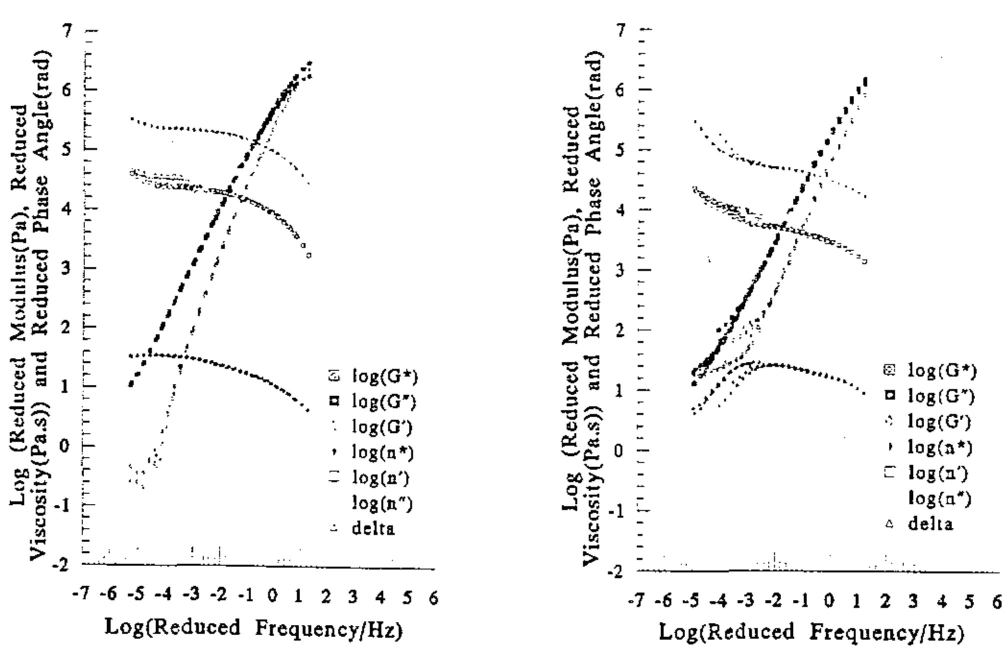




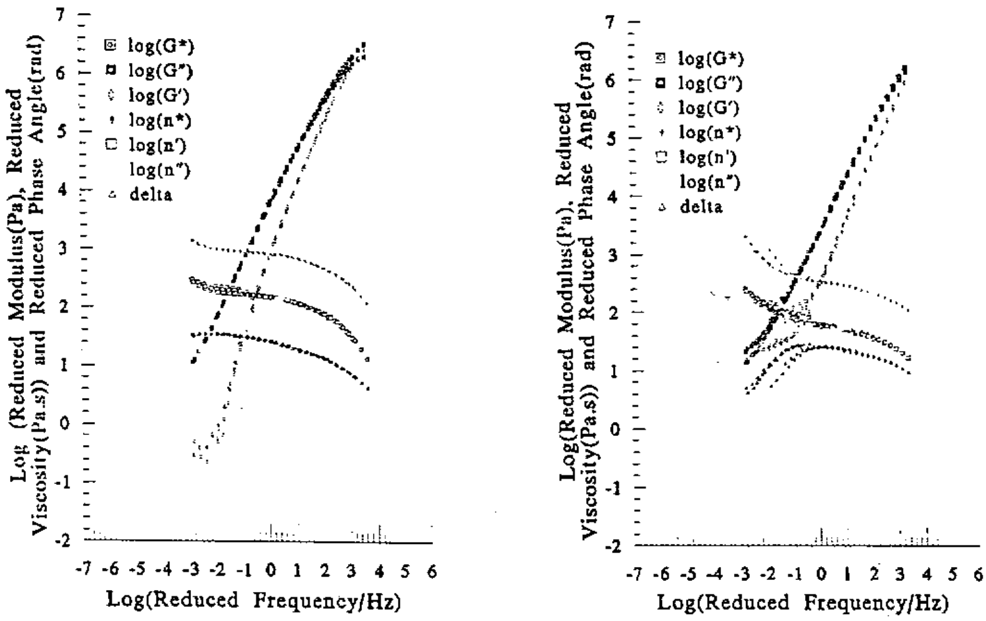
A.14. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Bitumen plus 2% 12/16's Rubber at 60 °C (blended at 160 °C for 1/2 hour).



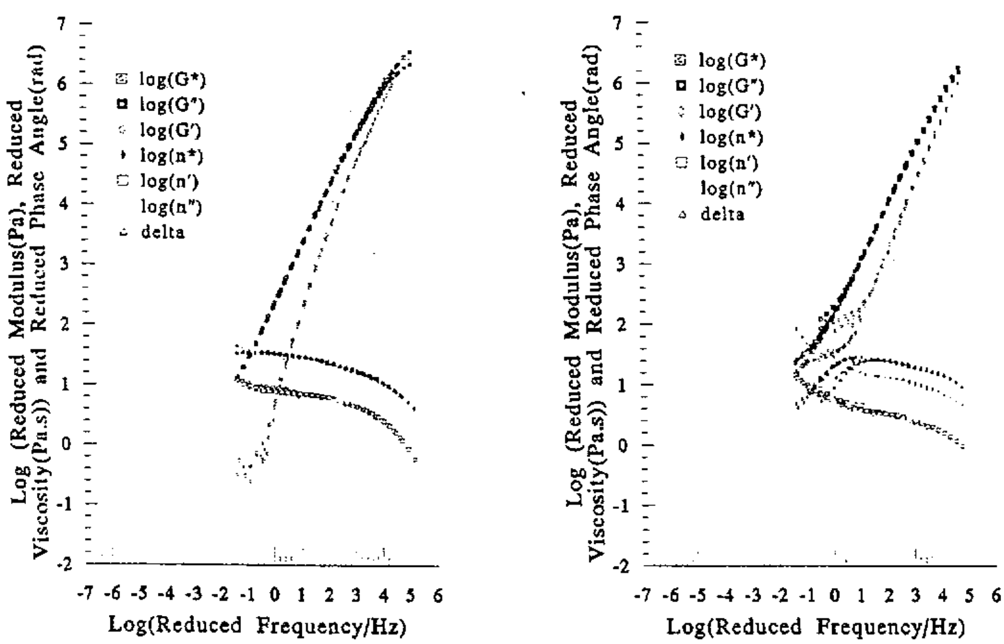
A.15. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Bitumen plus 2% 12/16's Rubber at 90 °C (blended at 160 °C for 1/2 hour).



A.16. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Bitumen plus 5% 12/16's Rubber at 30 °C (blended at 160 °C for 1/2 hour).



A.17. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Bitumen plus 5% 12/16's Rubber at 60 °C (blended at 160 °C for 1/2 hour).



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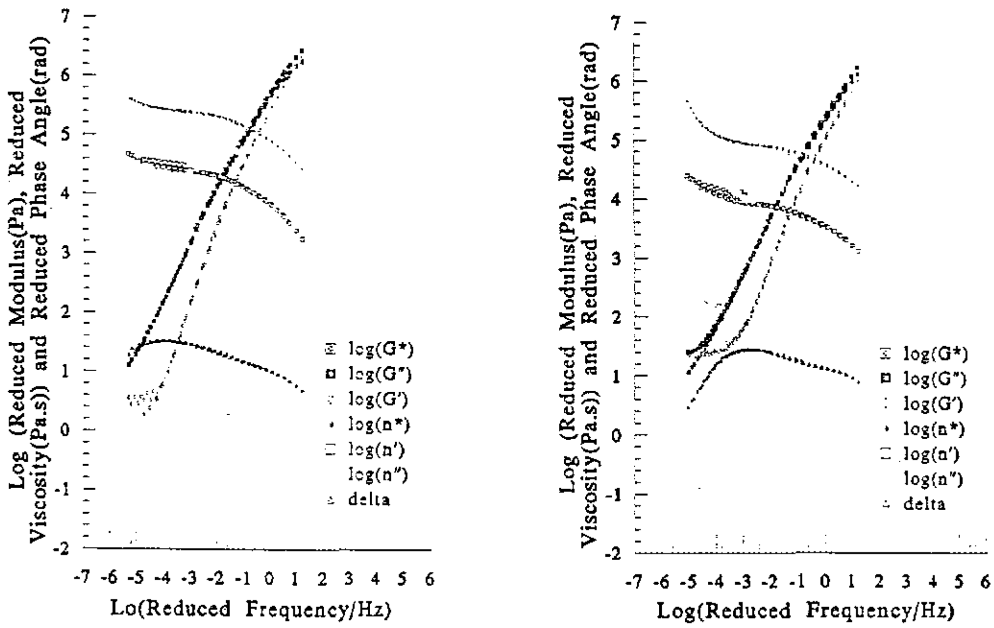

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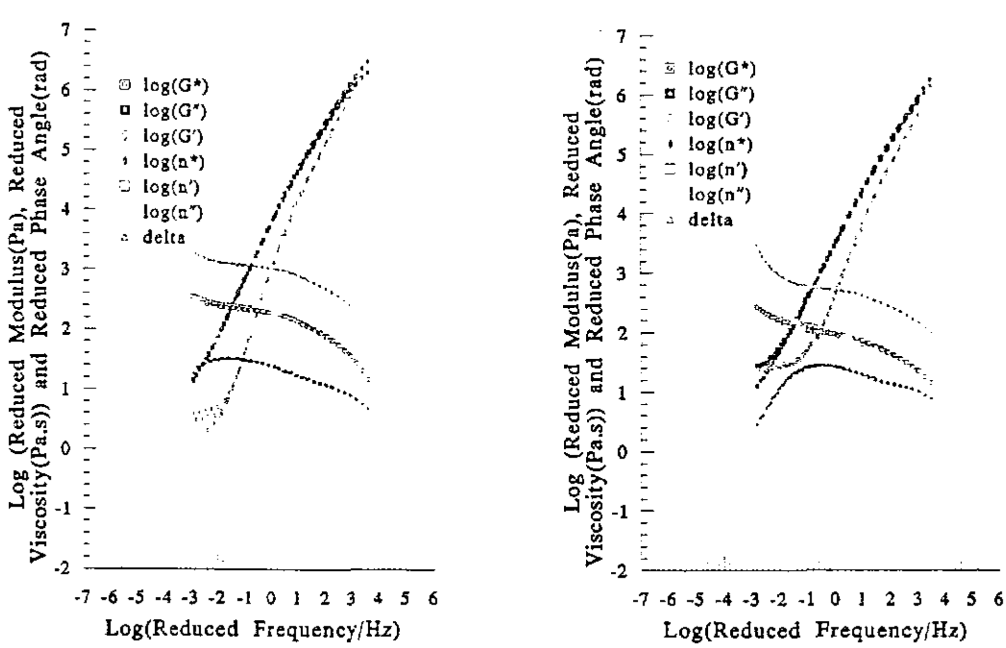
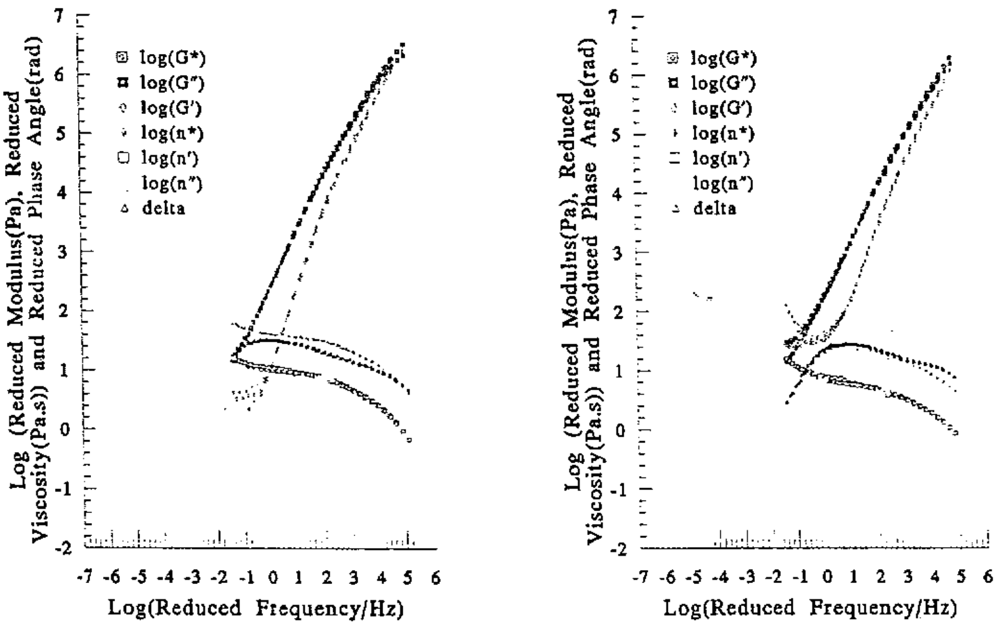
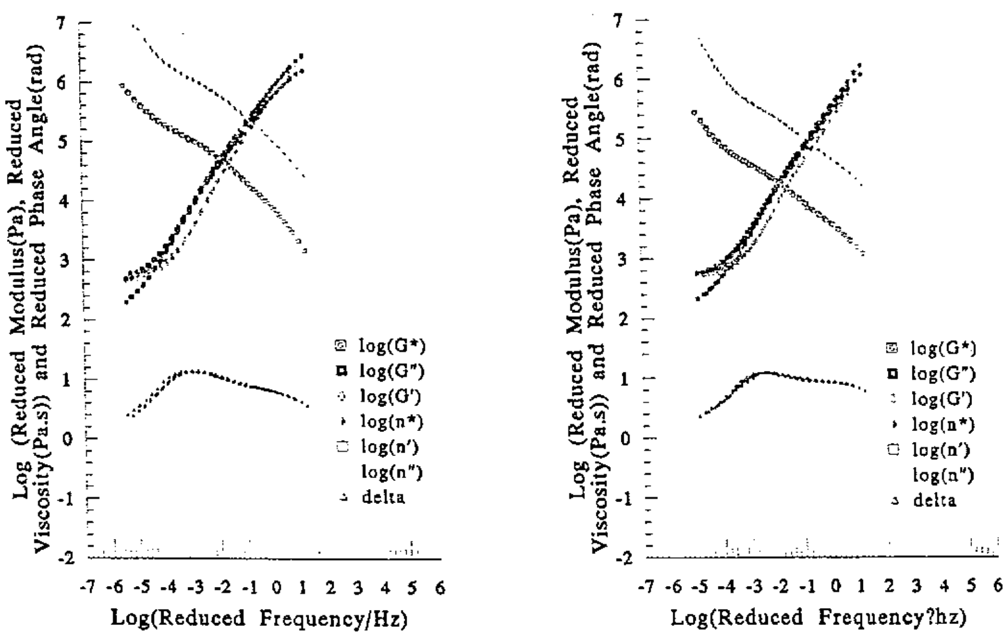


A.20. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Bitumen plus 10% 12/16's Rubber at 60 °C (blended at 160 °C for 1/2 hour).



A.21. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Bitumen plus 10% 12/16's Rubber at 90 °C (blended at 160 °C for 1/2 hour).



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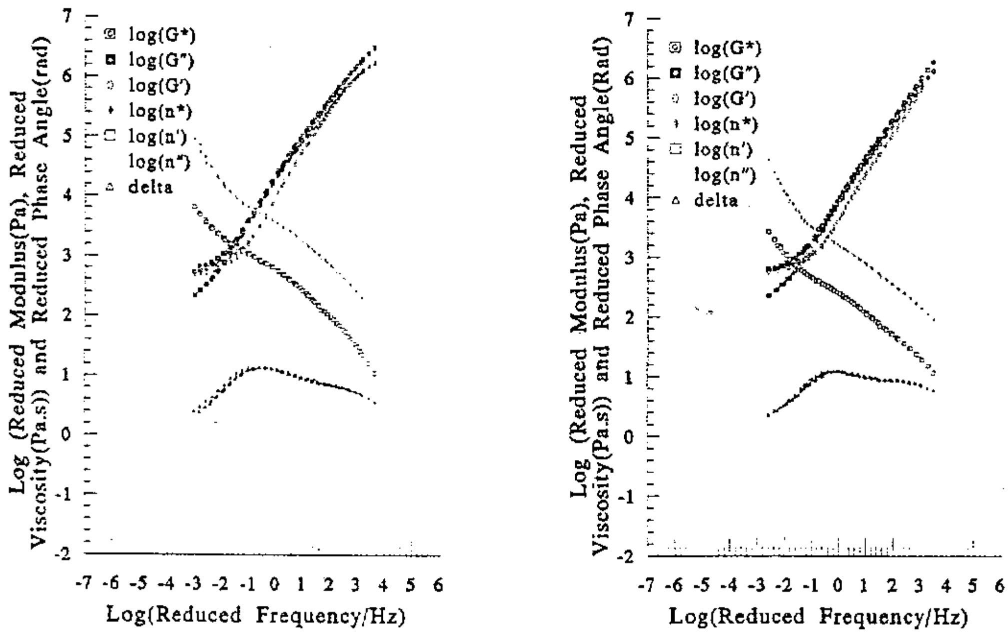
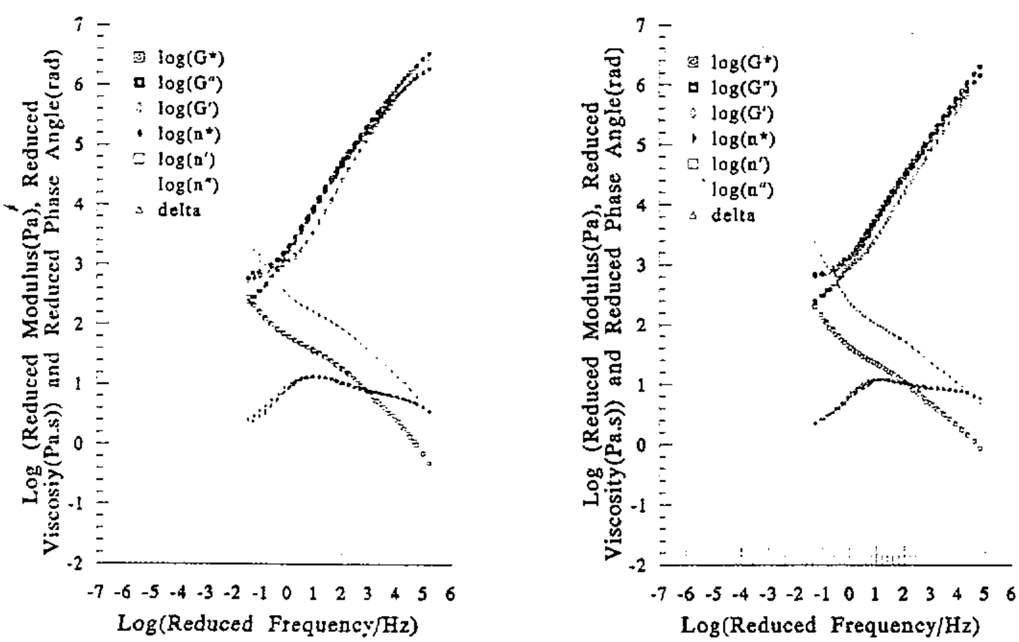
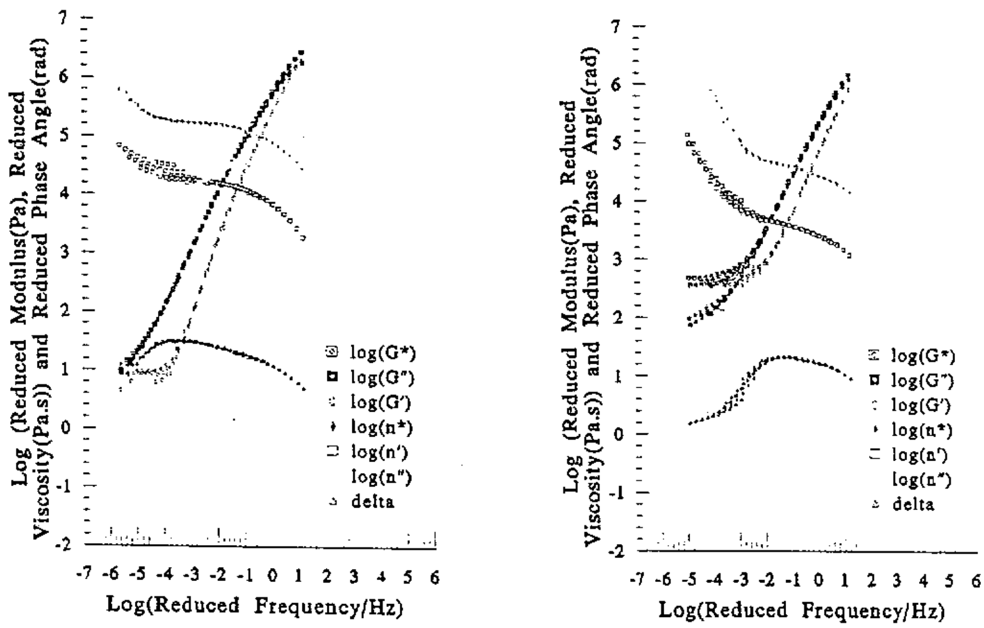


A.23. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Bitumen plus 20% 12/16's Rubber at 60 °C (blended at 160 °C for 1/2 hour).



A.24. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Bitumen plus 20% 12/16's Rubber at 90 °C (blended at 160 °C for 1/2 hour).



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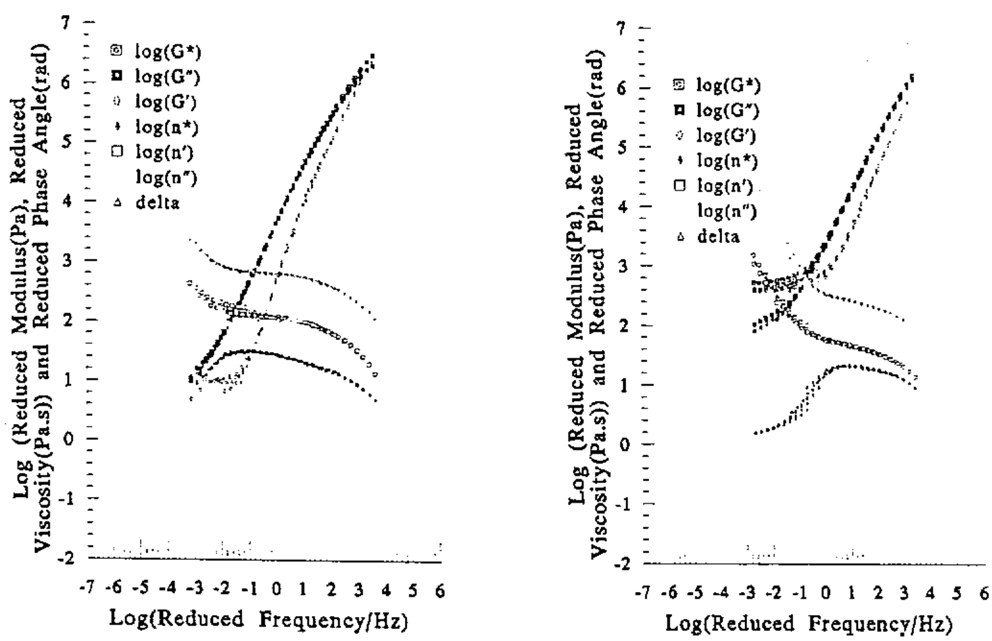

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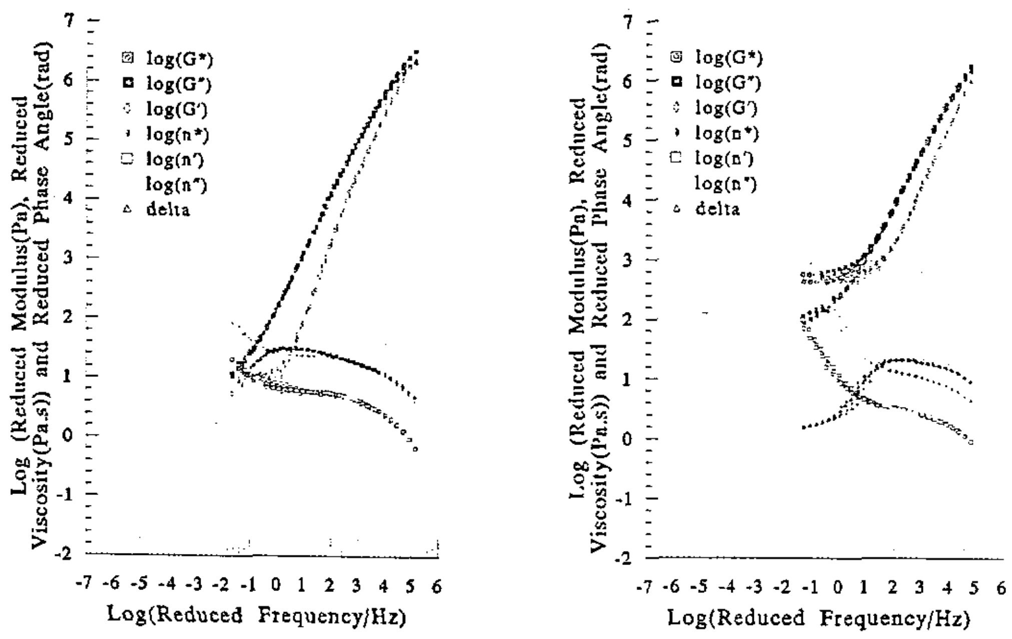

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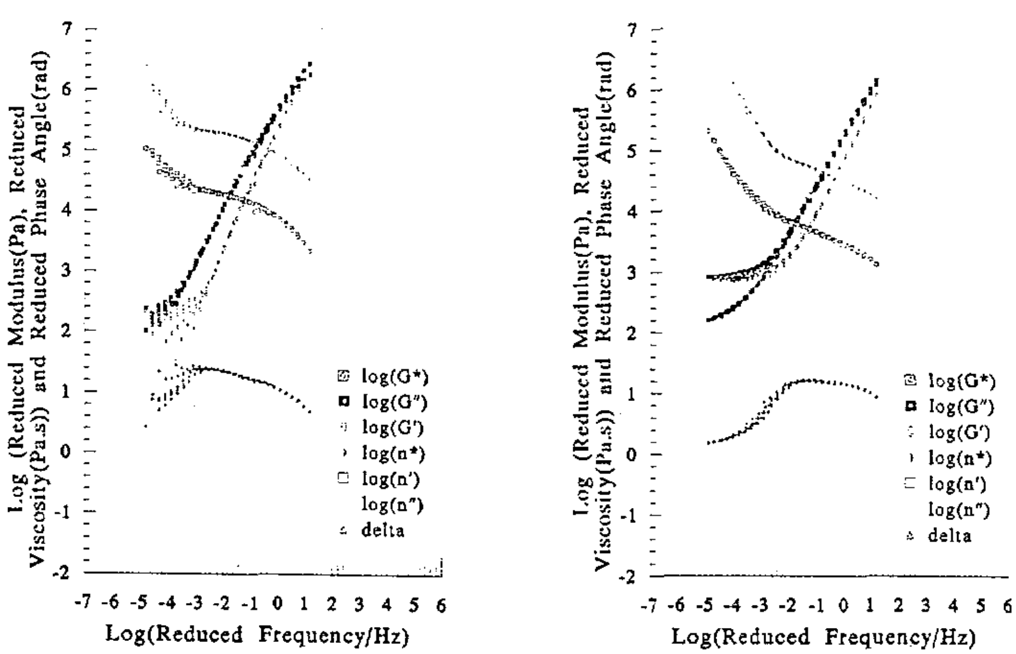
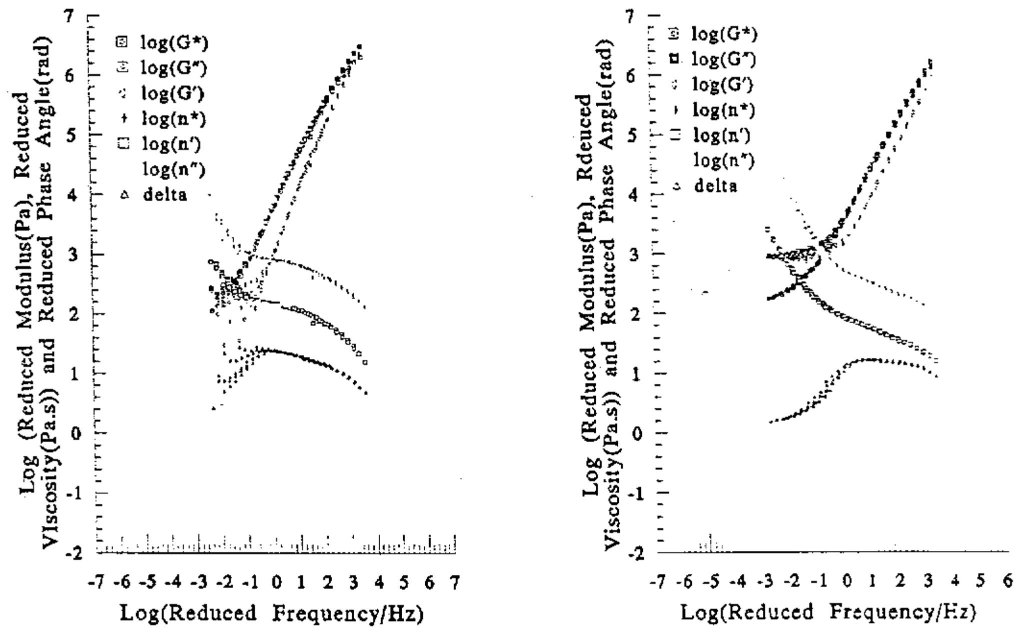


A.28. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 5% 12's Rubber at 30 °C (blended at 160 °C for 1/2 hour).



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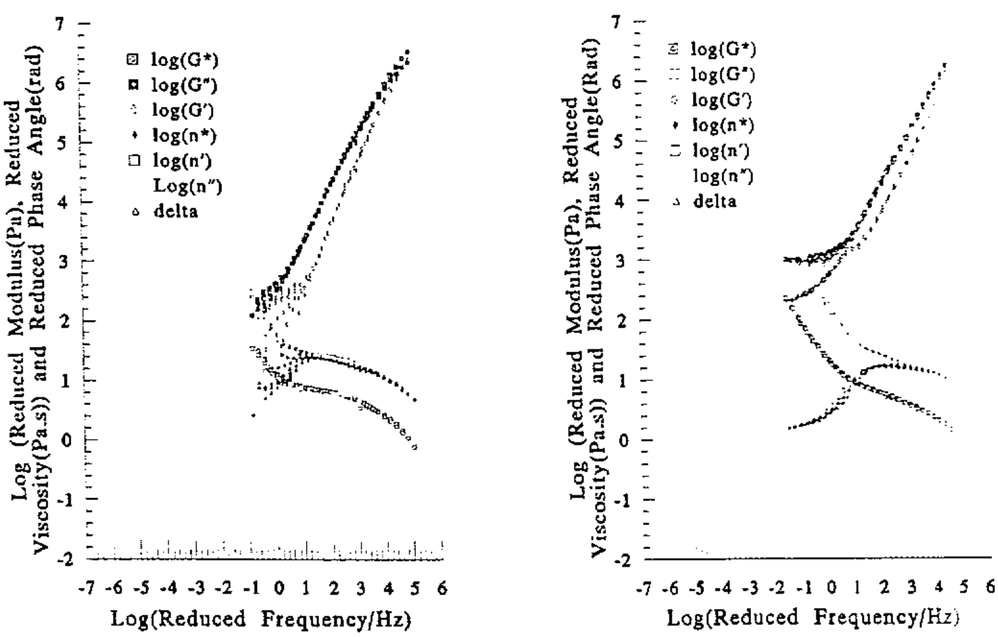

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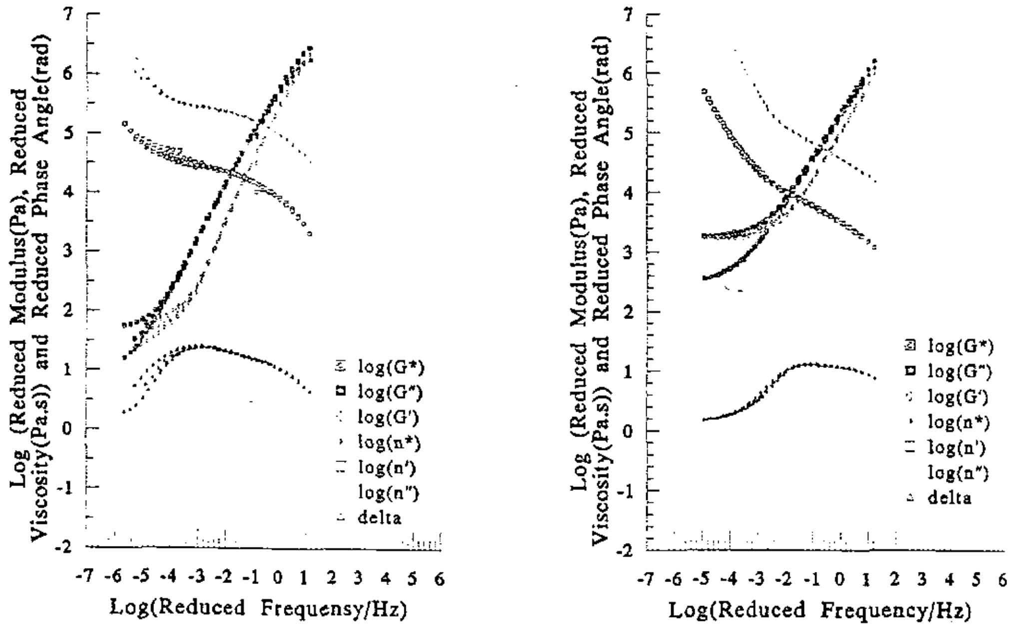

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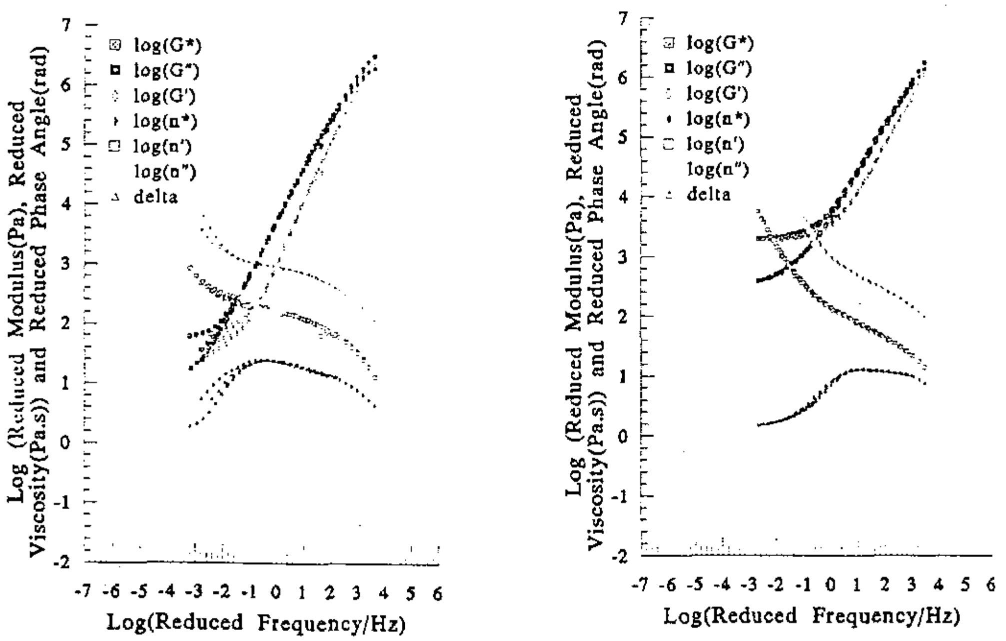

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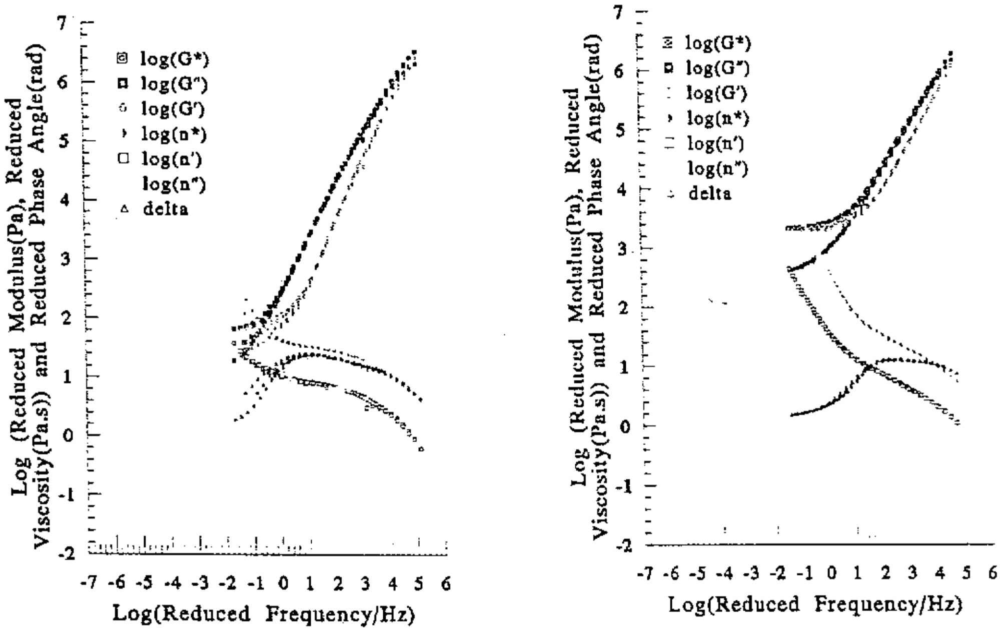

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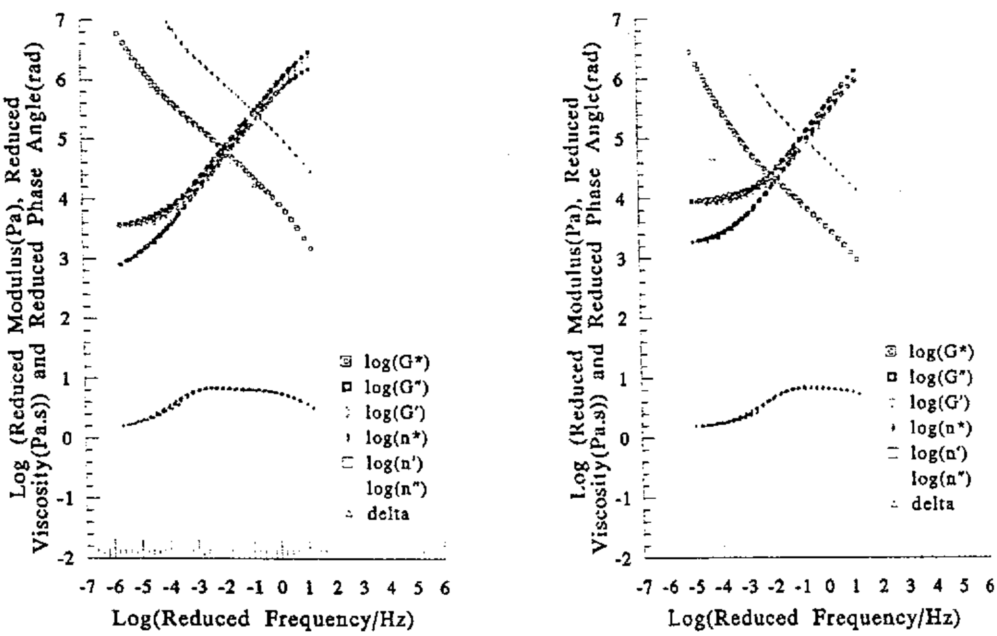

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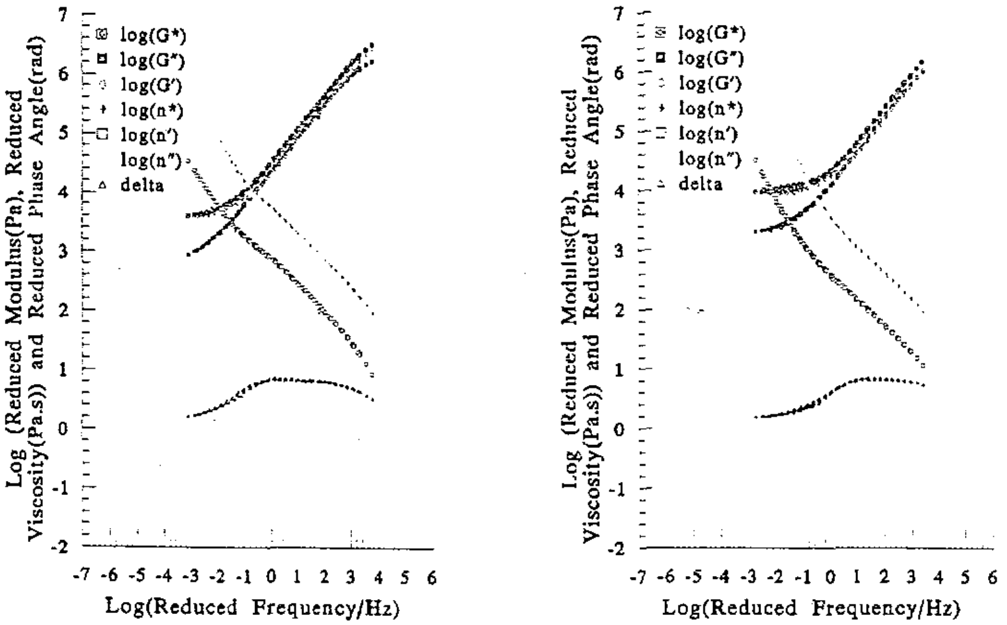

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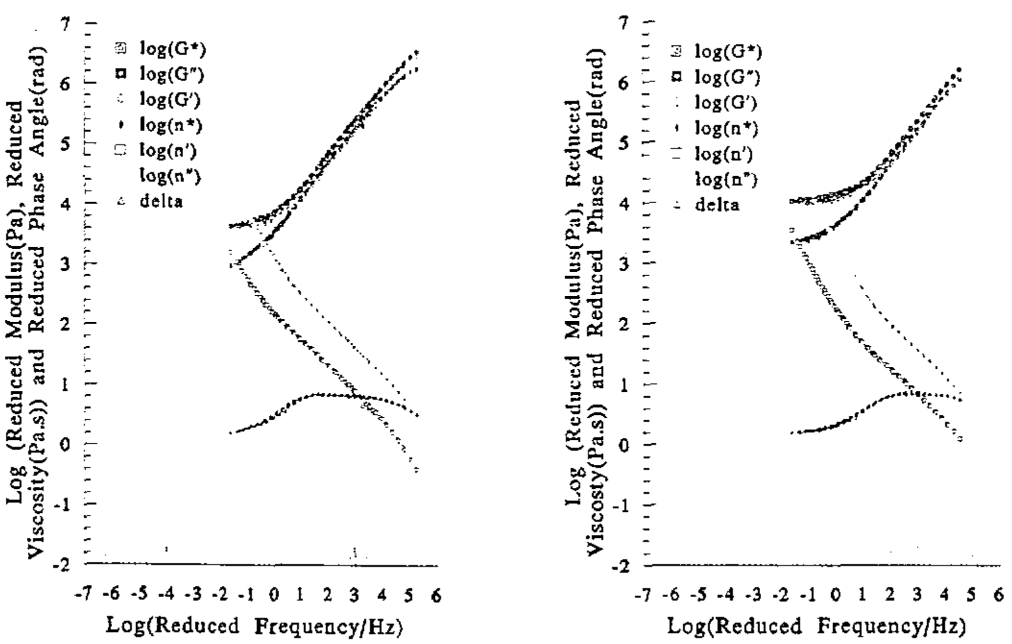
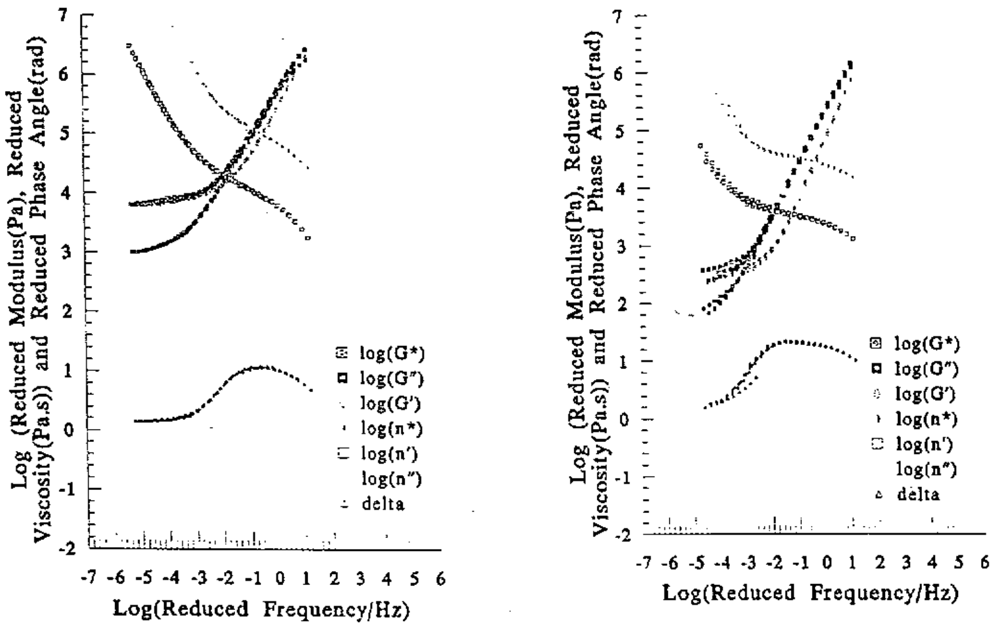
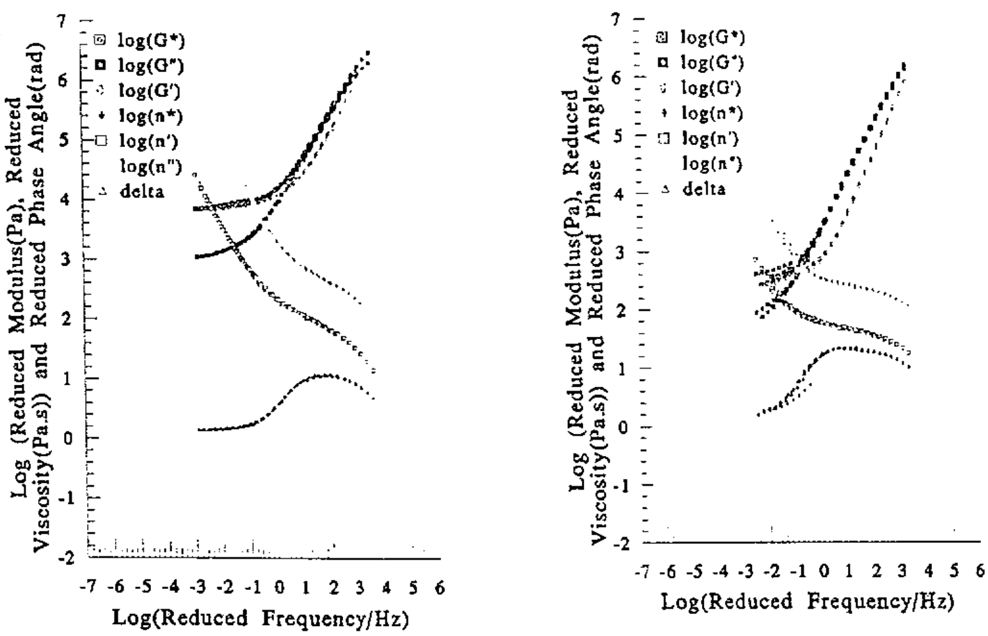


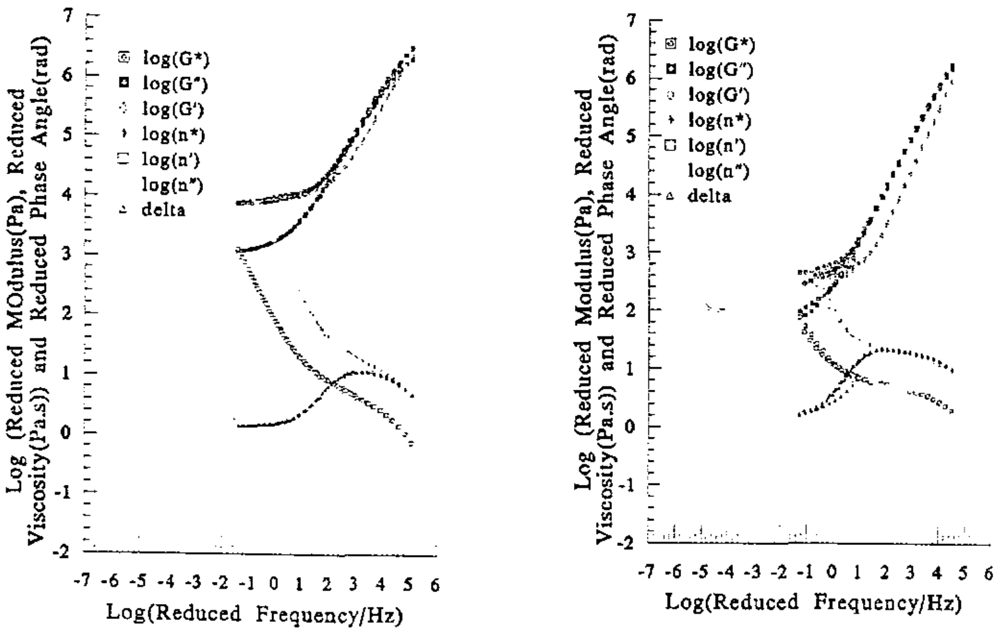
A.36. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Bitumen plus 20% 12's Rubber at 90 °C (blended at 160 °C for 1/2 hour).



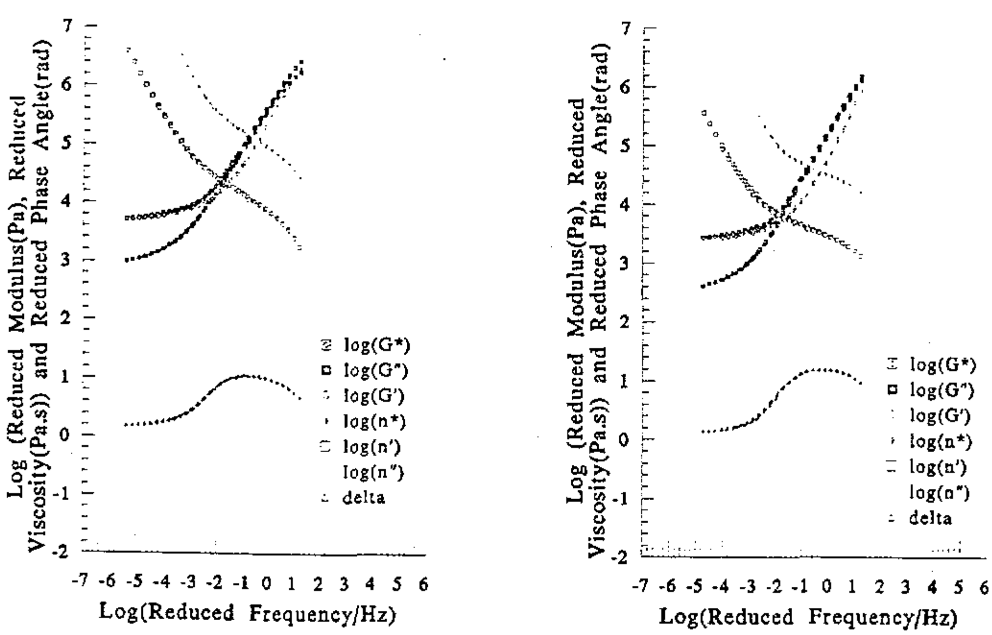
A.37. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 2% 3's Rubber at 30 °C (blended at 160 °C for 1/2 hour).



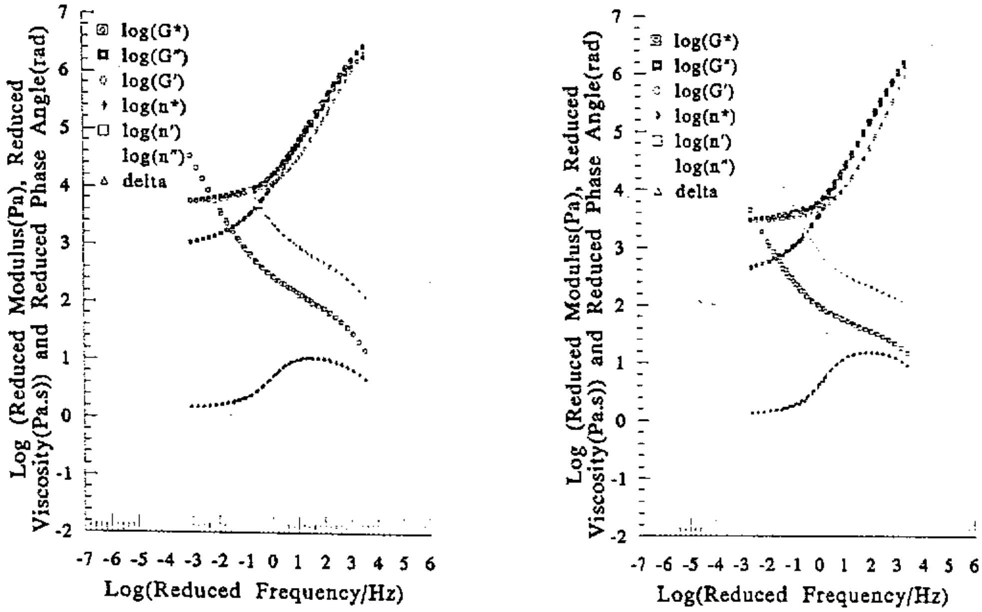
A.38. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 2% 3's Rubber at 60 °C (blended at 160 °C for 1/2 hour).



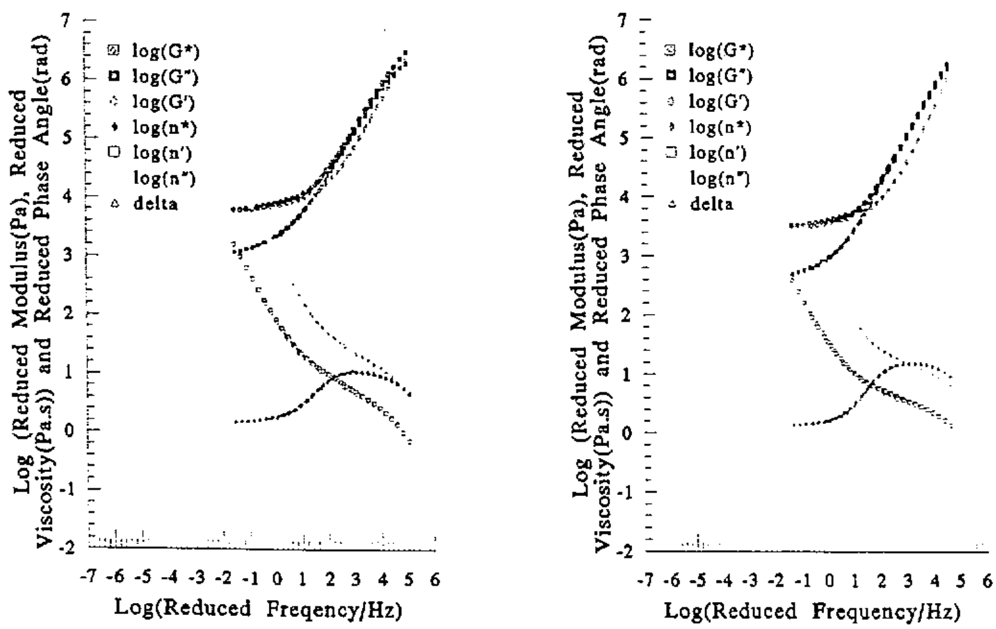
A.39. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 2% 3's Rubber at 90 °C (blended at 160 °C for 1/2 hour).



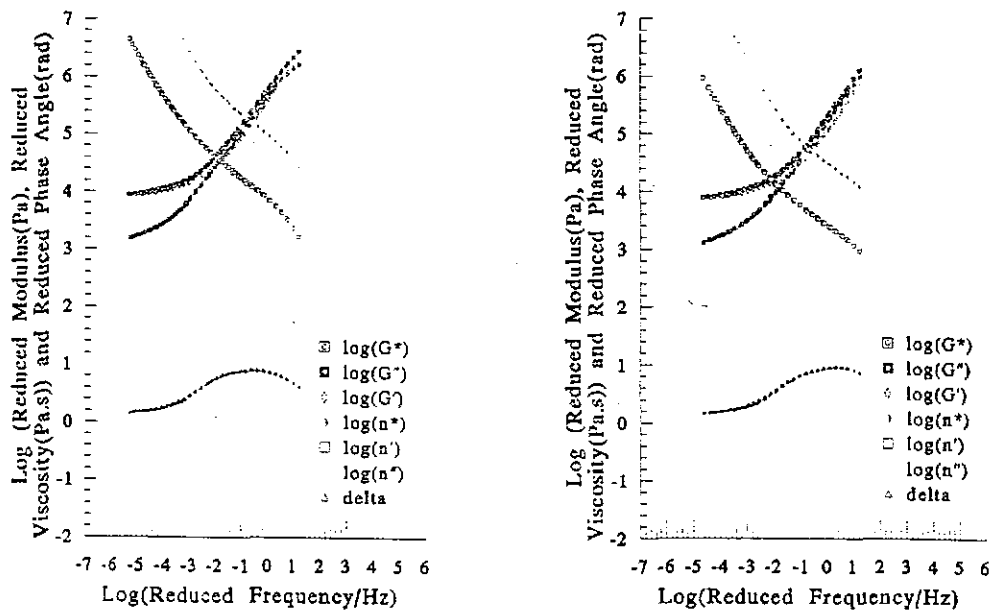
A.40. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 5% 3's Rubber at 30 °C (blended at 160 °C for 1/2 hour).



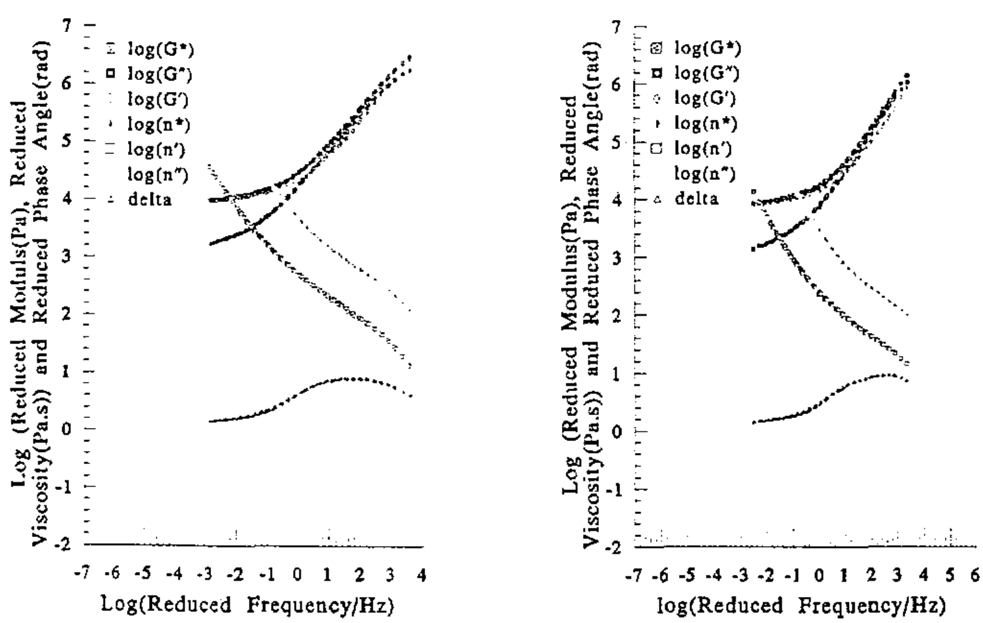
A.41. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 5% 3's Rubber at 60 °C (blended at 160 °C for 1/2 hour).



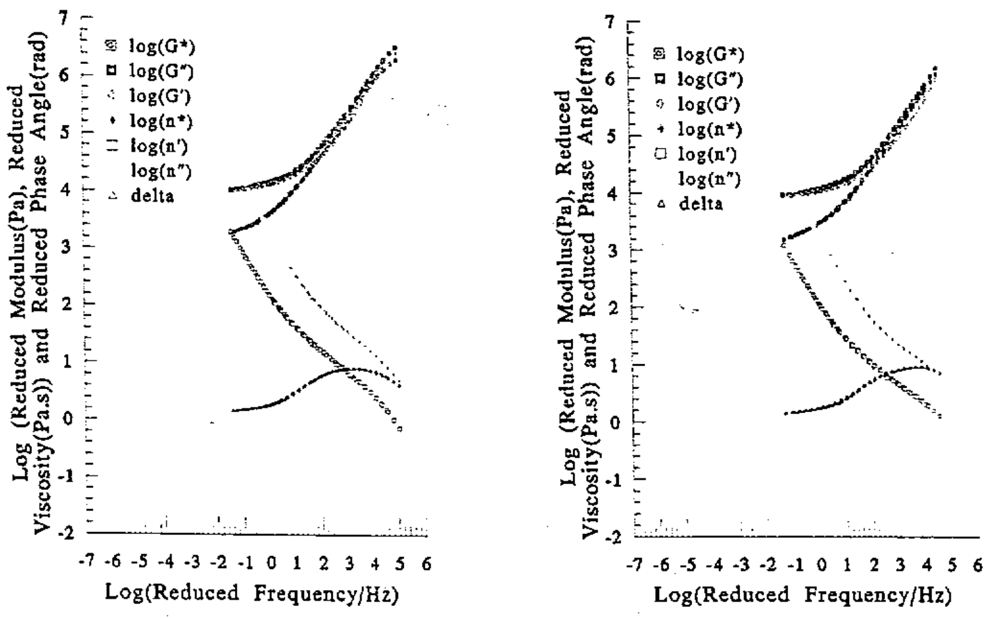
A.42. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 5% 3's Rubber at 90 °C (blended at 160 °C for 1/2 hour).



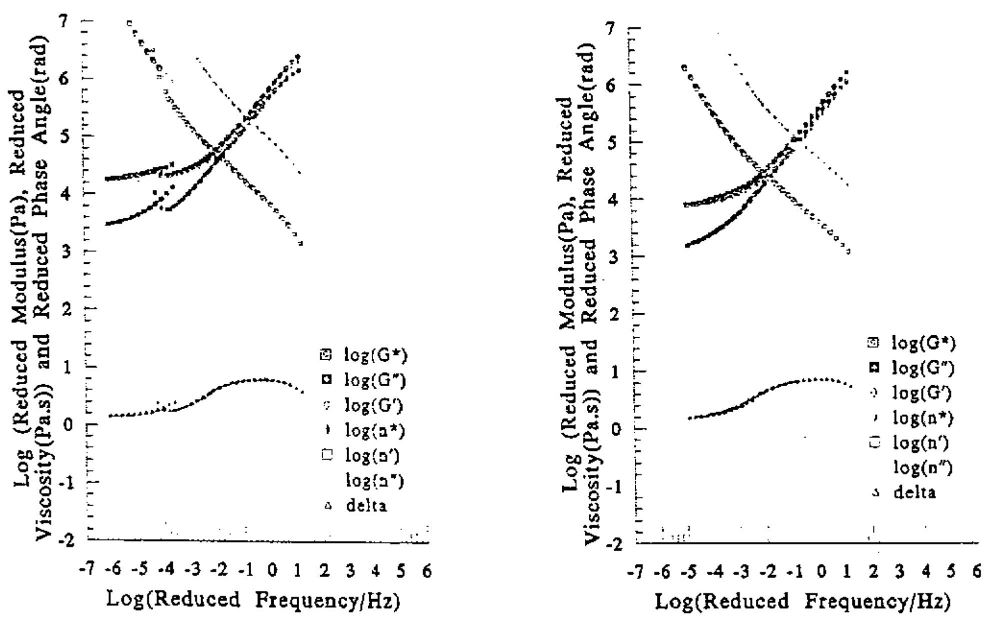
A.43. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 10% 3's Rubber at 30 °C (blended at 160 °C for 1/2 hour).



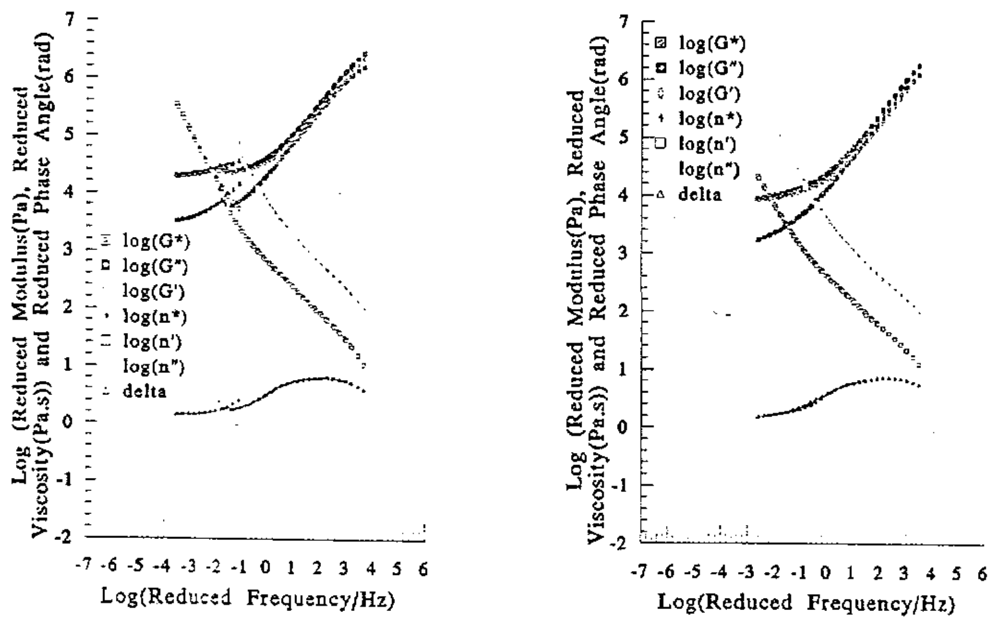
A.44. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 10% 3's Rubber at 60 °C (blended at 160 °C for 1/2 hour).



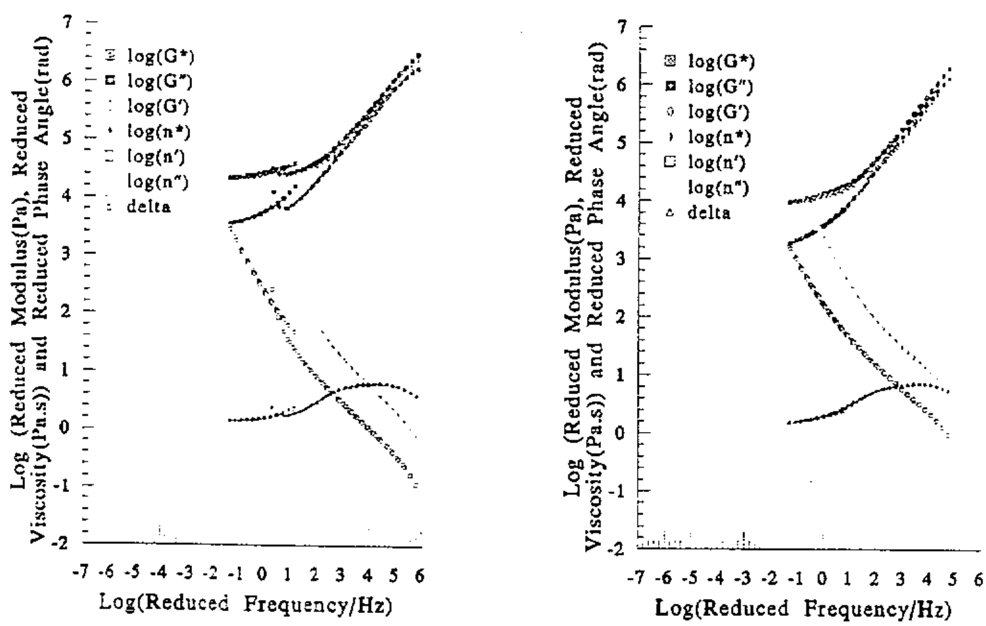
A.45. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 10% 3's Rubber at 90 °C (blended at 160 °C for 1/2 hour).



A.46. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 20% 3's Rubber at 30 °C (blended at 160 °C for 1/2 hour).

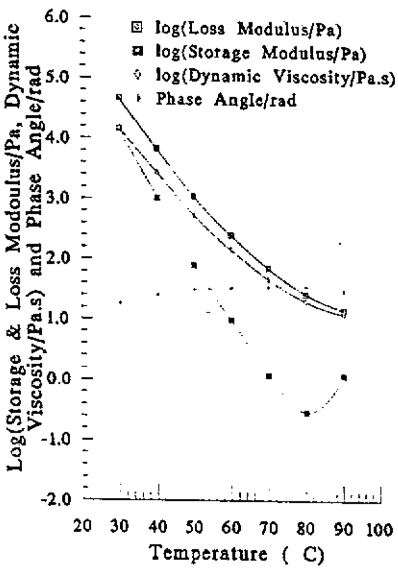


A.47. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 20% 3's Rubber at 60 °C (blended at 160 °C for 1/2 hour).



A.48. Rheological Parameters of Rubber Bitumen Mix Made With 50 (left) and 100 (right) pen. Base Bitumen plus 20% 3's Rubber at 90 °C (blended at 160 °C for 1/2 hour).

APPENDIX B



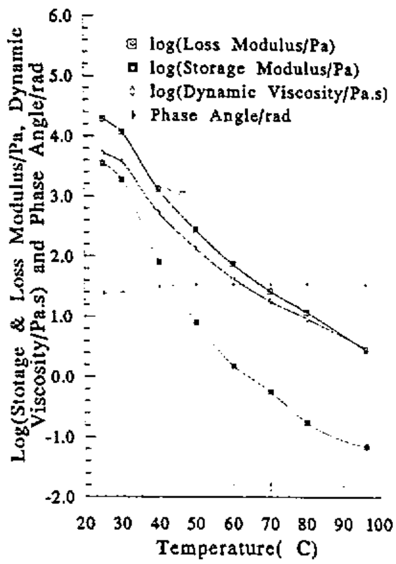
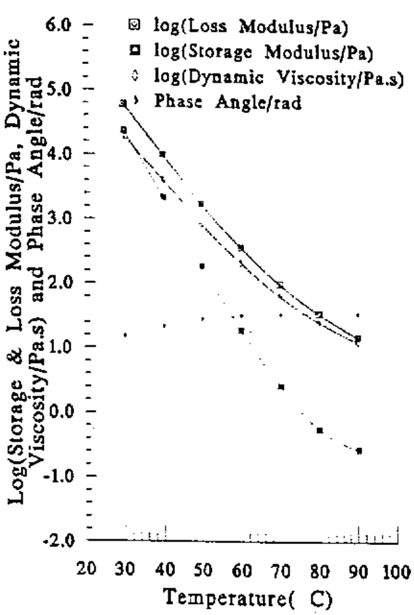


Figure B.1. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 2% 40's size Shredded Rubber.



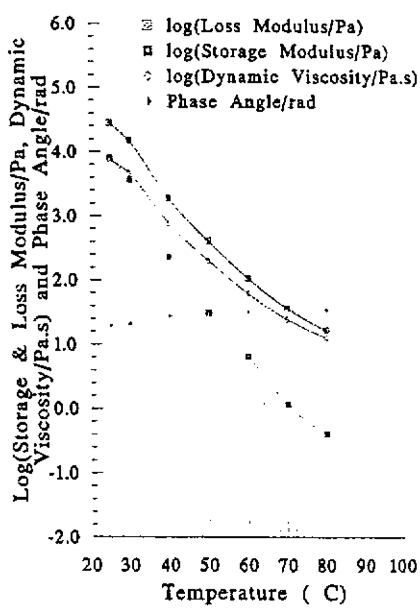


Figure B.2. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 5% 40's size Shredded Rubber.

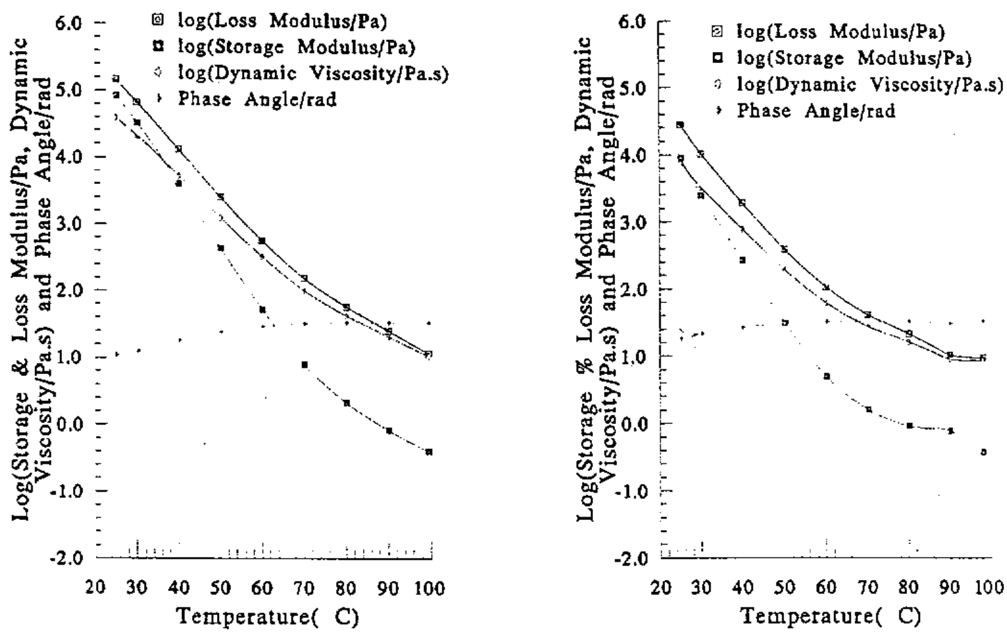


Figure B.3. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 10% 40's size Shredded Rubber.

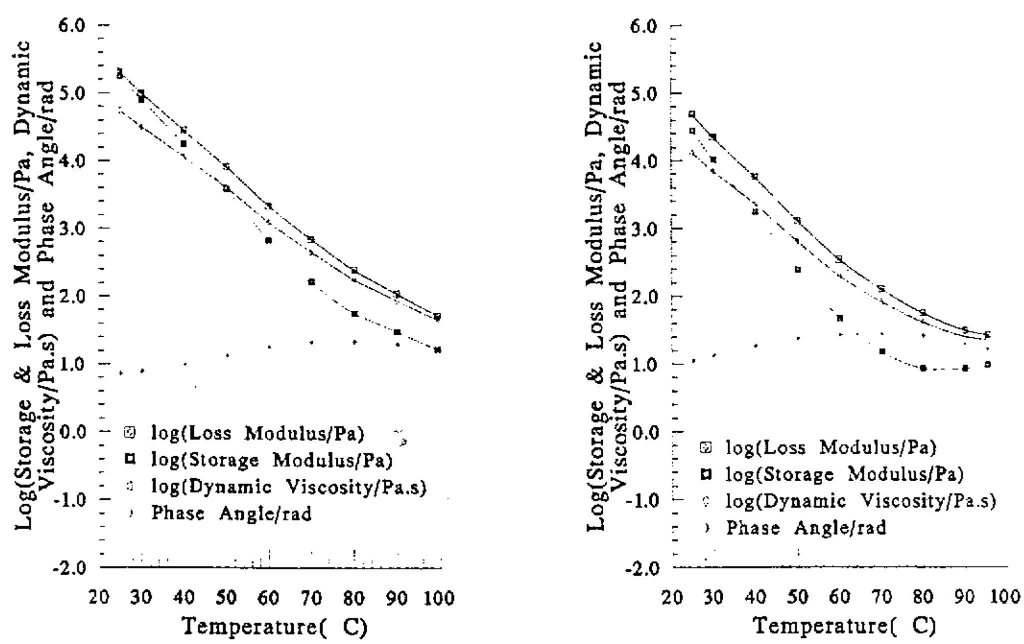


Figure B.4. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 20% 40's size Shredded Rubber.

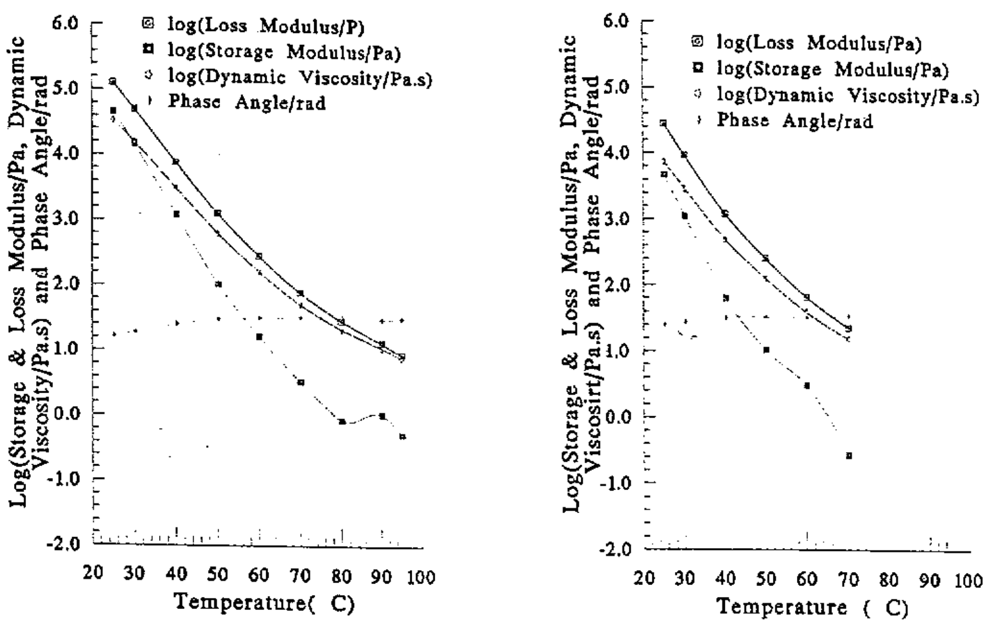


Figure B.5. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 2% 12/16's size Shredded Rubber.

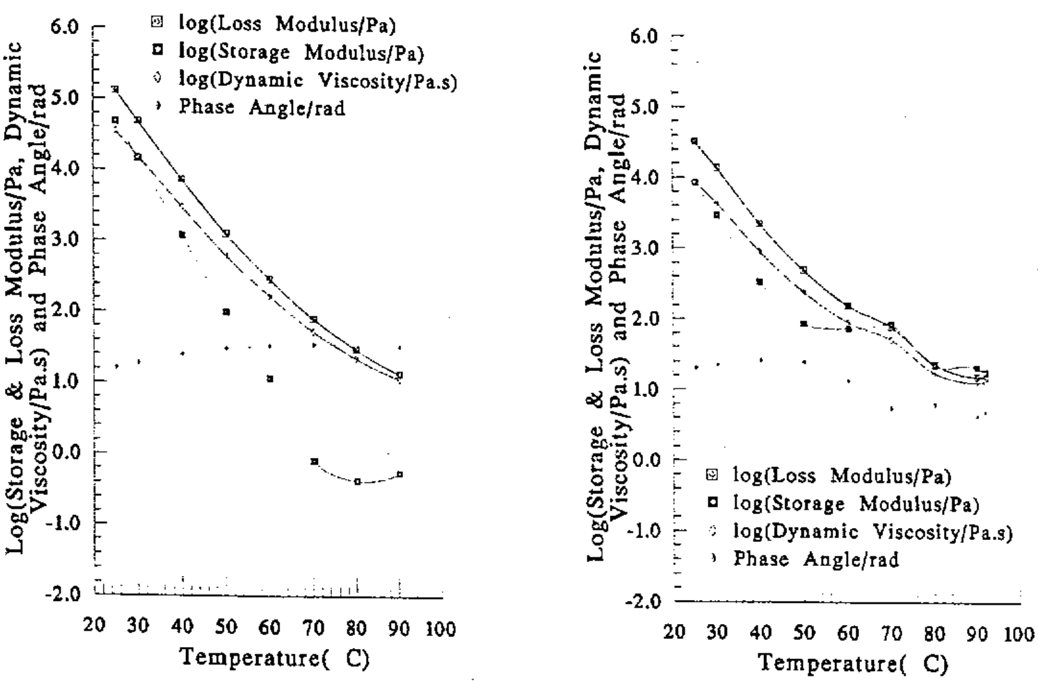


Figure B.6. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 5% 12/16's size Shredded Rubber.

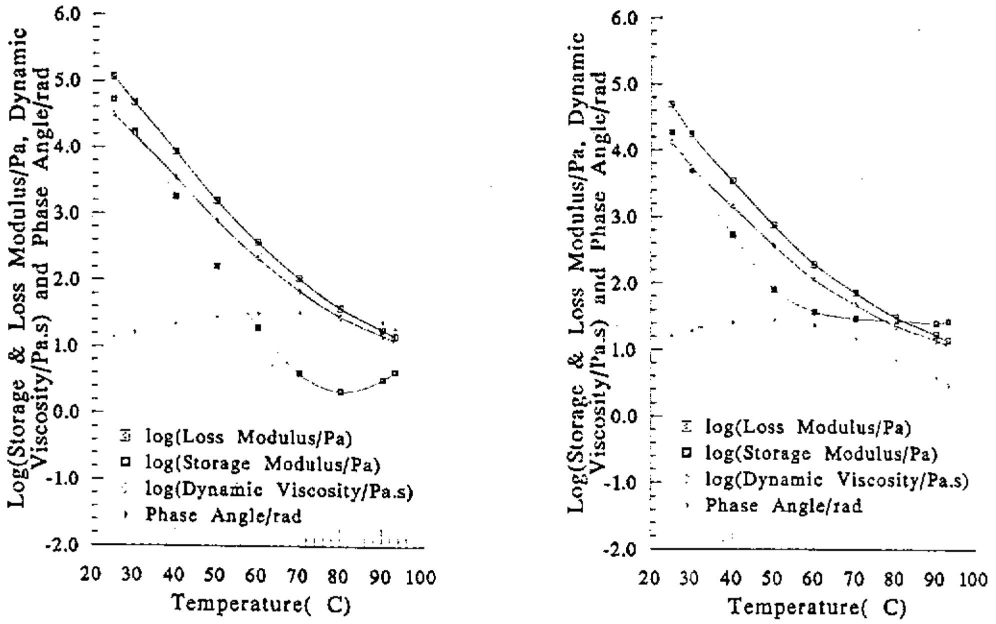


Figure B.7. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 10% 12/16's size Shredded Rubber.

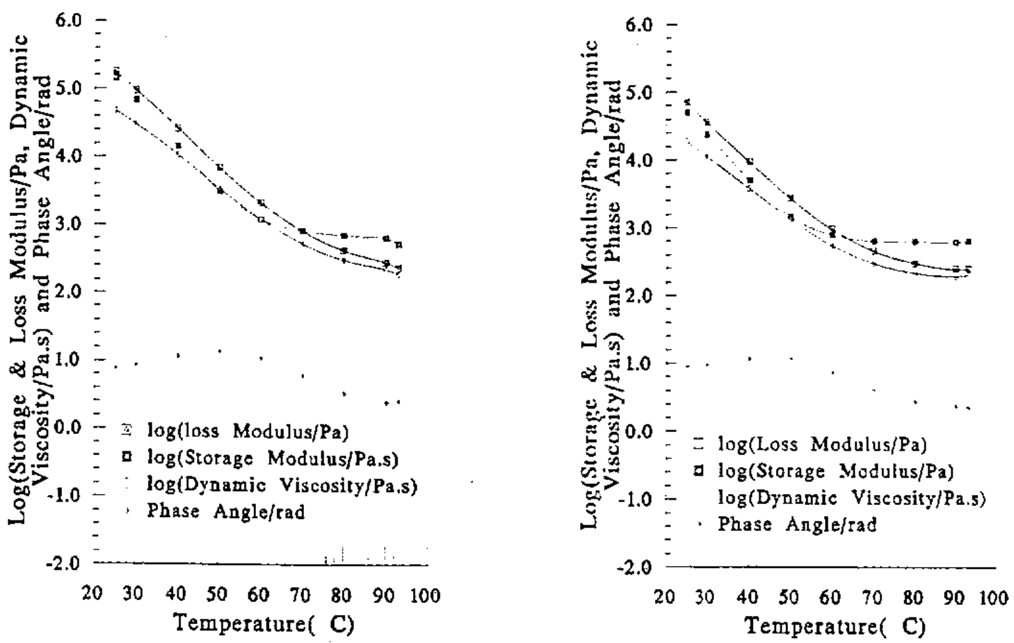


Figure B.8. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 20% 12/16's size Shredded Rubber.

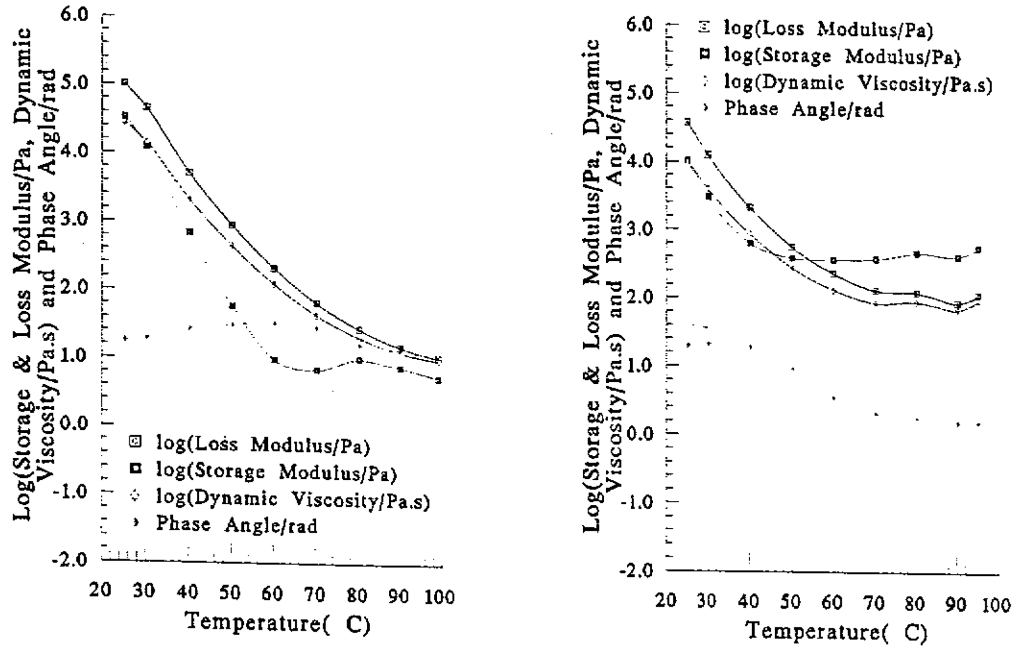


Figure B.9. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 2% 12's size Shredded Rubber.

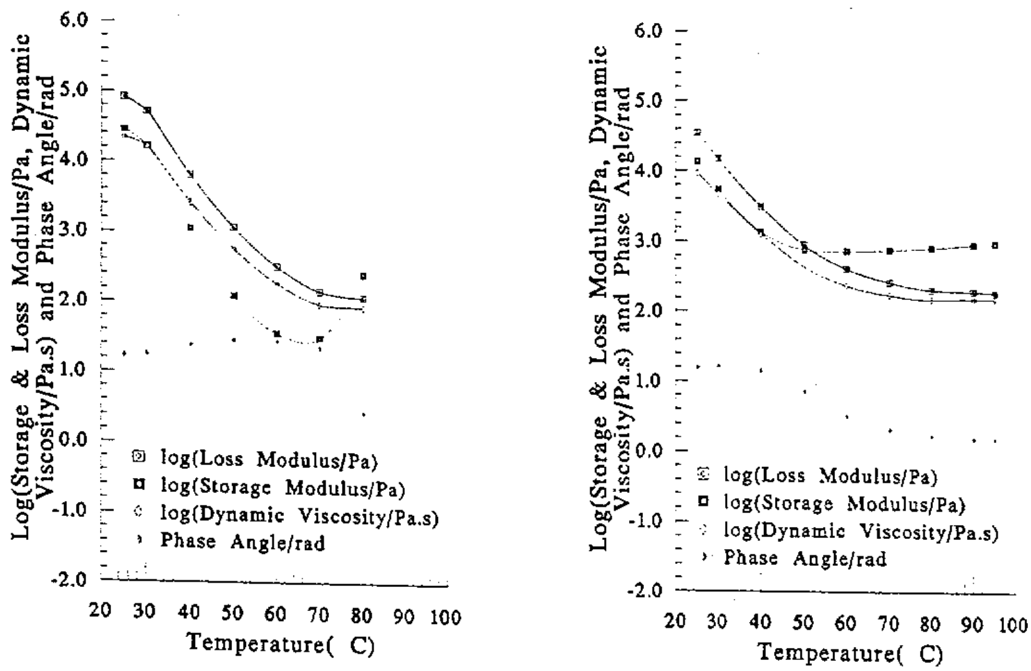


Figure B.10. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 5% 12's size Shredded Rubber.

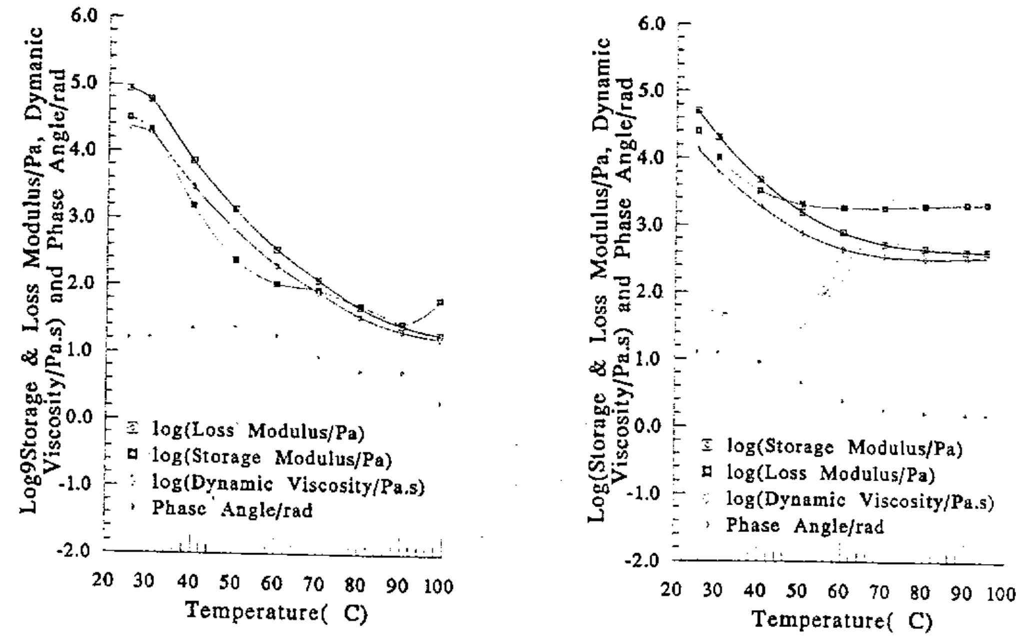


Figure B.11. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 10% 12's size Shredded Rubber.

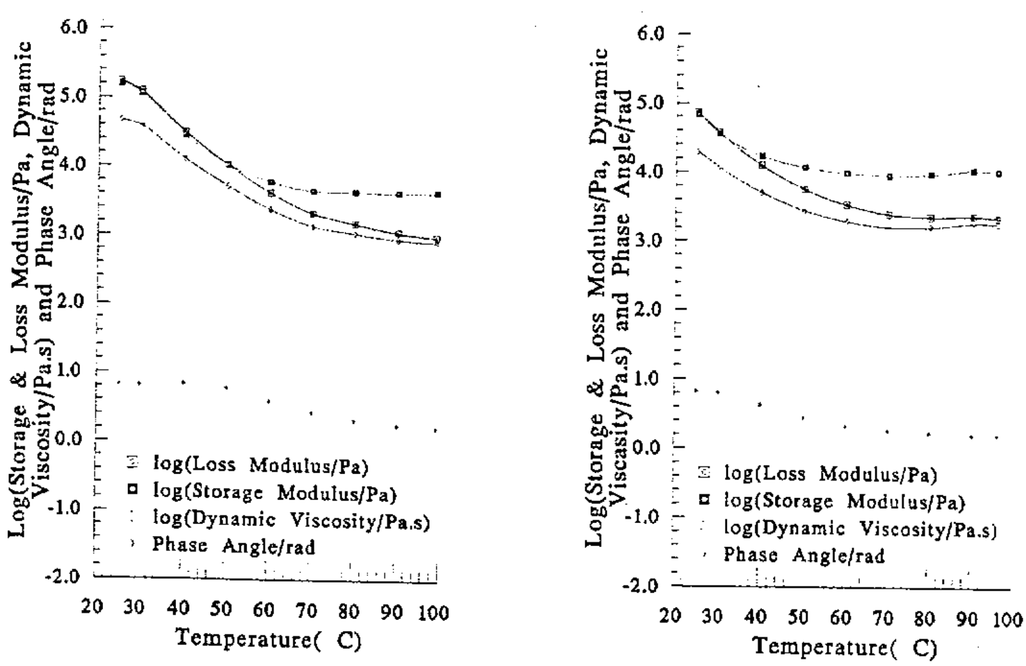
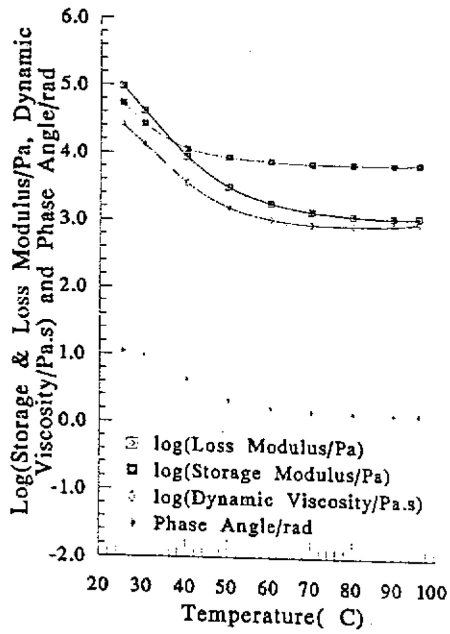


Figure B.12. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 20% 12's size Shredded Rubber.



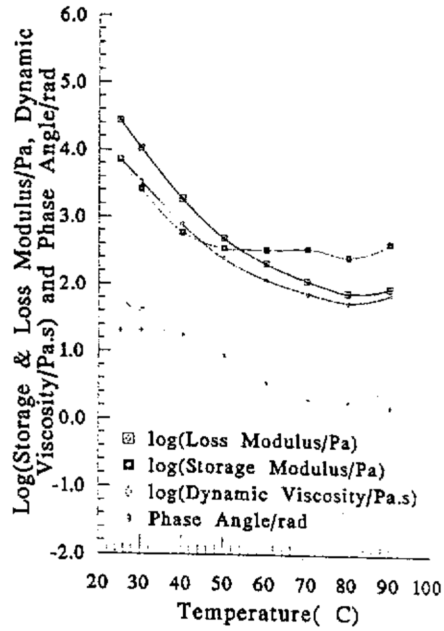
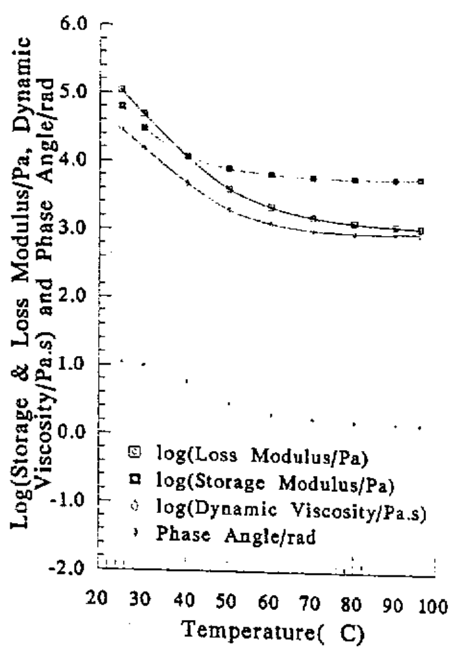


Figure B.13. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 2% 3's size Shredded Rubber.



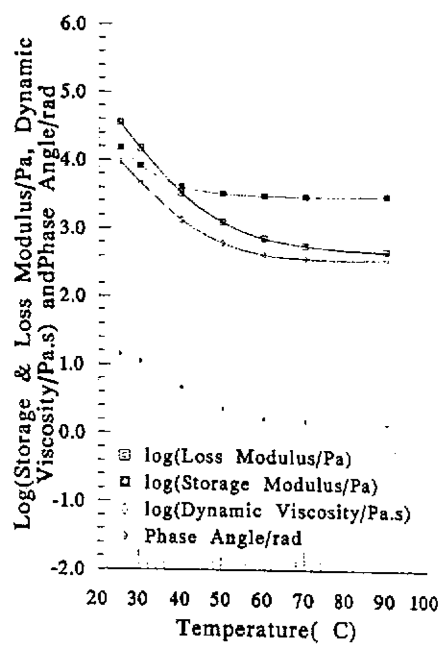


Figure B.14. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 5% 3's size Shredded Rubber.

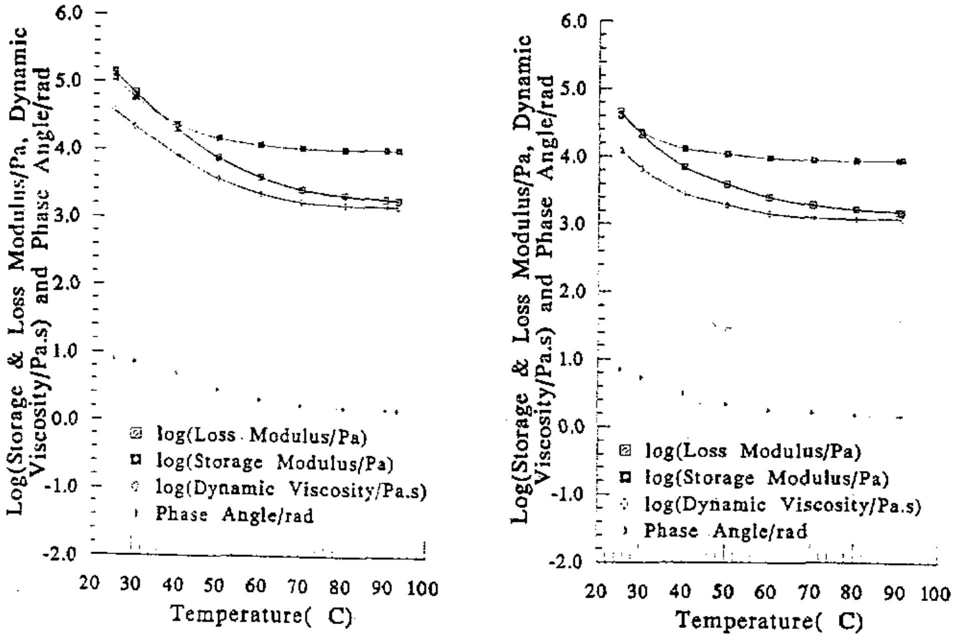


Figure B.15. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 10% 3's size Shredded Rubber.

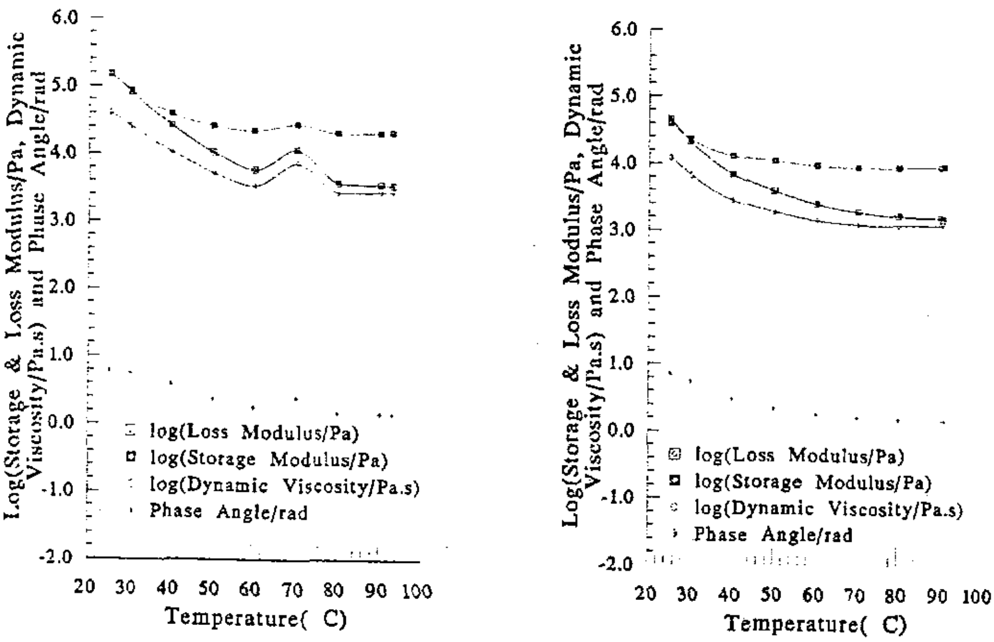


Figure B.16. Temperature vs. Rheological Properties of Rubber-Bitumen Blends made with 50 (left) and 100 (right) pen. Base Bitumen plus 20% 3's size Shredded Rubber.

APPENDIX C

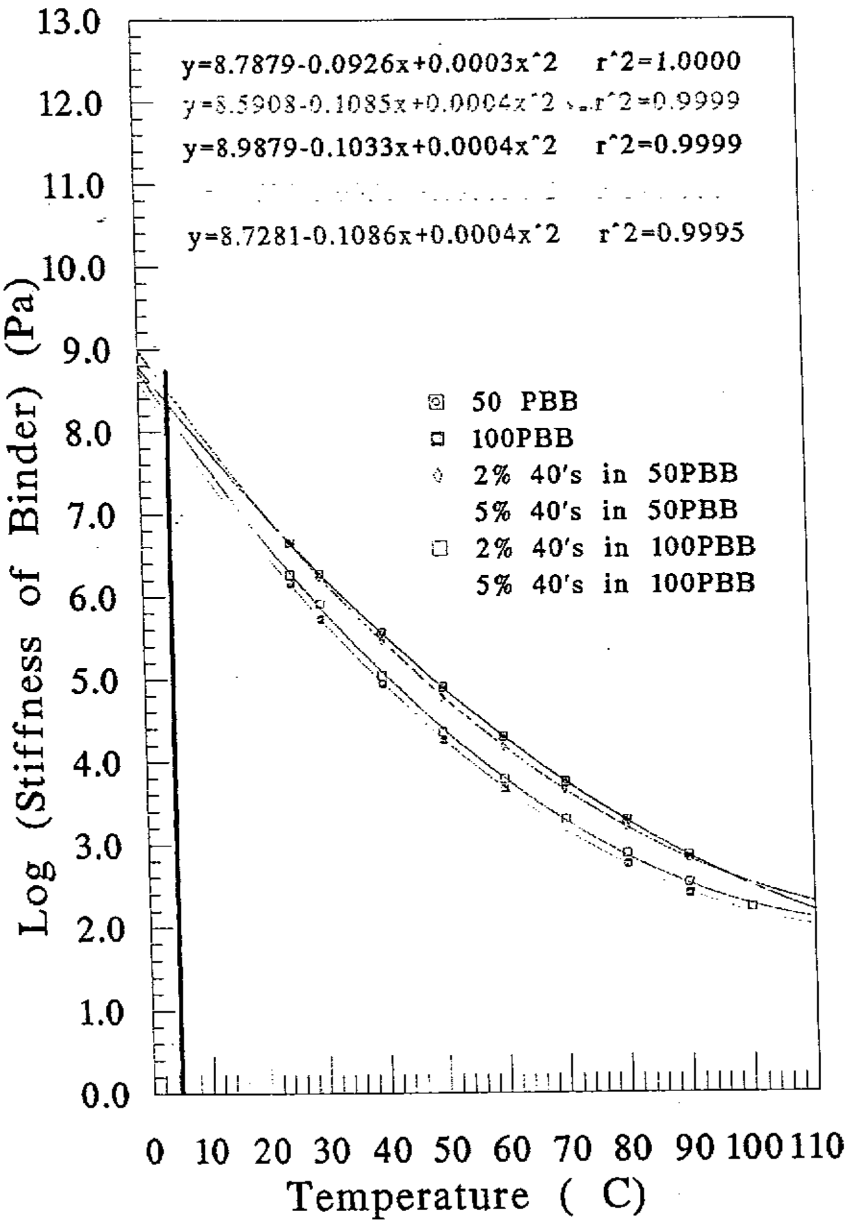


Figure C.1 Binders Stiffness calculated using by Oscillatory Shear Analysis at loading time of 3Hz for fatigue test.