Laboratory evaluation of hedmanite and lime modified asphalt concrete mixes

Mirza Ghouse Baig

Civil Engineering

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Abstract

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Laboratory Evaluation of Hedmanite and Lime Modified Asphalt Concrete Mixes

by

Mirza Ghouse Baig

A Thesis Presented to the

FACULTY OF THE COLLEGE OF GRADUATE STUDIES

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In Partial Fulfillment of the
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In

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Dated

بسم الله الرحمن الرحيم

(In the name of Allah the Most Beneficent, The Most Merciful)

سبحنك لا علم لنا إلا ما علمتنا انك أنت العليم الحكيم

Glory be to You, We have no knowledge except what You have taught us. Verily, It's You, The all Knower, The all Wise. ***(Holy Qura'n 2:32)***

صدق الله العظيم

بسم ا لله الرحمن الرحيم

إهداء

قال تعالى : " وقُل ربِّ إرحَمهُما كماَ ربّيَاني صَغيراً "

And Say: " Oh My Lord ! Bestow on them Your Mercy as They did bring me up When I was Small." (Al - Qura'n 17:24)

أهدي هذا الكتاب إلى والديّ العزيزين

Dedicated to
My Father and Mother

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THESIS ABSTRACT

Name: Mirza Ghouse Baig
Title of Study: "Laboratory Evaluation of Hedmanite and Lime Modified Asphalt Concrete Mixes"
Major Field: Civil Engineering (Transportation)
Date of Degree: October 1995.

The area of asphalt additives and extenders is somewhat complex, a variety of products and suppliers exist, and the evidence of behavior and performance is largely scattered and inconclusive. For these reasons, prospective users in the kingdom's highway agencies are facing a rather difficult task in sorting out this subject and assessing whether or not a particular additive or extender can be applied to their problem as per the environment and loading conditions, and with what technical and economic effectiveness.

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MASTER OF SCIENCE DEGREE

KING FAHD UNIVERSITY OF PETROLEUM AND MINERALS
Dhahran, Saudi Arabia.

ملخص الرسالة

الإسم : مرزا غوث بيج
عنوان البحث : التقييم المخبري لتحسين الخلطات الأسفلتية بإستخدام كل من الهدمانايت والجير.
التخصص : هندسة مدنية (طرق)
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يعتبر مجال تحسين الخلطات الأسفلتية بإضافة المحسنات مجالاً معقداً حيث يوجد هناك العديد من المواد والعديد من المنتجين، وكذلك فإن هناك تباين واضح في أداء الخلطات المحسنة . ولهذه الأسباب يواجه القائمون على تصميم الخلطات الأسفلتية في المملكة مهاماً صعبة في هذا الموضوع ، حيث توجد صعوبة في إتخاذ القرار من حيث إمكانية إستخدام مضاف معين لحل مشكلة معينة ، وكذلك تقرير تمكن الخلطة المحسنة على تحمل الطقس والأوزان التي ستمر عليها وإذا ما كان هذا الحل إقتصادياً ام لا .

وبناءً على هذا الوضع ، تم إجراء هذا البحث لتقييم أداء مادة محسنة جديدة تسمى (هدمانايت) (ألياف الصوف الصخري الطبيعي) وذلك بإضافتها إلى خليط الأسفلت لتحل محل مادة الحشو في الحصمة . تم إستخدام حصمة أبو حدرية في تجهيز عينات الخلطة المحسنة بنسب مختلفة من الهدمانايت والجير وذلك كمواد لتحل محل حشوة الحصمة . كما وتم إستخدام طريقة مارشال لتصميم الخلطات الأسفلتية في تعين النسبة المثلى لمحتوى الأسفلت في كل من خلطتي السطح والقاعدة الأسفلتيتين وتم إستخدام تلك النسبة في جميع الخلطات المحسنة بعد ذلك . إستخدمت الإختبارات الميكانيكية لتعيين الخصائص الهندسية للخلطات المحكمة والخلطات المحسنة والتي تم تصميمها بطريقة مارشال . وشملت هذه الإختبارات إختبار معامل المرونة ونسبة فقدان القوة وإختبار قوة مقاومة الإنشطار بالشد وإختبار الزحف ، وإختبار الإجهاد وكذلك إختبار التشوه الدائم . وقد لوحظ حدوث زيادة في معامل المرونة للخلطات المحسنة بنسب معينة من الهدمانايت والجير . ولكن كان تأثير الهدمانايت سلبياً في كل من إختباري الإنشطار بالشد وفقدان القوة نتيجة التعرض للماء ، ولم يطرأ أي تغيير في نتائج إختبار الرص عند إستخدام الخلطات المحسنة . كما وأبدت الخلطات المحسنة بالجير مقاومة أفضل لكل من الإجهاد والتشوه الدائم عن تلك التي تم تحسينها بالهدمانايت . دلت النتائج عموماً على أن لإستخدام مادة الجير كمادة حشو للحصمة المحلية فاعلية أكثر من مادة الهدمانايت في مجال تحسين الخلطات الأسفلتية .

رسالة ماجستير في العلوم

جامعة الملك فهد للبترول والمعادن
الظهران – المملكة العربية السعودية

Chapter 1

INTRODUCTION

1. 1 General

The development of modern highways has always depended upon the material available to build them. Early attempts to seal a pavement with tar or pitch led to the use of bitumen and to the high performance materials currently in use worldwide. This process of development continues today and although there are no completely new paving materials that are likely to become available in the near future, existing materials can be made to perform better by the addition of comparatively small quantities of additives.

The primary function of the pavement is to give the users a smooth, comfortable and safe ride at economical cost. One of the main drawbacks of bituminous pavement materials is that they combine `elastic' and `plastic' behavior, that is, when they deflect under load, a small part of the deflection becomes permanent. After individual loads these permanent deformations are practically invisible, but after repeated loading this effect can lead to rutting. The phenomenon is particularly prevalent in warm climates and where the growing weights or tire pressure of the vehicles are present.

The behavior of flexible pavement is very complex due to the inter-relation between factors influencing its performance. Some of the major observed asphalt pavement problems can be listed as [1]

- stripping
- Rutting
- Thermal and fatigue cracking
- Hardening of binder
- Flushing

In order to cope with these problems (if any), use of different types of additives in asphalt concrete mix was proposed and is in use worldwide. For example, different types of "Filler Materials" available is one such type of additive, which is known to affect greatly the properties of the mix produced.

Therefore any refinement of knowledge to enhance the use of such additives in asphalt and their potential benefits, will find a good place in todays world where lot of concern exist for these widespread asphalt concrete problems of major importance.

1. 2 PROBLEM DEFINITION

The Kingdom of Saudi Arabia is large, occupying an area approximately the size of western Europe and one-fourth the size of the U.S The population is scattered throughout the country with major concentrations in several distinctive regions. Generally arid and Hot. To provide communication between them, huge investment has been placed in constructing high quality roads that covered great distances, under extreme climatic and topographical conditions. Roads were designed for a design life of 15-20 years before any major maintenance is needed. However, during the past few years, these roads with asphalt concrete layers have been experiencing an early failure distress.

One of the major contributor to failure apart from heavy axle loads, high tire pressure and climatic conditions, is the low quality of local material used for highway construction. The local construction industry faces a serious situation in finding good performance aggregate. The solid formations along the gulf coast, which are the primary source of local coarse aggregate material for asphalt concrete, consists mostly of weak, dusty and absorptive limestone. On highways and urban roads many damaged spots can be seen after the seasonal rains, especially in eastern province where aggregate are weak limestone and sensitive to water. Since the transport of good quality aggregate from other nearby provinces is uneconomical, certain modifications to the local material must be made to ensure a durable mix .

There are a number of factors which may affect the performance of an asphaltic concrete layer. The major factors known to affect the materials characteristics and behavior under traffic loading are ; asphalt type and content, temperature variation, material type and gradation, air voids, mixture density, filler type and wheel loading or stress level. Among these asphalt content and quality of a given mix have direct effect on the stability and durability of the mixtures.

Considerable research and development has been done World-wide to achieve a mix which can satisfactorily resist the major distress problems in pavements. One of the major steps towards this is achieved by incorporating additives in asphalt mixes to improve its temperature susceptibility, especially for extreme climatic regions. Use of additives were reported by many researchers to significantly improve the rheological properties of the asphalt concrete mixes such as temperature susceptibility, strength and durability. Such promising results could present a cure for different types of distress in the pavement.[2]

Joe .W. Button in his report on " Summary of asphalt additives performance at selected sites" in 1990, stated that, on the basis of laboratory test results and findings from the older field tests in the U.S and Europe, certain polymers and microfiller additives properly applied can be expected to provide improved pavement performance [3].

General comments from representatives of state departments of Transportation about use of polymeric asphalt additives in Hot Mix Asphalt Concrete

(HMAC), in 1990 was that "They believe polymer additives offer improvements in asphalt pavement performance but they are not usually cost effective" and are expensive to use routinely on long stretches of interstate highways. In their view certain microfiller additives properly applied can reasonably be expected to provide cost effective pavement performance. [3]

Among various types of asphalt cement modifiers used "Filler Material" is one, which is considered to improve the mix properties without affecting much on the overall economy of asphalt concrete pavements. There are different kinds of filler-material available and is in use worldwide depending upon the improvement needed and the relevant functions they provide. Although different kinds of filler used in AC mixes, may additionally results in performance improvements or better economy. It is reported that each one has their own limitations. For example:

a) Hydrated lime is widely used as an antistripping agent in asphalt concrete mix, but it is found to be asphalt thirsty and thus increases the asphalt requirement of the mix thereby affecting economy and some other properties.

b) Asbestos fibers is reported to be an excellent filler material in asphalt concrete, but due to health hazard it was discouraged from using [4].

Therefore, correct selection and use of a particular type and amount of filler additive among various available and new emerging products becomes important to ensure a properly designed mix as per the local existing environmental and loading conditions.

In view of the above facts, a material called "Hedmanite" available in local market has been investigated in laboratory to evaluate the potential benefits of this material to be used in asphalt concrete mixes as a filler. Hedmanite is the commercial name of a Rockwool kind of natural fiber, obtained by crushing the rock called Lizardite. This material containing fibers in powdered form and non-pathogenic in nature has successfully proven itself as an excellent improver of asphalt mixes, especially in overlays. And is used in Canada, Austria, and in some other European countries [4]. As the cost of Hedmanite-Lizardite mineral filler is far less than other asphalt additives and it may permit the use of thinner layers of better, longer lasting pavement, It could be expected that the overall quantity of raw materials and the overall cost of paving can be considerably reduced [4].

Also literature indicates that, other types of Rockwool fibers had been used in Great Britain and France, where fibers were added to the mixture during mixing, it was reported that it improved the resistance to reflective cracking, deflection in pavement and there were no construction constraints. [5]

Filler materials such as, Baghouse fines, cement dust, limestone, fly-ash, were already investigated in the gulf region for use in AC mixes. Since Hedmanite mineral filler has not been tested before as an additive for asphalt concrete mixes in the middle-east. This research is designed to investigate the engineering properties of modified asphalt concrete mixes prepared using Hedmanite as a filler, and look for any improvements obtained as compared to lime modified mixes (since lime is considered

to be an effective modifier as filler in asphalt concrete mix) and conventional crushed stone filler mix. The key elements of the study program is shown in Figure 1.1.

1. 3 Research Objective :

The main objective of this research was to study the engineering properties of the asphalt concrete mixes prepared using Hedmanite as filler material, and to compare them with lime modified mixes and the conventional asphalt mix (containing crushed stone filler). Which require the following steps :

1. Material characterization i.e., Aggregate and Asphalt in the laboratory.

2. Mix design using Marshall procedure to come up with optimum mixes for the gradations of wearing course and base course as specified by Ministry of Communication (MOC) specification. For the mixes having crushed stone, hedmanite and lime as filler.

3. To study various characteristics of the modified asphalt mixes and carry out comparative analysis.

1. 4 Expected Benefits :

The area of asphalt additives and extenders is somewhat complex, a variety of products and suppliers exist, and the evidence of behavior and performance is largely

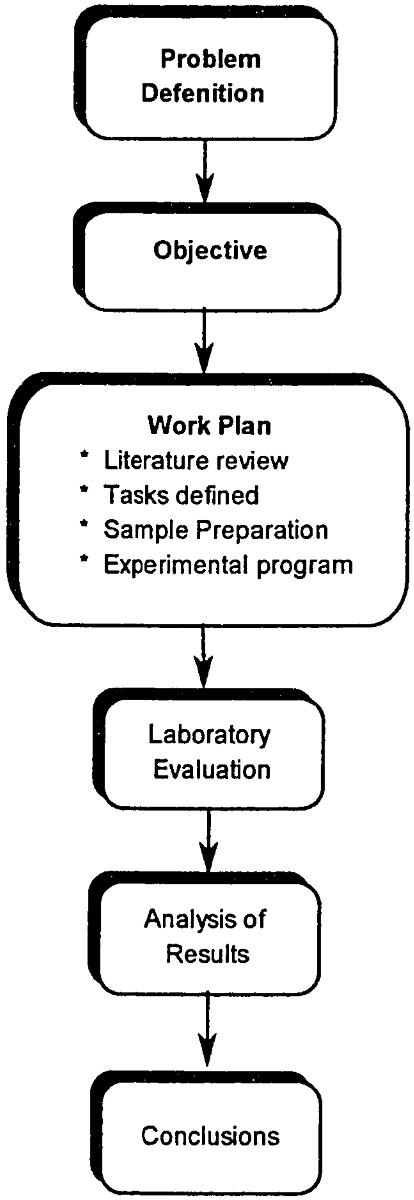


Figure 1.1: <u>Key Elements of Current Study Program</u>

scattered and inconclusive. For these reasons, prospective users in the Kingdom's highway agencies are facing a rather difficult task in sorting out this subject area and assessing whether or not a particular additive or extender can be applied to their problem as per the environment, and with what technical and economic effectiveness. In the Kingdom several studies were undertaken in order to cope up with major pavement distress problems, such as, National Research Project on Rutting in which one of the recommendation was to use the filler additives in the asphalt mix. Some other studies were also conducted to explore the use of polymeric additives like, polybelt, novophalt, crumb rubber, sulfur etc. In view of this situation the output of this research program is expected to be a step in evaluating or deciding the use of new locally marketed material in Saudi Arabia.

1. 5 Study Approach:

In order to achieve the study objectives, a systematic approach consisting of three main interconnected phases have been proposed :

The first phase consist of material collection and characterization. The second phase involve mix design and laboratory evaluation. The third phase involve data analysis, conclusions and recommendation. A schematic flow chart for study approach is shown in Figure 1.2.

Phase 1: Material collection and Characterization.

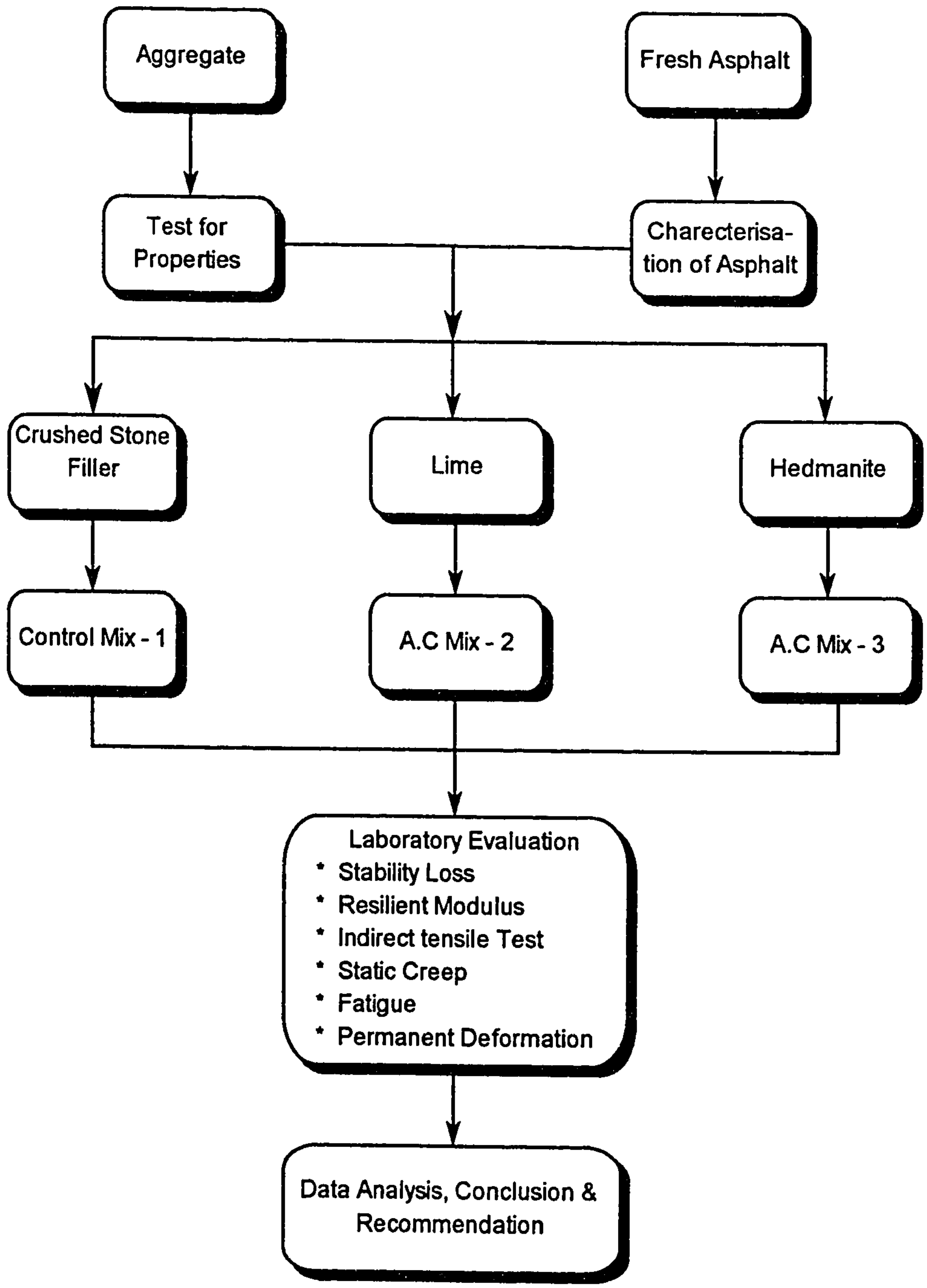


Figure 1.2 : EXPERIMENTAL PROGRAM

i) Material collection: This involve collection of materials to be used in the Mix design. Which include Abu-hadriyah Aggregate, Asphalt from Ras-Tannurah Refinery, Hydrated Lime and Hedmanite from local suppliers.

ii) Characterization of the binder is as follows:

- Specific gravity (ASTM D-70)
- Viscosity @ 135° C and 60° C (AASHTO T-202)
- Penetration (ASTM D-5)
- Softening point (ASTM D-36)
- Ductility (ASTM D-113)
- Flash point (ASTM D-92)
- Asphalt Aging by Thin-Film Oven test (TFO) (ASTM D-1754)

iii) Aggregate testing include:

- Specific Gravity
  * Coarse aggregate (ASTM C-127)
  * Fine aggregate (ASTM C- 128)
  * Filler (ASTM C- 128)
- L.A. Abrasion Test (ASTM C-131)

- Soundness Test (ASTM C-88)
- Sand Equivalent (ASTM D-2419)
- Plasticity (AASHTO T-90)

Phase 2: Mix Design and Laboratory Evaluation

This Phase involves the following tasks:

i) Optimization of mixes by Marshall Mix Design Method (ASTM D-1159) using the aggregate gradations as specified in modified MOC specifications (Table 1.1). For different fillers i.e.;

- ♦ Crushed Stone
- ♦ Hedmanite
- ♦ Lime

ii) Evaluation of optimized mixes includes

- Marshall Stability test @ 25° C and 60° C. (ASTM D-1559)
- Modulus of Resilient test @ 45° C. (ASTM D-3497)
- Indirect Tensile Strength @ 25° C. (AASHTO T-245)
- Static Creep test @ 60° C. (Reference # 48)

Table 1: Agregate Gradation (as per modified MOC specifications*).

Seive Size	% Passing	
	G1	G2
1 1/2"	-	100
1"	-	75 - 90
3/4"	100	65 - 80
1/2"	76 - 92	55 - 70
3/8"	64 - 79	45 - 60
# 4	41 - 64	31 - 46
# 8	23 - 37	-
# 10	-	18 - 33
# 40	7 - 20	5 - 18
# 80	5 - 13	3 - 13
# 200	3 - 8	2 - 9

G1 - Wearing Course
G2 - Base Course
' - ' Indicates the size is not included
' * ' MOC Specifications, March 1986. [36]

- Fatigue test @ 45°C and 60°C. (ASTM D-3497)
- Permanent Deformation @ 45°C and 60°C. (Reference # 51)

These tests will provide a base for comparison between the necessary properties of modified and conventional mixes.

Phase 3: Data Analysis and Results

This phase involves the analysis of test results obtained from different tests. It shows the various characteristics of modified asphaltic concrete mixes and the conventional asphalt mix for the comparision. And in order to come up with conclusions and recommendations regarding the engineering properties of Hedmanite and Lime modified asphalt concrete mixes.

Chapter 2

LITERATURE REVIEW

2. 1 Asphalt Additives

An asphalt cement (AC) additive is a material which would normally be added to and/or mixed with the asphalt before mix production, or during mix production, to improve the properties and/or performance of the resulting binder and/or the mix, or where an aged binder is involved, as in recycling, to improve or restore the original properties of the aged binder. An asphalt cement extender is an additive which replaces a part of the AC that would normally be used in the mix, and may additionally result in performance improvements or better economy [5].

The justification or reasons for using an additive or extender would include the following:

1. Solve or alleviate a pavement problem.

2. Realize some benefits such as

i) Economy

ii) Environmental

iii) Energy

iv) Application and Performance.

2. 1 .1 Introduction

The concept of modifying asphalt binders and mixtures is certainly not new, but has become much more prominent during the past few years. One reason for this resurgence in interest has been the changing process of how oil refineries obtain and process crude oil. Following the 1973 Arab oil embargo, the traditional sources and supply lines changed. Many refineries that were accustomed to a single crude source and supply lines changed. Also many refineries that were accustomed to a single crude source were faced with the prospect of processing oil from multiple sources. These changes made it more difficult to meet specifications for paving grade asphalt cement. This situation provided additional opportunities for enhancing asphalt cement through modification.

Other factors that may have some influence on an increased interest in modifying asphalt cement include at least the following:[2]

- Traffic factors have increased; including heavier loads, higher volume, and higher tire pressures.

- To accommodate the shift from larger projects such as the Interstate System to smaller projects such as maintenance of the existing road network.
- Higher costs have created a tendency to construct thinner pavements, thus reducing the service lives of pavements.
- Environmental and economic pressure to dispose of certain industrial waste materials (i.e., tires, glass, ash, etc.) has prompted the idea of converting them to asphalt cement additives.

Because some of these problems existed in Europe prior to their emergence in the U.S., there was an earlier move towards asphalt modification in Europe. Also, some countries require contractor guarantees for performance and this promoted the use of modifiers in an attempt to ensure better performance and to lower life cycle costs. This approach is in contrast to the U.S. where low initial cost is the governing factor in the existing bid process [2].

A family of products and processors are aimed at a variety of pavement application. The highway engineer knows that the complexity of pavement distress requires a choice of repair or rehabilitation options (one or two methods may not suffice). The appropriate modification of asphalt binders has broadened the choices available to the engineer.

Engineers who are familiar with the field performance of Hot Mix Asphalt (HMA) pavements generally agree on three potential modes of distress: [2]

1. Distortion

    a) Settelment

    b) Rutting

2. Cracking

    a) Repeated load (fatigue cracking)

    b) Non-load (thermal cracking)

3. Disintegration

    a) Raveling

    b) Stripping (moisture damage)

Although most HMA pavements perform satisfactorily, problems still do occur. Consequently, there is an increased interest in making changes that include several possibilities:

- Improved pavement design (structural, drainage, materials, etc.).
- Revision of specifications for paving materials and pavements.
- Improvement in the quality control of construction.
- Improvement of binders systems.

All of these will be necessary for improvement of pavement performance, however the binder system has gained a primary interest [5].

2.1.2 Types of Additives

The generic classification has led to the following types of asphalt cement additives [2].

* Filler
* Extender
* Polymer
* Rubber
* Plastic
* Fibers
* Oxidant
* Antioxidant
* Hydrocarbon
* Antistripping agents
* Combinations

Each of the additives noted above provides benefits and improvements to the asphalt binder and for mixture, either actual or perceived. The impetus to use one or more of these modifiers is generally based upon several factors. For example, a user agency may have a particular pavement problem and is in need of a solution. They in turn

seek out additives or modifiers that provide some hope. Another approach has been to seek new markets for materials that are already available and have traditionally been used in other applications [5].

2. 1 .3 Outlook for Additives

Vehicle weights, traffic volume, and tire pressures are steadily increasing and demanding more and more from pavement structures. Engineers are faced with serious problems regarding quality of paving material. Often aggregates are shipped long distances at high cost because local aggregate supplies of high quality have been depleted. As a result, bituminous binder additives have been widely accepted by the paving industry for the present time. The concept of additives is logical, and results from laboratory testing look positive. Even though field test results using many additives are incomplete, many of those responsible for pavement quality are willing to use because the results appear to be in their favour [3].

The bituminous binder additive industry and associated technology are advancing at a rapid rate. By the time results from the field are available for the additives being currently marketed, it is reasonable to assume that a whole new generation of bitumen additives will be on the market. It is, therefore, surmised that the outlook for additives in asphalt paving materials is excellent.

2. 2 *Mineral Fillers as Additive*

2. 2 .1 Definition

Any fine powder added to bituminous mixture in the course of manufacture, and which has been ground to such a degree of fineness that not less than 85 percent by weight passes a 75 micron sieve [6] is called a **"Filler"** .

Examples of Filler are :

1. Mineral Fillers :

    i) Crusher fines

    ii) Lime

    iii) Portland Cement

    iv) Fly ash

    v) Granite dust

2. Others :

    i) Carbon Black

    ii) Sulfur

    iii) China Clay and Fuller's earth.

Mineral Fillers: They are generally considered to be fine inert mineral materials a high proportion (at least 65 percent by ASTM and AASHTO specifications) of which will pass the No. 200 sieve". [7]

This description is improved by adding a statement to the effect that filler is important because of the surface area involved, and that properties of a pavement which may be improved by the use of filler include strength, plasticity, amount of voids, resistance to water action, and resistance to weathering. In short, if filler is to be adequately described it is necessary to turn to the literature to try to determine what others have learned about it, or to attempt independent analysis in the laboratory and field.

2. 2 .2 Background

Extensive research, most of it from the early part of the century, has been done on the properties of mineral filler and its influence on asphaltic concrete mixtures.

Richardson [8] was one of the first investigators to report on the effects of mineral fillers. He postulated that the function of the filler is more than mere void filling, inferring that some sort of physicochemical interaction occurs when fine mineral dust is added to asphalt cement.

By the late 1930's many studies had been completed on the properties of mineral fillers and mineral filler-asphalt investigation of fillers with respect to their performance in asphaltic concrete, Traxler [9] considered size and size distribution as fundamental filler properties in that they affect the void content and average void diameter of packed powders. More recent work by Traxler confirms his earlier findings [10].

Mitchell and Lee [11] also attempted to find a single parameter that would adequately predict the ability of a mineral filler to stiffen the asphalt to which it is

added. Their data were obtained for mineral filler-asphalt mixtures with relatively small concentrations of solids. Their results indicated that the bulk settled volume of filler in benzene is a good predictor of the performance of the mineral filler.

A very extensive series of experiments on mineral fillers and mineral filler-asphalt systems has been reported by Rigden [12]. In particular, he studied the relationship between filler properties and the viscosity of mineral filler-asphalt mixtures. At filler-asphalt ratios similar to those found in typical asphaltic concrete mixtures, the fillers stiffened the asphalt by as much as three orders of magnitude. His data also indicate that fillers affects the temperature susceptibility of the asphalt, however, the stiffening effect did not correlate with any of the fundamental properties of the fillers.

The rheology of mineral filler-asphalt systems has been studied by Winniford [13] using the sliding plate microviscometer. Winniford suggested that the role of the filler is more than volume filling, and postulated additional stiffening mechanisms including:

(1) A gelation of the asphalt by the mineral surface, which increases the non-Newtonian flow characteristics and lowers temperature susceptibility.

(2) Formation of thick viscous coatings which increase the effective solids concentrations, and

(3) Surface shielding by absorbed asphaltenes. It was also shown that the stiffening effect of the mineral fillers was more pronounced with smaller sized material.

Tunnicliff has comprehensively reviewed the research on mineral fillers prior to 1967 [14,15]. He concluded that a substantial amount of the mineral filler acts as though it is part of the asphalt film.

Warden et al. [16] presented data on filler-asphalt mixes in conjunction with field observations. This study was motivated by field failures that were attributed to filler type. An easily measured parameter was sought that would predict the performance of the filler in the field. The tests performed on the fillers were empirical tests in use in the late 1950s. A reexamination of the early work by Traxler again demonstrated that no single parameter was sufficient to predict the behavior of different mineral fillers. The softening point of the filler-asphalt mixtures was found to be critical with respect to filler type.

Puzinauskas [17] reporting on The Asphalt Institute study of mineral fillers, concluded that the mineral filler plays a dual role in asphalt mixtures. He stated that "they are part of the mineral aggregate, they fill the interstices and provide contact points between larger aggregate particles, when mixed with asphalt, mineral fillers form a high consistency binder or matrix which cements larger aggregate particles together".

Anderson and Goetz [18] used rheological parameters to study the stiffening effect of fine mineral powders on filler-asphalt mixtures. A number of powders were separated into closely sized fractions: 0.63 to 1.25 μm, 2.5 to 5.0 μm, and 10 to 20 μm. Their studies showed that the rheological behavior of the mineral filler-asphalt mixtures appended on the size and mineral properties of the filler and the source of the

asphalt. The stiffening effects of the filler were relatively small at short loading times or low temperatures, but were very large at higher temperatures and long loading times. The temperature susceptibility of the asphalt increased with the addition of mineral filler. The authors concluded that a single test on mineral filler cannot be expected to predict the behavior of the filler in an asphalt mixture.

Craus et al. [19] dealt with the effect of the physicochemical properties of filler on mixture performance. In particular, they examined the geometric characteristics (shape, angularity, and surface texture), adsorption intensity at the filler asphalt interface, and the selective adsorption of the filler-asphalt system. They concluded that the physicochemical interaction between filler and asphalt increased with the adsorption intensity, geometric irregularities, and selected adsorption of the fillers.

2. 2 .3 Theory of filler

Two fundamental theories, based on the results of studies, observations, and experience, have emerged regarding the functions of fillers in bituminous mixes.

1. **Filler theory :**

The filler theory postulates that "the filler serves to fill voids in the mineral aggregates and thereby create a denser mix".

This theory presumes that each particle of the filler is individually coated with asphalt and that such coated particles, either discrete or attached to an aggregate

particle, serve to fill the voids in the aggregate. By virtue of such filling of voids, mixes of higher stability and density can be attained. [20]

2. Mastic Theory :

The Mastic theory proposes that the filler and asphalt combine to form a mastic which acts to fill voids and also bind aggregate particles together into a dense mass [20].

When filler is added to asphalt, part of it will have a mechanical function where physical contact is not established, then filler and asphalt work together in the form of what can be called a binder. This finest portion of filler will be suspended in the asphalt, changing the properties of binder films. It will act as a filler within the asphalt itself, since it will replace a certain amount of asphalt in the mixture. A Mastic of this type is harder, stiffer, tougher, and possesses a lower temperature susceptibility than the original asphalt cement. [21]

2. 2 .4 Filler Attributes :

The desired practical and functional quality attributes in a filler material should include the following [16]:

The filler in the completed mix must be non-critical. Variations in the filler content which may be expected under normal plant operation must not cause undesirable fluctuations in the physical properties of the pavement. The yardstick or means of judging is the sensitivity of all the following quality attributes as a function of F / A (filler - asphalt) ratio.

The quantity of filler desired for functional reasons must not unfavorably affect the mixing, placing and compaction of the bituminous mixture. In other words at the desired concentration to meet design criteria the mortar softening point or consistency must not be so high that the mix is unworkable.

Added mineral filler should be economical (availability and cost) and should be readily transported, stored, proportioned and mixed with customary equipment. Yardsticks for storing and proportioning are that the filler be non-hygroscopic and not form lumps or cake or bridge in the bins.

A completed pavement surfacing must be stable and durable over a wide range of temperature and over an extended period of time. This means that from the functional viewpoint the type and quantity of filler in the bituminous mixture must be such that the viodage is maintained within the desired limits, both initially and after ultimate compaction, and that there is sufficient resistance to deformation by traffic at the highest service temperature. Concurrently the filler must not decrease the resistance to water or the bond of the asphalt or mortar to the aggregate and must not decrease durability through loss of flexibility by inducing cracking of the pavement [16].

2. 2 .5 Role of fillers in AC mixes :

In general the functions of a filler can be listed as follows [6]:

1. To increase the viscosity of the binder and hence increase density and stability of the mixture.

2. To enable a thicker film of binder to be held by the mixes.

3. To improve the resistance of the binder to weathering.

4. To increase the effective volume of the binder.

5. To reduce the apparent temperature susceptibility of the mixture (for dense surfacing - filler/binder mixtures have lower temperature susceptibility then straight binders of the same viscosity).

6. It tends to reduce the brittleness of a mix in cold weather, where the quantity of the filler can be considerably increased.

7. It gives a close texture on the surface after compaction.

The role of mineral fillers in asphalt mixtures was addressed in a comprehensive paper by Heukelom [22]. Bitumen number, dry compaction, and the kerosene absorption test were used to determine the void characteristics of mineral filler. The bulk volume (definied as the total filler volume to filler solids plus voids, at the condition of densest packing) determined from the kerosene absorption test yielded approximately 17 percent greater than that obtained from the dry compaction procedure. Assuming that the penetration index (temperature susceptibility) of the asphalt and the filler-asphalt mixtures is the same, Heukelom measured the softening point of the filler-asphalt mixtures and calculated the stiffness of the mixtures. A unique relationship was found between stiffness ratio (stiffness of filler-asphalt mixtures to the stiffness of neat asphalt) and percent bulk volume, $\%V_{DB}$ (defined as the bulk volume obtained from compaction divided by the total volume of the filler-

asphalt mixtures). When this concept was extended to asphaltic concrete mixtures, it was found that stiffness and compatibility are roughly related to the percent bulk volume of the filler.

In the Kingdom, Al-Abdul wahhab H.I [21] in his research at KFUPM, study the effects of baghouse fines on asphalt mix. A number of mixes that had various ratios of filler to baghouse fine were analyzed. The study indicated that baghouse fines can greatly affect the optimum asphalt content, stability, and stability loss of the mix. He stated that the stability loss, which is a main factor in the design of local mixes was found to be decreased drastically by the inclusion of baghouse fines.

Bassam A. Anani et.al [37], in their study on control of filler contents and compaction on asphalt mix properties had reported that, the degree of compaction and filler content can vary and still produce acceptable mixes. Air voids is the most important, which can directly affects the MR and the water resistance of asphalt mixes.

Other researchers have related the void properties of the filler to the Marshall mixture properties. For example, Hudson and Vokac [23] have related the activity coefficient to Marshall stability. The activity coefficient is defined as the bulk volume of the filler to the solid volume of the filler. The bulk volume of the filler was determined from the settled volume of the filler in kerosene. For a given mixture, it was found that the activity coefficient is related to Marshall stability. It was concluded , however, that the stability is a function of both filler type and concentration.

Craus et. al. [19] concluded that the physicochemical interaction between filler and asphalt increased with the adsorption intensity, geometric irregularities, and selected adsorption of the fillers. These effects strengthen the filler-asphalt bonds producing a mixture with a higher strength.

Summarizing the key points from the state-of-the-art-review on mineral fillers, it can be concluded that:

1. Mineral fillers stiffen asphalt, and the degree of stiffening varies significantly between different fillers.
2. For a given filler source, the finer the filler the greater the stiffening effect.
3. Although performance varies for different fillers, there are no exact tests that can adequately predict their performance.
4. Different fillers may react differently with different asphalts.

Fiber material used as filler in AC mixes :

Fiber provide some sort of reinforcement in the AC mixtures. They also provide a finely divided material in the mix with a high surface area that permits the application of thicker then normal films of asphalt cement on the aggregate [2].

Fibers are of two types [2]

1. Natural Fibers.

Example : Asbestos (Hazardous),

Rockwool (Non-Hazardous).

2. Man-made Fibers.

Example : Polyester,

Fiberglass,

Steel Fibers.

Natural, Synthetic and Steel Fibers have all been used in Hot Mix Asphalt (HMA). The usual approach is to incorporate very fine, short Fibers into the binder (usually conventional asphalt cement) or aggregate mixture, depending upon its form, chemistry, and intended function.

Thomas L. Speer et. al. [24] in their study on control of asphalt pavement rutting with fiber had reported that, Chrysotile asbestos fiber was the most effective mineral tested and the only admixture that reduced rutting below critical levels at the highest operating temperatures. The asbestos permitted a large increase in asphalt content, from 30 to 50 percent above that used in standard asphalt mixes yielding acceptable performance records. They found that a 2.5 percent asbestos addition produced the desired reduction in rutting at temperatures up to 140° F.

According to this research, Fiber linkage is a mechanism that may explain the resistance to rutting which asbestos imports in asphalt paving mixtures. Selective adsorption on the short chrysotile asbestos fiber could bond or link together the heavy, viscous asphalt fraction. Pavement stability against rutting would then depend only

on the strength of the heavy fraction, the amount present in the paving asphalt, and the proportion adsorbed by the asbestos fiber [24].

To illustrate as an example, when asbestos-asphalt paving was proving itself in the 1960's, the price of asphalt was $18 per metric ton, about half the price of asbestos. Even at that price in 1969, Five thousand feet of runway at St. Louis, Missouri Airport was surfaced in $3\frac{1}{2}$ days with a 4 inch layer of regular asphalt mix topped with a 1 inch course of asbestos-asphalt costing only $193,370. Concrete would have required an 8 inch slab costing up to $8 million, with shutting down the runway for 2 to 3 months. The Engineers figured that any life beyond one year would be dividends. Those dividends have been multiplied by more than 15 years.

Apart from asbestos, non-hazardous Rockwool fibers have been used in Great Britain and France, where fibers were added to the mixture during mixing, it was reported that it improved resistance to reflective cracking, deflection and there were no construction constraints. [5]

2. 3 Lime and Hedmanite

2. 3 .1 Lime

Hydrated Lime, calcium hydroxide $Ca(OH)_2$, commonly used in soil stabilization have also traditionally been used in HMA as a filler to improve AC mixtures properties. Lime perhaps have special binding qualities in addition to the role of filler. It has been

used for the purpose of providing stiffening or reinforcement to the binder as well as `Filling in' the voids in the aggregate matrix [25].

Hydrated lime is a dry powder obtained by hydrating quicklime with enough water to satisfy its chemical affinity, forming a hydroxide due to its chemically combined water [26]. It has a surface area of 17-24 m^2/gram.

Lime is a general term that connotes only a burned form of lime, usually quicklime, but may also refer to hydrated lime. It may be calcite, magnesium or dolomitic. It does not apply to limestone or any carbonate form of lime (although it is often erroneously used in this way) [26].

Hydrated lime has gained considerable recognition as a useful additive for improving the performance of asphalt pavements. It is added to some low-grade aggregate to render them suitable in asphalt mixtures for use in highway construction. Sometimes it is difficult to coat certain aggregate with asphalt because of their siliceous or acidic surfaces. Hydrated lime, which is highly alkaline, starts a chemical reaction that changes the character of the aggregate surfaces and neutralize any acidic properties present in the asphalt. Adding hydrated lime often improves the coatabilty and bonding properties of asphalt of these aggregates [26].

Thomas W. Kennedy et. al. [27] in their study on "Techniques for reducing moisture damage in AC " reported that both dolomitic and calcite hydrated lime has been found to be a very effective additive. It is recommended that the lime be added to aggregate in the form of lime slurry, also adding dry lime is effective if the lime can be held on the aggregate surface until coated with asphalt. Nevertheless, the final

decision should be based on relative effectiveness and cost. Indirect test results indicated that both dry lime and lime slurry were effective in reducing stripping and moisture damage and 1.5% lime has given highest split tensile strength.

Plancher, Dorrence and Peterson [28] in their research suggested that the Hydrated lime absorbs carboxylic acids in the asphalt which increases the water resistance and asphalt aggregate bonds.

Welch and Wiley [29] studied the effect of hydrated lime on asphalt and aggregate mixtures and found that hydrated lime changes the mechanical properties of asphalt mixtures. It has been shown by several investigators that the addition of minor quantity of basic oxides such as calcium hydroxide, calcium oxide, and Portland cement helps to maintain adhesion in the presence of water, and retard oxidative hardening [30].

The report on "Lime Treatment of Asphalt Mixes to reduce age hardening and improve flow properties" by a distinguished scientist, Peterson J. Claine indicated that lime treatment of asphalts reduced asphalt age hardening, increased the high-temperature stiffness of unaged asphalts, reduced the stiffness in aged asphalts at higher temperatures, and increased the asphalt tensile-elongation at low temperatures. These effects will benefit asphalt pavements by increasing asphalt durability, reducing rutting, shoving and other forms of permanent pavement deformation, improving fatigue resistance in aged pavements, and improving pavement resistance to low-temperature transverse cracking. These benefits are in addition to the well-documented effect of lime in increasing the resistance of pavements to moisture

damage. Although the relative response to lime treatment varied as a function of asphalt source, all sources studied in their research were benefited significantly by lime treatment. The net result of the combined effects of lime treatment should result in longer lasting pavements with improved performance during the life of the pavement. The beneficial effects of hydrated lime on the aging characteristics and the low-temperature flow properties were not found in case of pulverized limestone [31].

When dry powder is added to dry aggregate, the batch of mineral aggregate shall be dried, composited, and heated to 300° F. The required quantity of additive shall be added to the aggregate, and the entire mass shall be thoroughly mixed until a uniform distribution of additive has been achieved. Care shall be taken to minimize loss of additive to the atmosphere in the form of dust. It is unified that the addition of hydrated lime to AC does increase stability and reduce the hardening rate of asphalts.

Hydrated lime is usually added to aggregate at the pugmill. It may serve as a filler in the aggregate material. With the addition of hydrated lime, upto 1% additional asphalt over the normal asphalt content can be used in the mixtures without ravelling or bleeding of the finished pavement. This produces a firmer, denser pavement with more durable surface [25].

Saleh A. Al-barrak [38] in his work at KFUPM, carried out an extensive research to find the most effective and economical treatment for water resistance of asphalt concrete and the effects of different antistripping agents in reducing the loss of stability. Based on the findings, he concluded that Hydrated lime was found to be most effective in improving the water resistance of asphalt concrete.

2. 3 .2 Hedmanite

2. 3 .2 .1 Introduction :

Hedmanite is the commercial name of a Rockwool kind of natural fiber, obtained by crushing the rocks called Lizardite. The lizardite rocks are extremely fine grained matrix material that commonly contains veins of chrysotile. It is the most abundant of the three principal forms of serpentine group of rocks in geology. All the three principal minerals of the serpentine group namely, chrysotile, lizardite and antigorite have the approximate composition $H_4Mg_3Si_2O_9$ and comparatively little substitution of other ions is found to occur in natural specimens. The most well known serpentine mineral, chrysotile, often occurs in veins of silky Fibers and is the most important source of commercial asbestos.

The structure of all serpentines is essentially a tri-octahedral analogue of the kaolinite structure. The name serpentine alludes to the appearance of many impure serpentine rocks, the surface pattern of which recalls the skin of a serpent [32].

CHEMISTRY :The chemistry of the serpentine group as a whole is relatively simple in that most natural specimens deviate little from the ideal composition $H_4Mg_3Si_2O_9$. The principal replacements which do occur are of silicon by aluminum, and of magnesium by aluminum. ferrous iron and ferric iron. The chemical relationships between the different serpentine varieties, chrysotile, lizardite and antigorite, are not fully understood. It is conceivable that these are purely polymorphic forms with identical chemical composition so that their existence would

be attributed to the different physical stability fields of the three structures. In lizardites, however, there is no evidence of either a tabular or corrugated structure, so that these might be expected to have high aluminum context. Again chemical analysis do not consistently support this expectation, some lizardites usually matrix material bearing chrysotile vines, have almost as little aluminum as chrysotile itself [32]. The mechanical strength combined with thermal stability and low thermal conductivity make it extremely useful in a wide range of important products like, Brake and Clutch linings [4].

Hedmanite lizardite mineral powder filler CAS No. 12161-84-1 is a talc related product having different desirable characteristics. Heated to 800° C, it has low dielectric constant and hence a low loss of material. Thus can control dust. It has no free silica and is nontoxic. It is found in abundance in Canada and Austria [4].

2. 3 .2 .2 Typical Chemical Analysis

Electron microscopic analysis of Hedmanite shows that 86% of the material is lizardite and 14% chrysotile with an average particle size of 2.5 microns. It has fibrous nature with a surface area of 14-15 m^2/gm. Technical data of a typical chemical analysis is shown in table 2.1 .

Table 2.1 : Hedmanite - Typical Chemical Analysis Data*

Constituent	Percentage %
Silica (SiO_2)	40.98
Ferrous Oxide (FeO)	2.05
Ferric Oxide (Fe_2O_3)	2.41
Alumina (Al_2O_3)	2.52
Lime (CaO)	0.30
Magnesia (MgO)	38.07
Manganese Oxide (MnO)	0.21
Chromic Oxide (Cr_2O_3)	0.44
Nickel Oxide (NiO)	0.24
Carbondioxide (CO_2)	0.22
Molecular H_2O @ 982° C	12.93
Moisture - Oven drying @ 205° C	0.50

*Source: "Hedman Resource Limited", Ontario,Canada. [4]

Weill H. in his report to World Health Organization on "Biological effects of Mineral fibers", indicated that fibers less than 5 or 8 microns are more easily cleared from the lung than longer fibers. He showed that chrysotile present in Hedmanite with an average particle size of 2.5 microns, seems to break down in lung tissue resulting in a relatively reduced level of fiber type pathogenicity [4].

Another report to the Royal Commission on matters of Health and Safety in Ontario, supports the evidence that Hedmanite is relatively less hazardous than other fiber material. And it can be cleared easily by the body fluids. [4]

2. 3 .2 .3 General Uses

Hedmanite has wide application in asphaltic adhesives, corrosion resistance coatings, roofing compounds, automotive undercoatings, pavements, sealants, cell putties, some cement products to improve crack resistance, friction material, texture and rust proof paints, primers, stains and wood preservers, grease, welding rod flux coating, refractory compounds and hot topping.

2. 3 .2 .4 Application as a Filler in AC Mix

According to "Hedman Resources Limited" research group, the incorporation of Hedmanite lizardite mineral filler in asphalt for paving of roads, airport runways,

parking lots, parking garages, tennis courts etc. reduces cost, increases the life and helps reduce the use of energy derivatives through the use of thinner layers of overlays. It is suggested from experience in Canada and Europe, that using 30mm Hedmanite-asphalt overlays is a good strategy [4].

It is reported that the use of Hedmanite permits thinner layers, of longer lasting pavement (less permeable to water penetration, better withstands freeze-thaw cycles and ultra violet rays, increases pavement stability, increases skid resistance and decreases cracking). The addition of Hedmanite increases dimensional stability, delays early maintenance and increases pavement life. When Hedmanite is used in asphalt paving, Air-voids are lowered considerably, resulting in a reduction of the hardening rate of asphalt bitumen, thereby extending the life of the pavement.

Hedmanite has significantly large viscosity building characteristics. It causes formulation to be very much shear rate dependent. If Hedmanite is mixed at a high shear rate, the viscosity remains low. But at a low shear rate, the viscosity goes up.

In Ontario, Canada using $1\frac{1}{2}$% of Hedmanite mineral filler in the mix for a Highway project showed that it was very easy to handle when applied on Highway [4].

To those who were convinced and gave talks on the virtues of asbestos fibers in the asphalt paving, but for health reasons were later discouraged from using it, this material is believed to provide a better fibrous substitute to be used in asphalt concrete as a filler. Thus, this material is selected in this research to evaluate its suitability as a filler in AC mixes.

2. 4 Mix Design and Evaluation Methods

The major properties to be incorporated in bituminous paving mixtures are stability, durability, flexibility and skid resistance (in case of wearing surface). The mix design methods are established to determine the optimum asphalt content that would perform satisfactorily, particularly with respect to stability and durability.

Stability as defined by many engineers is the " resistance to deformation" with an implied emphasis towards resistance to flow or rutting, including resistance to tensile, compressive, and shear stresses that causes failure in a pavement surface. While durability has been defined as the resistance to the effects of weather and its combination with other forces. Durability is enhanced with high asphalt content, however, resistance to flow or deformation is impaired with high asphalt content. As a consequence, the amount of asphalt to be used in a paving mixture must be in a balance to optimize durability but yet maintain adequate stability [33].

There are many mix design methods used throughout the world e.g. Marshall mix design method , Hubbard-field mix design method, Hveem mix design method, Asphalt Institute Triaxial method of mix design etc. Out of these Marshall mix design method used in this research will be discussed in detail.

2. 4 .1 Marshall Mix Design Method

The Marshall procedure as applied to design and control of asphalt mixtures used in the U.S Army Corps of Engineers, was evolved during the period from World

War II to late 1950's. Motivation for its development came from the need for a mix design procedure to proportion aggregate and asphalt border to sustain increasing wheel load and tire pressure of Military Aircraft during World War II. In order to achieve these needs, Corps began an investigation to select a test apparatus that was simple and easily portable and could be used in the field for control purposes. The second phase of this study was to determine the method of compacting laboratory specimen in order to achieve the density as that obtained in field. The third phase of this investigation was the establishment of satisfactory design criteria and control procedure [34].

The Corps of Engineers selected a testing machine and a method of pavement mixture design conceived by Bruce Marshall of Mississippi State Highway Department. In order to determine laboratory method of compaction for specimens and to establish criteria on certain mixture properties as evaluated by Marshall testing device, it became necessary to construct a large scale test track, that incorporated such variables such as asphalt content and gradation of aggregates. Loaded trailers were pulled over this test track for number of times, so that the effect of compaction due to traffic loads could be determined. From this study, the U.S Corps of Engineers through extensive research and correlation studies, improved and added certain features to Marshall's test procedure, and ultimately developed a mix design criteria. It was adopted by MOC with some modifications and is shown in Table 2.2. The Marshall test procedure have been standardized by the American Society for Testing and Materials. Procedures are given in detail by ASTM designation D-1559 " Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus".

The use of these criteria must be limited to hot mix asphalt paving mixtures using penetration grades of asphalt cement and containing aggregate size of 1 inch or less. The Corps of Engineers found that, in order to have the proper balance between durability and stability, the air voids in the total mix should be limited to between 3 and 5 percent. The voids in the aggregate mass filled with asphalt should be limited to between 75 and 85 percent. The local MOC standards requires the air voids to be between 4 and 7 percent for wearing coarse and between 5 and 8 percent for base coarse.

Since its development in 1940's the Marshall method has increasingly been accepted by highway agencies throughout the world to design and control bituminous paving mixtures. The general acceptance of this procedure appears to be based on the simplicity and its good portability for field control of paving mixtures. A review of literature indicates that Marshall stability value is a measure of tensile strength. Smith V. R [42], wrote in a discussion that the Marshall stability values are affected primarily " by the tensile strength or cohesion properties of a mixture". Others such as Benson [43] , found a linear relationship between Marshall stability and cohesiometer value. It would seem to be apparent that the Marshall test does give a measure of tensile strength and that the methods success in preventing shear deformation (rutting) failure come from the control of aggregate texture and gradation, asphalt content, and compaction.

Table 2.2 : Marshall Mix Design Criteria (MOC Specifications)

Marshall Mix Criteria	Light Traffic		Medium Traffic		Heavy Traffic	
	Surface & Base		Surface & Base		Surface & Base	
	Min	Max	Min	Max	Min	Max
Compaction (No. of blows on each end)	35		50		75	
Stability , N	2224	-	3336	-	6672	-
(lb)	(500)	-	(750)	-	(1500)	-
Flow , 0.25 mm	2	8	2	6	2	4
Percent Air Viods	4	8	4	8	4	8
Percent VMA	Varies according to gradation					

During the past few years, other supplementary tests such as indirect tensile test, creep test etc. have been used to evaluate the engineering properties of asphalt mixtures.

2. 4 .2 Indirect Tensile Test

The indirect tensile test is one type of tensile strength test used for stabilized materials. Most of the reported test results have been for concrete or mortar; however, the test has been conducted on cement-treated gravel, lime-soil mixtures, and asphalt - stabilized materials [35].

The indirect tensile test can be used to characterize asphalt materials in terms of [46]

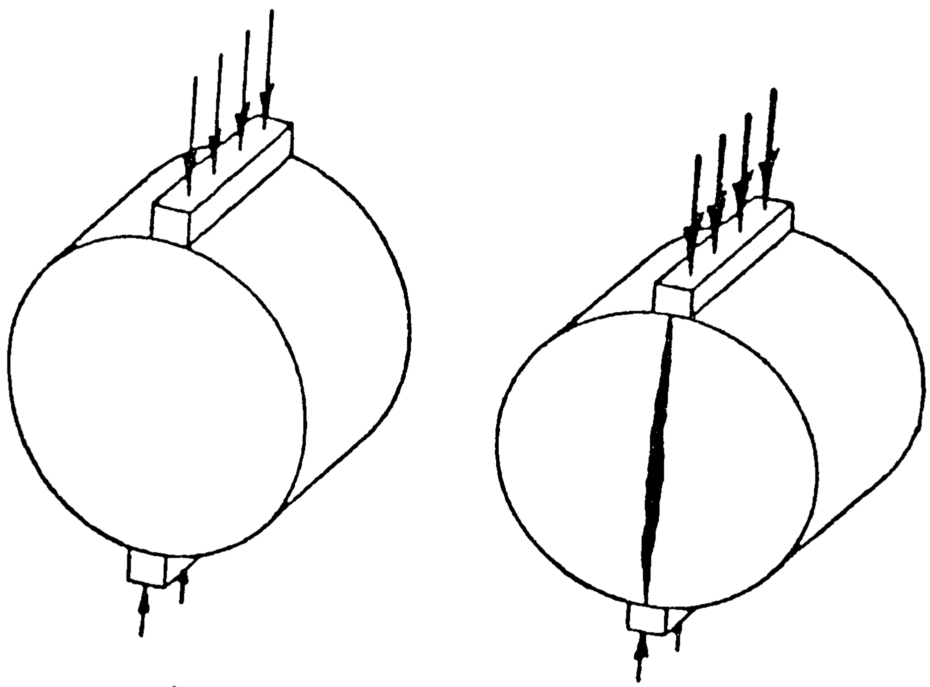
a) resilient elastic properties,

b) properties related to thermal cracking,

c) properties related to fatigue cracking, and

d) properties related to permanent deformation.

In addition to above, the test is simple and economical to conduct. The test is done by loading a cylindrical specimen with a single or repeated compressive load which acts parallel to and along vertical diametrical plane (Fig: 2.1). This loading configuration develops a relatively uniform tensile stress perpendicular to the direction of applied



(a)

(b)

Figure 2.1 : Loading Configuration and Failure of Indirect Tensile Test

load and along the vertical diametrical plane, which ultimately causes specimen to fail by splitting along vertical diameter . The development of stresses within cylindrical specimen subjected to load is reported by Kennedy and Hudson [45].

Turpienen et. al [34], suggested that the Marshall stability method should be replaced with more sensitive testing method. Both resilient modulus method and split tensile test are useful in a more sensitive estimation of deformation, low cost alternative to Marshall method.

This is a simple test especially when used in static mode and therefore, it can easily be augmented with existing mix design tests such as Marshall test. The effect of temperature can be evaluated by conducting indirect tensile test at different temperatures [46].

The equation employed in calculating the tensile strength is:

$$\sigma_T = 2P_{max} / \pi h D \qquad \text{...} \quad (1)$$

Where: σ_T = Indirect/split tensile strength

P_{max} = Load at failure, lbs

D = Diameter of sample (4 inches)

h = Sample thickness, inches

2. 4 .3 Resilient modulus Test

The elastic modulus of asphalt treated material can be determined by means of the diametral resilient modulus (M_R) device. This test is basically a repetitive load test using the stress distribution principles of the indirect tensile test previously discussed. Like the nonrepetitive indirect tensile test , the main advantage of this test procedure is the simplicity of the test equipment as well as the ability to test asphalt specimens similar in size to those used for the widely known Marshall and Hveem tests.

In the procedure a repetitive (pulsating load) of 0.1 second duration and 0.9 second dwell time is applied diametrically to the sample. The dynamic load, in turn, results in dynamic deformations across the horizontal diametrical plane. These deformations are recorded by transducers mounted on each side of the horizontal specimen axis. Knowledge of the dynamic load and deformation allows the M_R value to be calculated.

Thus, for an applied dynamic load of ' P ' in which the resulting horizontal dynamic deformation (δ_h) is measured, the modulus, the modulus or M_R value is given by [35].

$$M_R = \frac{P(\mu + 0.2734)}{t\delta_h} \qquad \text{..............................} \quad (2)$$

A commonly used value of Poisson's ratio (μ) for asphaltic materials is 0.35.

2. 4 .4 Creep Test

Shell researchers have developed a pavement design system in which rutting potential of asphalt concrete is characterized by a simple 'Creep test' [47,48]. This has led to the establishment of an empirical link between rheological properties of asphalt cement and viscoelastoplastic behavior of asphalt concrete.

This test has been designed for the following purposes: [49]

1. To measure compressive stiffness or compliance properties of the mixture.
2. To establish plastic flow potential of HMA under various stress states in terms of viscoplastic strains.

Van der Poel [50] through his research, indicated that static and dynamic test measurements has provided similar stiffness trends; hence, the static creep was viewed to be an adequate test for establishing stiffness trends. Thus the creep deformation of a cylindrical specimen under a uniaxial, static compressive load is measured as a function of time , for a mix with a conventional binder, the static creep data can be used to predict the permanent deformation under different traffic loading and temperatures. In this test, a constant stress (σ_o) is applied to the specimen and the resulting time dependent strain (ε_t) is measured. For permanent deformation characterization the relevant quantity is the stiffness modulus of the mix S_{mix} defined as [34].

$$S_{mix} = \frac{\sigma_o}{\varepsilon_t} \qquad \text{...} \quad (3)$$

where : σ_o = Applied Stress

ε_t = Measured strain at time $t = \frac{\Delta h}{h_o}$

where : Δh = Change in height of specimen

h_o = Original height of the specimen

Shell investigators [47], have argued that current method of designing mix composition for asphalt paving application are based on recipes of empirical tests that are specific to one type of mix. While they have the merit of being based on experience, these methods are not always certain of success and are difficult to extend to the use of new materials or more severe performance requirements because the test results are not directly related to performance. There is therefore , a need for laboratory test methods that allow the mechanical properties of an asphalt mix to be characterized in such a way that is possible to predict the depth of rut that will occur when this mix is used in pavement of given construction and subjected to specific loading and climatic conditions. Thus, these investigators have devoted considerable effort to the development of creep test .

The Shell investigators have also developed a procedure for predicting rutting in the field using creep test. However as Bolk has noted [48], it has been necessary to modify the values obtained from laboratory creep test by a factor to reflect the so-called dynamic effect of repeated loading.

Some investigators have developed limiting values of creep test moduli to be used in conjunction with mix design by Marshall test to insure that mixes so designed are suitable. One such group that has used such an approach is the NITRR of South Africa [49]. Based on preliminary studies, these investigators recommend a minimum creep modulus of 80 Mpa (120,000 psi) at 40 °C and stress of 200 kpa (30 psi) for conditions of heavy, slow moving traffic.

2. 4 .5 Fatigue and Permanent Deformation

Fatigue is the phenomena of repetitive load-induced cracking due to a repeated stress or strain level below the ultimate strength of the material. Fatigue tests may be conducted by several test methods and various specimens. Repeated load indirect tensile (split tensile) test have also been used. Recent work at Ohio State University has been based upon fracture mechanic principles applied to a more mechanistic solution of the fatigue problem. [35]

A common method for evaluating the fatigue characteristics of the asphalt concrete is by repeated flexural testing. In this testing a repeated load is applied to the specimen which is normally a haversine wave, with a certain adjusted loading and unloading (rest) time. Because of the effect of varying stiffness upon AC fatigue tests, a temperature control system should be used around the flexural load device. The range in stress level should be selected so as to yield a range in fatigue life between 1000 to 1,000,000 repetitions.

Fatigue testing may be conducted under two types of controlled loading. They are either (a) Controlled stress or (b) Controlled strain. In the controlled stress mode a constant load is continuously applied to the specimen. Because of progressive damage to the specimen, a decrease in stiffness results. This, in turn, causes an increase of the actual flexural strain with load applications. For the controlled strain (deflection) approach, the load is continuously changed to yield a constant beam deflection. This results in a stress that continuously decreases with load applications. However, since controlled stress conditions give more conservative estimate of the fatigue life and is easy to apply, this test may be safely employed.

For controlled stress testing, conducted in the laboratory, the effect of stiffness may be accounted for by plotting the fatigue results in a log strain applied (ε) versus log N_f relationship. This results in a relationship for fatigue tests of the form.

$$N_f = K\left(\frac{1}{\varepsilon}\right)^C \quad \text{.................................... (4)}$$

Where K and C are regression constants obtained from an analysis of fatigue data. The fatigue test applied in this research will be discussed later.

Permanent deformation: Permanent deformation is a longitudinal depression that forms in wheel track due to consolidation and/or movement in one or more of the

pavement layers due to repeated traffic load applications. The depressions or ruts are of concern for at least two reasons: [51]

* If the surface is impervious, the ruts trap water and at depths of about 0.2 in., hydroplaning (particularly for passenger cars) is a definite threat.

* As the ruts progress in depth, steering becomes increasingly difficult, leading to added safety concerns.

For pavements in moderate or hot climates and subjected to large number of heavy vehicles and/or vehicles operating at high tire pressures, rutting can be a controlling factor in mix design. Relative to mix design, two methods are generally used by the highway authorities to select the proper amount of binder, one is based on the Marshall test and the other on the Hveem stabilometer.

Noticeable rutting problems have appeared on Saudi road surfaces in the last 12 years. The exceptional growth rates in truck numbers and weights, high tire pressures, together with the local harsh climatic condition present a uniquely Saudi Arabian problem. Several studies were undertaken using a scientific approach to avoid the spread of rutting problem. for example, National Research Project for evaluation of permanent deformation in asphalt concrete pavements.

Ziauddin A. Khan [34], in his research at KFUPM on evaluation of local asphalt concrete mix design procedures, concluded that the Hveem method, seems to be a potential mix design method which can find application for Kingdoms roads, Since it can identify mixes with high rutting susceptibility.

Ramadhan R. H [40], through his work on prediction of pavement rutting found VESYS to be the best suitable model for rut depth and pavement performance prediction for Kingdoms roads.

In its most general form, relationships between permanent (plastic) strain, applied stress, and load repetitions for each of the pavement components are required. At a particular number of load repetitions the relationships can be stated as

$$\varepsilon_p = f(\sigma_{ij}) \quad \text{.......................................} \quad (5)$$

For a particular layer it is then possible to estimate the permanent deformation occuring in that layer.

CHAPTER 3

MATERIAL CHARACTERIZATION

AND MIX DESIGN

3. 1 Introduction

This Chapter lists the materials collected and details the laboratory procedures adopted for material characterization and design of mixtures by Marshall mix design procedure. Material characterization consists of evaluation of engineering properties of pavement component materials i.e., asphalt and aggregate while the laboratory mix design include determination of optimum asphalt content for both wearing coarse and base coarse gradation by Marshall mix design procedure. The sequence of testing is shown in Fig. 3.1

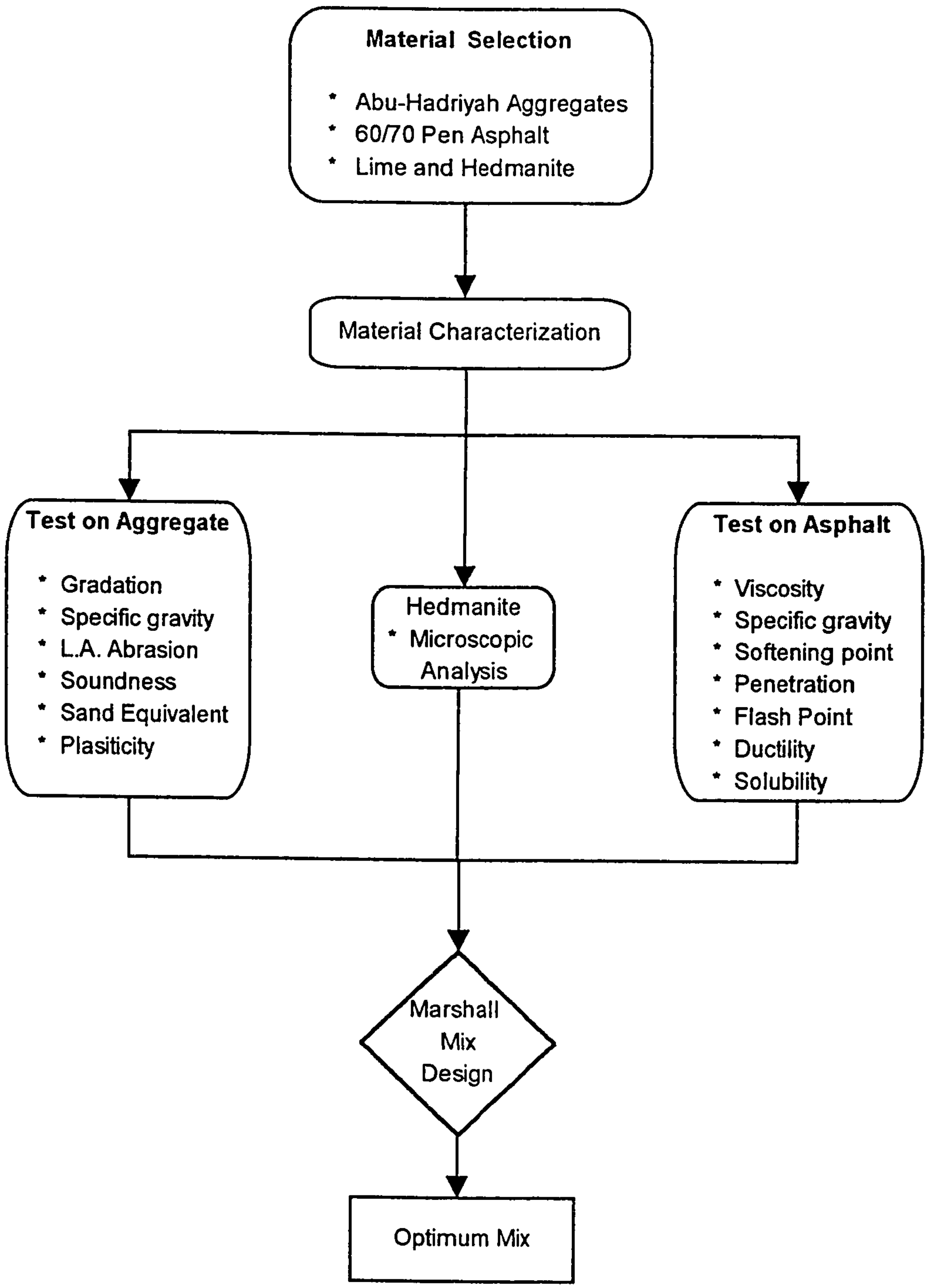


Figure 3.1 : Flow Diagram for Material Testing and Mix Design

3. 2 Material Selection

3. 2 .1 Aggregate

As noted from literature (38), that Abu-Hadriyah aggregate is the best quality aggregate among locally available material in Eastern province of the Kingdom. Hence Abu-Hadriyah aggregate is selected for the present study and is obtained from Al-Khodari crusher plant. Crushed stone fines, which are by product of aggregate crushing was also collected to be used as filler from the same source.

In order to produce a controlled gradation, aggregates were separated in different sieve sizes and then recombined to get the required gradations. In the design gradation for BC, passing 1" size aggregate is made 100%. Since size higher than one inch cannot be used for 2.5" height specimens. Two design gradations, one for wearing coarse (WC) and another for Base Coarse (BC) were selected according to the modified MOC Specification [36], which are shown in Table 3.1. The specifications are modified to be coarser than the old gradation in order to control the rutting problem. In the design gradation for BC, the aggregate passing 1 inch size is taken as 100 percent. Since, one inch and higher size aggregates cannot be used for specimens having 2.5 inch height.

The aggregates were subjected to further testing as per ASTM standard test methods to evaluate other physical properties which are of significance for HMA concrete. The tests include Los Angeles abrasion test, Water absorption test, Sand Equivalent, plasticity, and specific gravity test for coarse and fine aggregates. The test results

Table 3.1: Design Gradation for the Mixtures:

Sieve	Wearing Coarse*	Base coarse*
11/2" (37.5 mm)	-	100
1" (25.0 mm)	-	100
3/4" (19.0 mm)	100	75
1/2" (12.5 mm)	84	65
3/8" (9.5 mm)	71.5	55
# 4 (4.75 mm)	52.5	41
# 8 (2.0 mm)	30	-
# 10 (2.0 mm)	-	28
# 40 (0.425 mm)	13.5	14
# 80 (0.180 mm)	9.0	10.5
# 200 (0.075 mm)	5.5	5.5

" - " Indicates the size is not included.

* Percent passing as per modified MOC Specs, 1986. [36]

Table 3.2 : Results of Quality Tests on Aggregate.

TEST	MIX TYPE				MOC Specifications
	Wearing Coarse(G1)		Base Coarse (G2)		
L.A. Abrasion , % (ASTM C - 131)	30.4		31.5		40 Maximum
Specific Gravity	CA^{+}	FA^{++}	CA^{+}	FA^{++}	
(ASTM C -127) Bulk	2.600	2.440	2.599	2.422	-
(C-128) Apparent	2.663	2.707	2.663	2.709	
Absorption, %	2.628	3.80	2.620	3.600	4 Maximum
Soundness, % Loss (ASTM C-88)	3.72	2.98	4.28	3.12	10 Maximum
Apparent Specific Gravity of Filler (ASTM C - 128)	2.727		2.727		-
Plasticity Index (AASHTO T-90)	Non-plastic				3 Maximum
Clay lumps and friable particles, % (ASTM C-142)	0.00				1 Maximum
Sand Equivalent (ASTM D-2419)	50		48		45 Minimum

CA^{+} Indicates Coarse Aggregate.

FA^{++} Indicates Fine Aggregate.

together with specification limits from ministry of communication (MOC) are summarized in Table 3.2 . It is found that the aggregates has 30.5 percent wearing in L.A abrasion test, Sand equivalent value of 50, and an average absorption percentage as 3.1. These results are in agreement with MOC specification limits for hot mix asphalt concrete for both wearing coarse and base coarse.

3. 2 .2 Asphalt

Asphalt cement of grade 60/70 pen utilized in this research was obtained from Saudi - Aramco Ras-tannurah refinery. The main reason of using this grade is its wide use in all road projects in the kingdom.

A series of ASTM tests including penetration, specific gravity, softening point, viscosity, Thin film oven (TFO) test, flash point, ductility, and solubility in tri-chloroethylene were conducted for the identification of basic physical properties of asphalt used in this research. The results obtained were listed in Table 3.3 , along with ASTM and MOC Specifications. The properties measured indicates that the asphalt has a penetration value of 62 dmm, specific gravity as 1.017, softening point 52 ° C , and kinematic viscosity of 480 Cst. It meets the required ASTM as well as MOC specification.

3. 2 .3 Filler

As indicated in earlier chapters , this research was initiated to study the effect of 'Hedmanite' as a filler in AC mixtures. And to compare it with lime and conventional crushed stone filler. Hence, the three fillers used in this study are:

Table 3.3 : Physical Properties of Asphalt Cement.

Physical Properties & Test Designation	Test Results	ASTM Limits	MOC Limits
Fresh Sample			
Specific Gravity, @25°C (ASTM D-70)	1.017	-	-
Penetration dmm. @ 25°C (ASTM D-5)	62	60-70	60-70
Kinematic Viscosity in Cst @ 135°C (AASHTO T-202)	480	-	-
Absolute Viscosity in Poise @ 60°C (AASHTO T-202)	3980	-	-
Softening Point in ° C (ASTM D-36)	52	49-54°C	49-54°C
Flash Point, Cleavand Open Cup, °C (ASTM D-92)	307	232 Min.	232.2 Min.
Ductility, 25 ° C ASTM D-113	150+	-	100 Minimum
Solubility in Tri-chloro Ethylene (ASTM D-2040)	99.8	99.8 Minimum	99.5 Minimum
TFO Sample			
Percent loss (TFO) (ASTM D-1754)	0.0571	0.1 Maximum	0.1 Maximum
Penetration dmm @ 25°C @ 4°C (ASTM D-5)	 46 23	-	-
Softening Point in °C (ASTM D-36)	56	-	-

1) Crushed Stone fillers.

2) Hydrated Lime.

3) Hedmanite.

Crushed stone (passing # 200), obtained by sieving Abu-Hadriyah aggregates, was tested for specific gravity and plasticity. The apparent specific gravity of the filler is obtained as per ASTM C - 128 procedure and using 500 cu.cm pycnometer flask is 2.727. Determination of the plasticity index of the filler showed that the filler is non-plastic.

Hydrated lime or Hedmanite was added to the mix at four different percentages based on the weight of the total aggregate. The weight of the filler was reduced by the amount of lime or hedmanite used. Hydrated Lime was collected from a Saudi lime brick & building materials Co. Riyadh. While Hedmanite sample was obtained from Contemporary Trading Establishment, Al - Khobar (local agent for Hedmanite).

Microscopic analysis of Hedmanite was carried out at the Central Analytical and Materials Characterization Laboratories at the Research Institute, KFUPM . The objective of this analysis was to observe the microstructure and determine the chemical and physical composition. Chemical analysis shows that it is a hydrous magnesium silicate. The identity and physical data is listed in Table 3.4 . Micrographs were taken at 1000X , 5000X and 10,000X magnifications, with each revealing the microscopic structure of the sample, and are as shown in Figure. 3.2 and Figure. 3.3 . These micrographs show the fibrous nature exhibited by the particles. It is

Table 3.4 : Hedmanite Identity and Physical Data.

Identity and Physical Data	
Appearance	Fine Powder, White Blueish.
Chemical Family	Lizardite (Serpentine rocks group)
Molecular Formula	H_4O_4Si 1/2 H_2O 3/2 Mg
Bulk Density: ml/100g - Dry	310
Charge	Electropositive
PH	9.4
Specific Gravity	1.87
Surface Area	14-15 m^2/gram

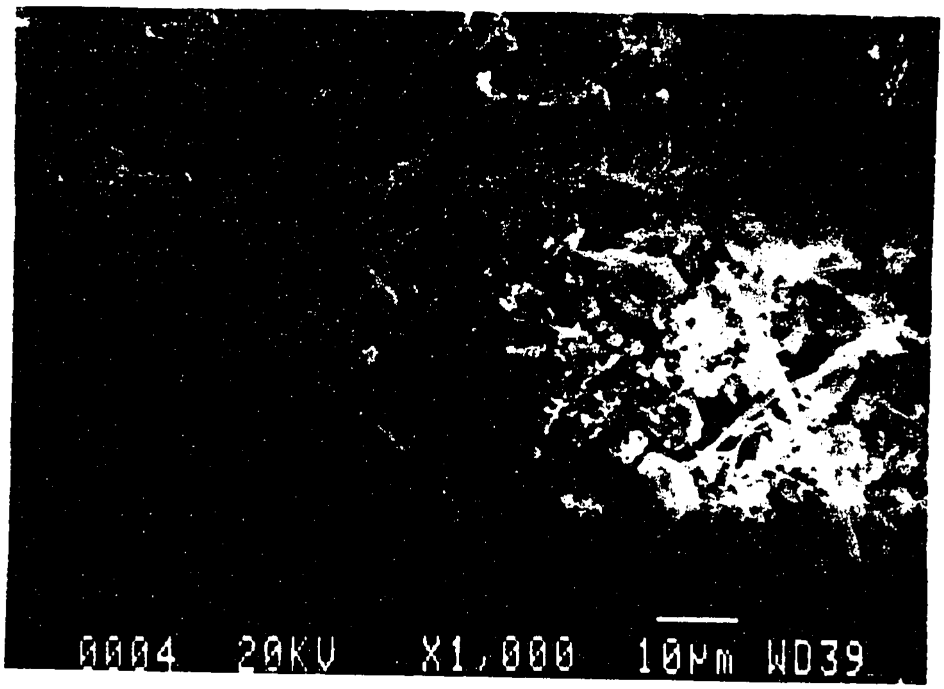


Figure 3.2 : Hedmanite Sample Micrograph at 1000X Magnification.

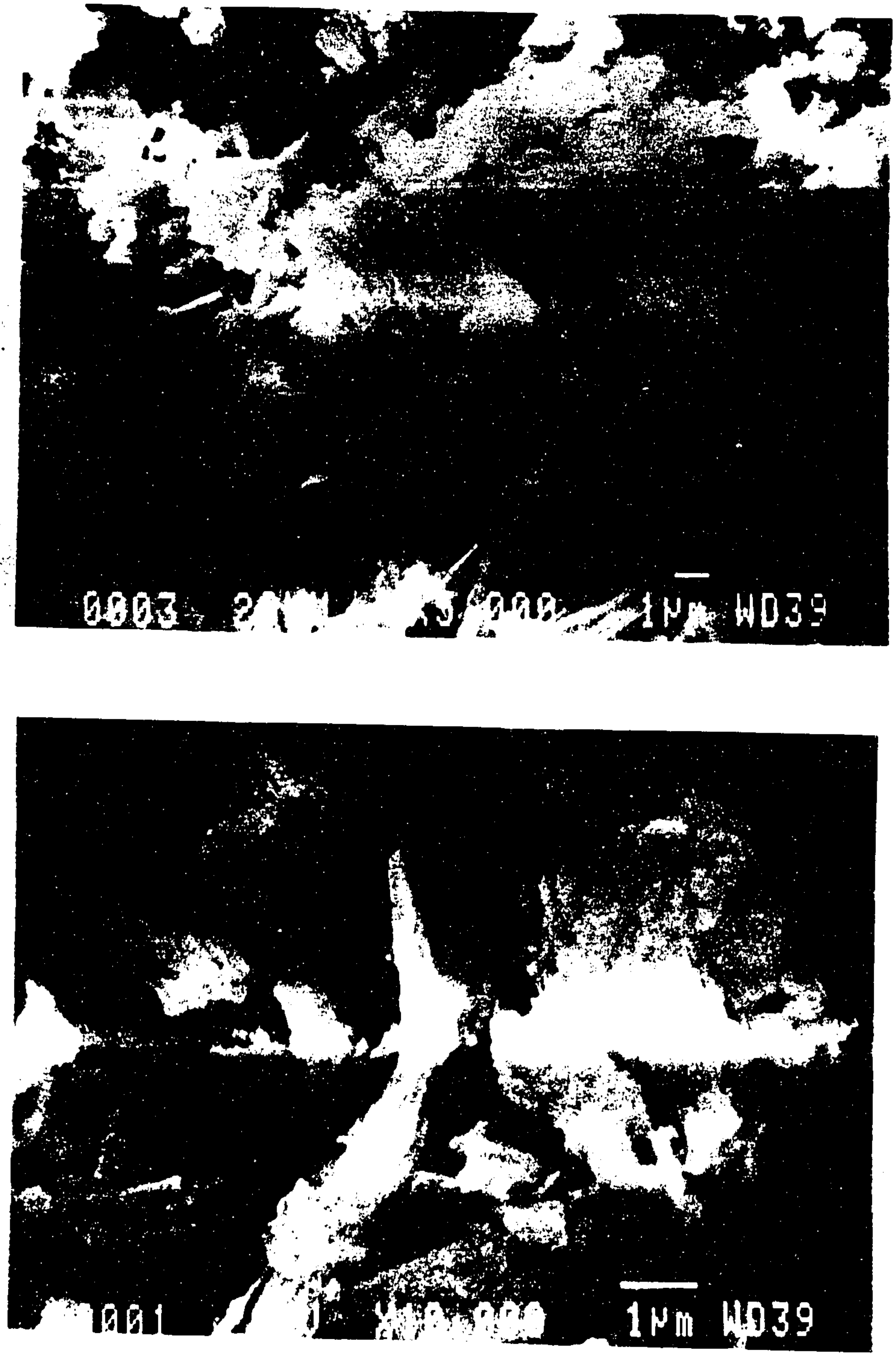


Figure 3.3 : Hedmanite Sample Micrographs at 5000X and 10000X Magnification.

anticipated that these fibrous particles may provide a better interlocking between the grains in the asphalt concrete mixture , and thus can result in better stability of the mixes.

Elemental composition were determined with the EDS (Electron diffraction spectrometry) and the relative weight percentages were calculated using the standardless semi - quantitative analysis program. The analysis report is attached in the Appendix-A.

3. 3 Mix Design

The optimum design of asphalt paving mixes is one that best satisfies a set of desirable mix properties at optimum construction and maintenance costs. These properties can be summarized as follows: [39]

1. Stability to meet traffic demands without distortion or displacement.
2. Skid resistance to meet the need for traffic safety, particularly under wet condition.
3. Fatigue and rutting resistance with longer life under repetitive traffic loading conditions without cracking or permanent deformation.
4. Durable mix resistant to climate without wear or cracks; yielding better riding conditions and lower maintenance costs.

5. Mix with sufficient voids to allow additional compaction under traffic loading without flushing or asphalt bleeding and loss of stability yet low enough to keep out harmful air and moisture.

6. Sufficiently workable to allow efficient placement without segregation.

3. 3 .1 Marshall Mix Design

3. 3 .1 .1 Preparation of Test Specimens.

Each aggregate sample (1200 gram) was blended for each specimen separately according to mix design formula. Aggregates were placed in an oven at a temperature of 160°C for eight hours to ensure hot and dry aggregate samples.

Asphalt was heated upto 140°C prior to mixing. Pre-heated asphalt were avoided, in order to achieve consistent results. Cox and Sons Self Heating Mixer was used for mixing aggregate and asphalt. This mixer is shown in Figure. 3.4. Asphalt is added to the hot aggregate in the bowl, and it is placed in position on the mixer and is allowed to mix for two minutes. Standard Marshall moulds, 4 inch (10 cm) diameter, 3 inch (7.5 cm) high, were heated in an oven upto 140°C. The aggregate, when thoroughly mixed with asphalt was placed in the mold and compacted with 75 blows on each face of the specimen , using mechanical compactor Figure. 3.5.

Test specimens were fabricated for a range of asphalt contents (3 to 6%) for wearing coarse and base coarse gradation. The crushed stone powder (passing # 200) is used

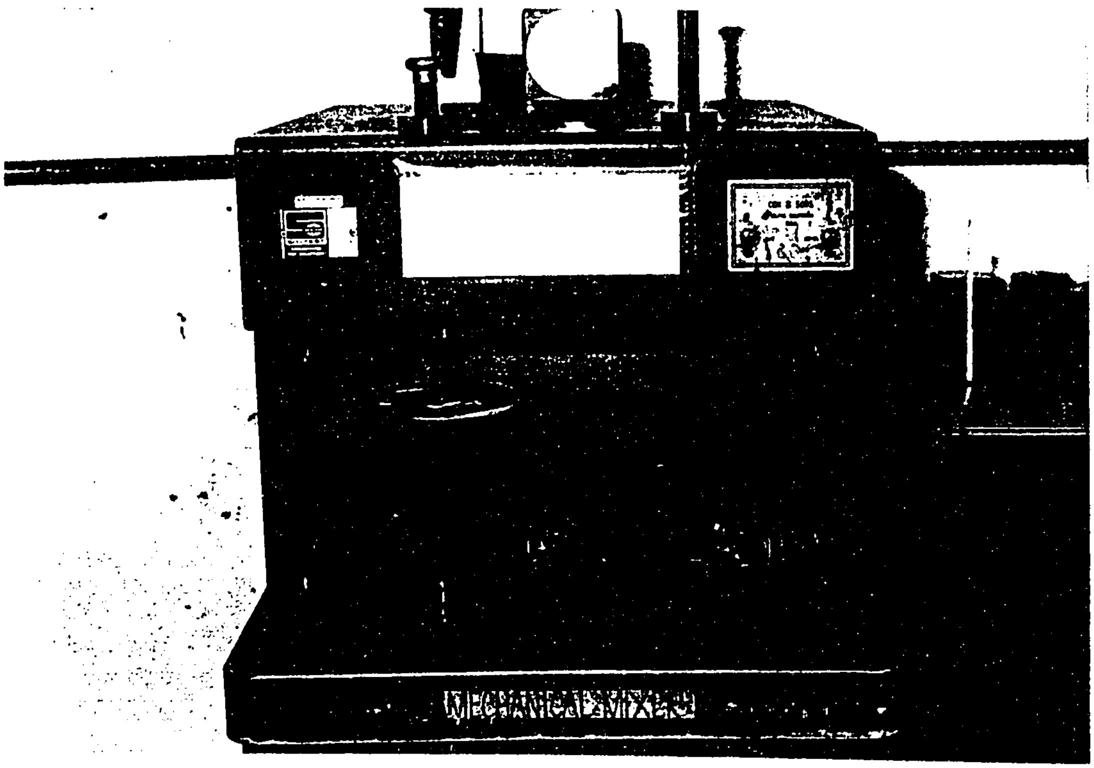


Figure 3.4 : Cox and Son's Self Heating Mixer.

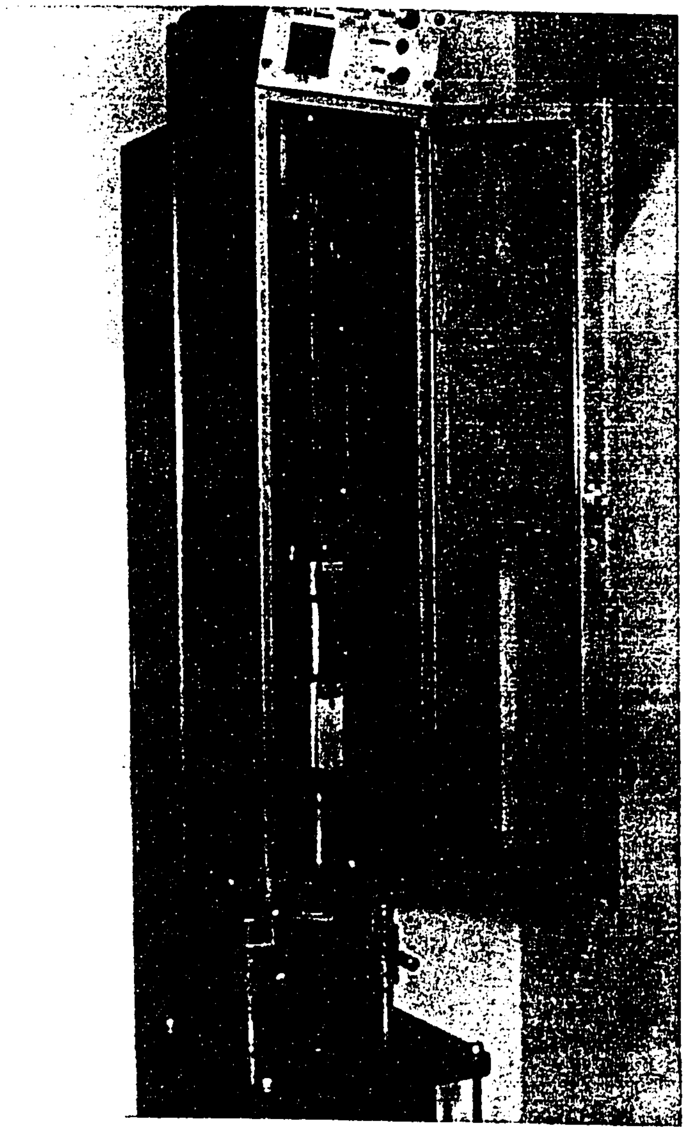


Figure 3.5 : Marshall Mechanical Compactor.

as filler material. Compacted specimens were left to cool down for atleast four hours before extrusion. Specimens were left to cure at room temperature for 24 hrs before testing.

3. 3 .1 .2 Test Procedure

Marshall test was conducted on a cylindrical specimen 4 inch (10 cm) diameter by 2.5 inch (6 cm) high. Prior to stability test, all specimens were weighed in air and submerged in water. From this information the bulk specific gravity of specimen was calculated as described in ASTM D2726. The specimens were then placed in a 60°C (140°F) water bath for 30 minutes. Upon removal from the water bath, the specimen was placed on its side in the breaking head of the Marshall Stability and Flow Apparatus (Fig. 3.6) and a load was continuously applied on the outer circumference of the specimen at a rate of 2 inch (5 cm) per minute until failure. The maximum load in Kilograms was recorded as the stability value. The deformation undergone by the specimen (in 0.01 inch) during loading to maximum value was measured by the flowmeter (Fig.3.6) and was reported as flow value.

Percentages of air voids in the specimens were determined from bulk specific gravity of the specimens (ASTM D2726) and the maximum theoretical specific gravity of the voidless mix (ASTM D2041) . Stability loss, after 24 hours immersion in water at 60°C (140°F), was also determined to check the resistance to stripping which was estimated on the basis of Marshall strength index calculated by dividing the stability of the specimen conditioned in water for 24 hours by normal 1/2 hour stability.

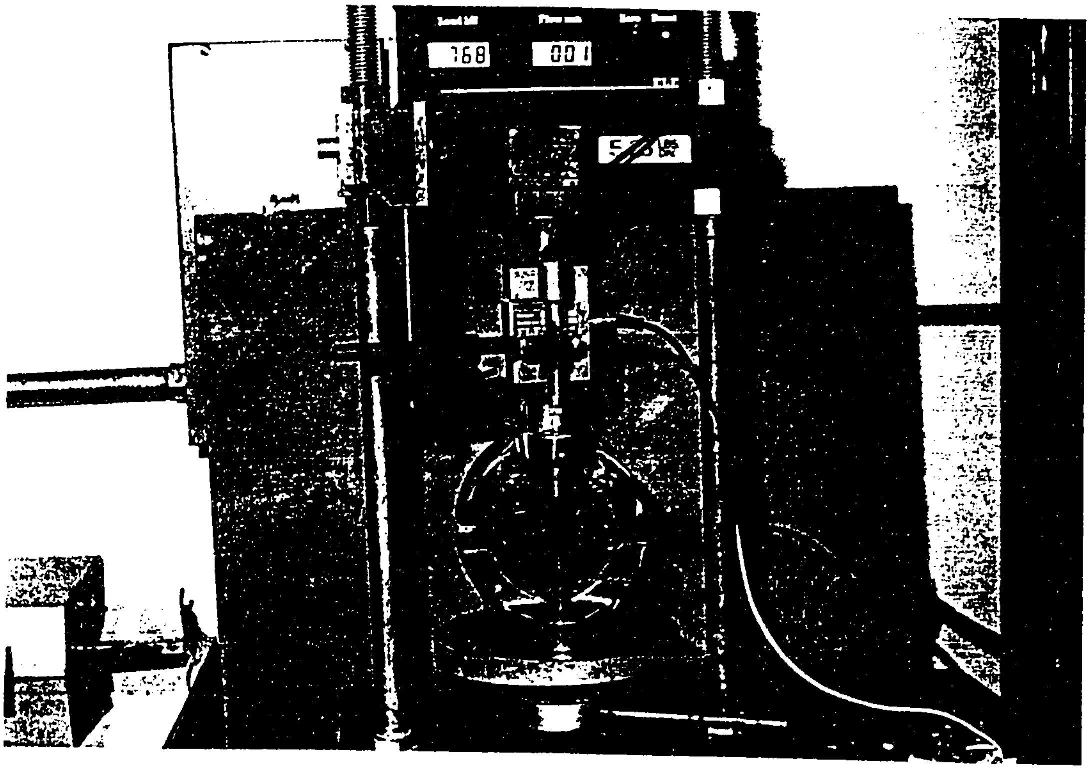


Figure 3.6 : Marshall Stability and Flow Apparatus.

3. 3 .1 .3 Results

The results of each test were plotted as percent of asphalt (by total weight of the mix) on a linear scale , the plots are presented in Figure. 3.7 & 3.8, for wearing coarse & base coarse respectively. Each point shown on the plot is an average of triplicate test specimens. Asphalt contents were determined corresponding to the following:

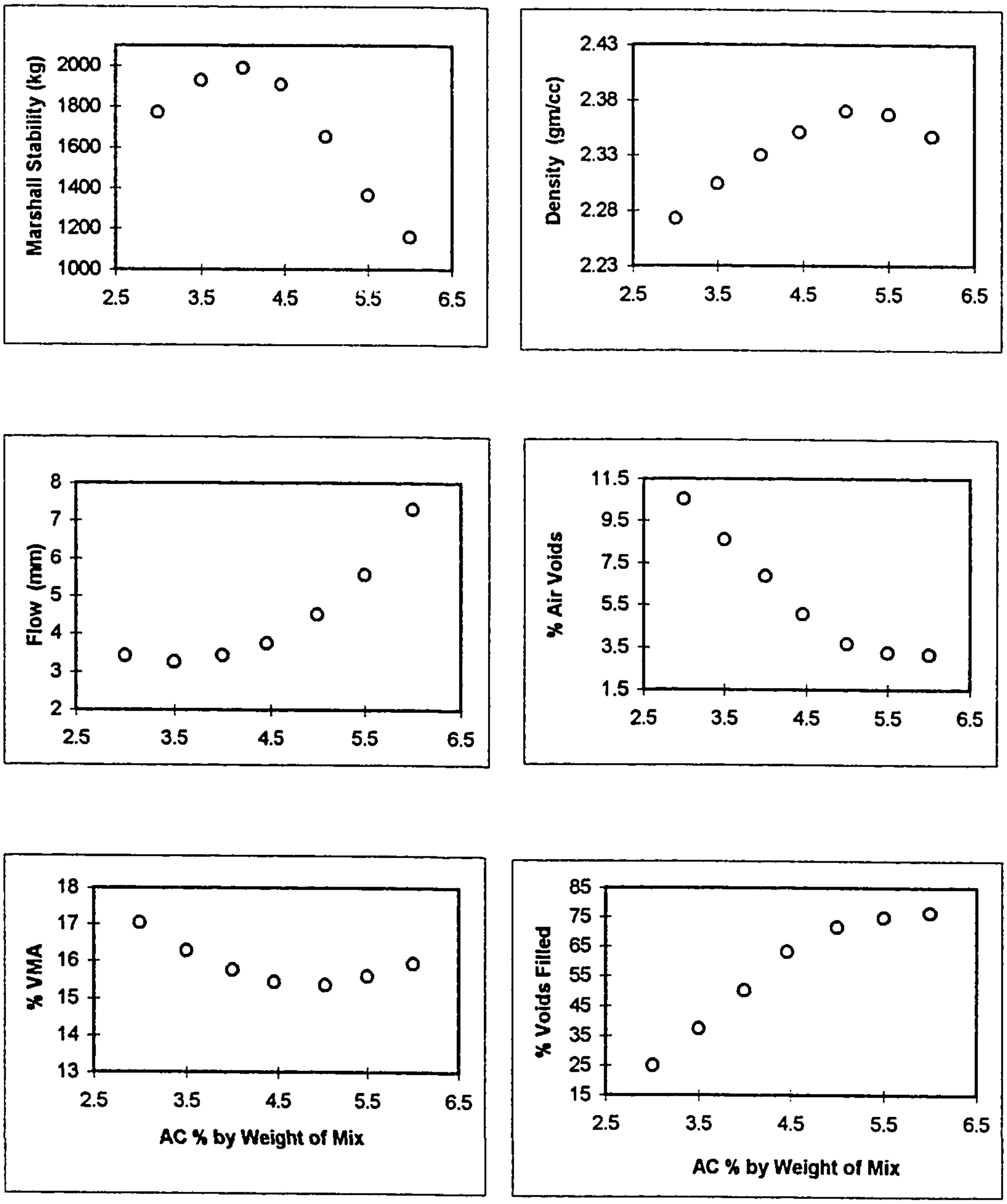
(a) Maximum Stability.

(b) Maximum Density.

(c) 5% Air Voids.

The optimum asphalt content of the mix was then calculated as the numerical average of the values of the asphalt contents determined as noted above. It was found that the optimum asphalt content required is 4.5% for Wearing coarse and 3.9 % for Base coarse.

The Marshall properties were then determined at optimum asphalt percentage for wearing coarse and base coarse mix using the curves shown in Figure. 3.7 and Figure. 3.8. Marshall mix properties evaluated at optimum asphalt contents are summarized in Table. 3.5, along with MOC specifications limits for wearing coarse and base coarse . At the optimum asphalt contents, marshall stability values obtained are 1900 and 1550 (kg) for wearing coarse and base coarse respectively, which are well above the specified minimum value of 1000 kg. The value of percentage air voids obtained as 5.3 and 5.7 for wearing and base coarse, are within the limits of 4 to 8 percent given by MOC standards.



'Figure 3.7 : Marshall Mix Design Curves for Wearing Coarse (G1)

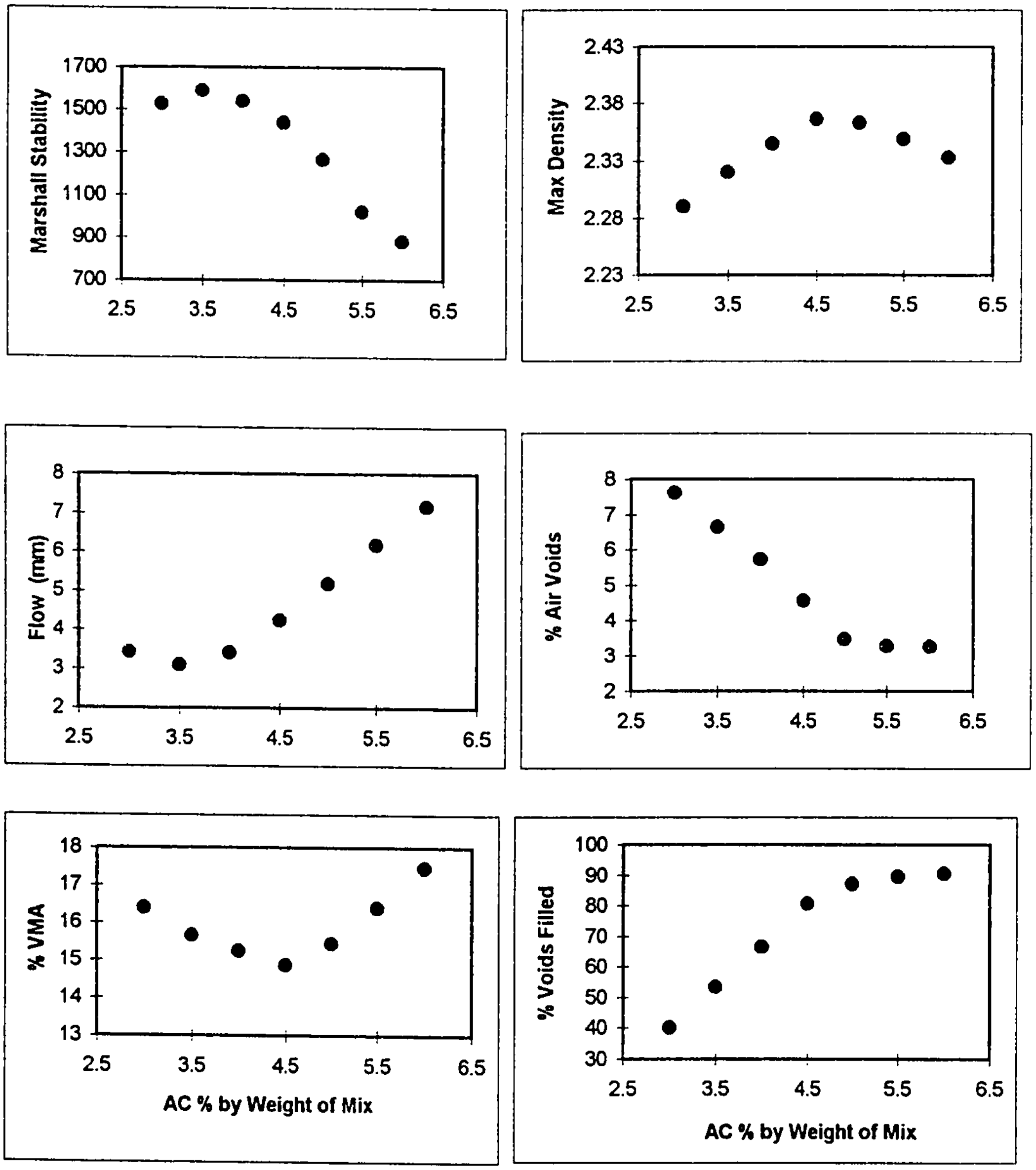


Figure 3.8 : Marshall Mix Design Curves for Base Coarse Mix (G2)

Table 3.5: Marshall Properties for Wearing Coarse and Base Coarse Mixes.

Mix design criteria	Wearing coarse (G1)	MOC Specification*	Base coarse (G2)	MOC Specification*
Optimum Asphalt content, %	4.5	-	3.9	-
Marshall stability, Kg	1900	1000 Minimum	1550	1000 Minimum
Air voids, %	5.31	4 -- 7	5.7	5 -- 8
Voids filled with Asphalt, %	65	-	60	-
VMA, %	15.44	15 Minimum	15.2	13 Minimum
Flow (0.25mm)	3.8	2 -- 4	3.4	2 -- 3.5
Marshall Density (gm/cc)	2.351	-	2.343	-
Rigidity ratio: Stability / Flow	500	-	456	-

* Reference [36].

While the value of flow noted as 3.8 and 3.4 mm, also satisfies the required 2 to 4 (mm) criteria.

3. 4 Summary

After studying the test results of Marshall mix design method for both wearing coarse and base coarse mixes. It is found that for the selected aggregate gradation, the optimum asphalt content required for wearing coarse is 4.5 % and for base coarse is 3.9 %. These optimum asphalt percentages will be used in preparing further mixes with different percentages of lime and Hedmanite as substitute to the conventional crushed stone filler. The prepared mixes will be subjected to further tests, such as Stability loss, Resilient modulus, Split tensile strength, Creep , Fatigue and Rutting.

Chapter-4 details the different laboratory evaluation tests carried out for conventional and modified mixes.

Chapter 4

Laboratory Evaluation and Test Results

4-1 Introduction

This chapter details the laboratory testing carried out to evaluate the engineering properties of asphalt concrete mixtures. A series of dynamic and static tests were carried out to characterize the various mixes designed by Marshall mix design procedure as discussed in Chapter 3. To simulate their behavior under field conditions, various tests were conducted that include,

(1) Marshall stability loss at 60° C.

(2) Unconfined Creep test at 60° C.

(3) Split Tensile Strength test at 25° C.

(4) Resilient modulus test at 45° C.

(5) Fatigue and Rutting at 45° C & 60° C.

The experimental work is described for each test and results are presented, along with some observations.

4 -2 Experimental Program

Mixes were prepared by Marshall method using different percentages of lime and Hedmanite as a substitute to crushed stone filler in the aggregate gradation. Optimum asphalt content as obtained in Chapter 3 for wearing coarse and base coarse mix was used in all of these mixes. In order to study the effect of filler content on characteristics of designed mixes, samples with 0%,1%, 2%, 4%, & 5.5% of lime and Hedmanite as a replacement of filler were fabricated. (i.e., the weight of crushed stone filler is reduced by the amount of lime or hedmanite added, based on the weight of total aggregate). Samples having 0% lime or hedmanite represents the control mix. Twenty nine samples for each percentage of lime and Hedmanite were fabricated for both gradations. Distribution of these samples for each test is as indicated in Table- 4.1. Three specimens were tested for resilient modulus at 45°C, three tested for split tensile strength at 25°C (2hrs). Further three samples were subjected to split tensile test after conditioning for 24hrs at 60°C followed by 2hrs at 25°C. Six specimens were tested for Marshall stability, three at 60°C for 35min, and three at 60°C for 24hrs. Two samples tested for static creep at stress level of 60 psi (60°C), from the remaining, six samples each were tested for fatigue & Rutting at 45°C and 60°C. In this way all the samples for each percentages shown above, were utilized in five different characterization tests of Asphalt Concrete mixes.

In all these tests, for representing different types of mixes the following nomenclature was used.

Table 4.1 : Experimental Design for WC & BC mixes

Test		Control Mix	Hedmanite				Lime			
			1%	2%	4%	5.5%	1%	2%	4%	5.5%
Stability Loss @ 60°C		6	6	6	6	6	6	6	6	6
Resilient Modulus M_R @45°C		3	3	3	3	3	3	3	3	3
Split Tensile test	@ 25°C (2 hrs)	3	3	3	3	3	3	3	3	3
	@ 60°C (24 hrs) +@ 25°C (2 hrs)	3	3	3	3	3	3	3	3	3
Creep Test @ 60°C		2	2	2	2	2	2	2	2	2
Fatigue & Rutting	@ 45°C	6	-	6	-	6	-	6	-	6
	@ 60°C	6	-	6	-	6	-	6	-	6

WC → Wearing course (G1)
BC → Base course (G2)
" - " → Not considered.

i) The symbol G1 and G2 was used to represent Wearing Coarse (WC) and Base Course (BC) mixes.

ii) For Lime and Hedmanite, the symbol 'L' and 'H' was used.

iii) Mixes with 0% Lime/Hedmanite indicates the 'Control Mix' (Having conventional crushed stone powder as filler).

iv) Mixes with 1%, 2%, 4% of Lime/Hedmanite indicates the percentage of crushed stone filler replaced in the mix by these materials.

v) Mixes with 5.5% of Lime/Hedmanite shows the total replacement of crushed stone filler by these materials as filler in the aggregate gradation.

4 -3 Repeated Load Testing System.

This system is shown in fig. 4.1. It includes an air powered penumatic testing apparatus and a control cabinet through which dynamic and static load can be controlled. Load duration frequency can be controlled through this control cabinet, which consists of electro-pneumatic system, consisting of Bellofram air cylinder, a shuttle valve and a mac valve. The detailed functioning of control cabinet and electrical system is briefed in reference [41].

The loading frame used to apply repeated dynamic vertical load is housed in an environmental chamber so that the tests can be conducted at designated temperature ranging from -20° C to 85° C.

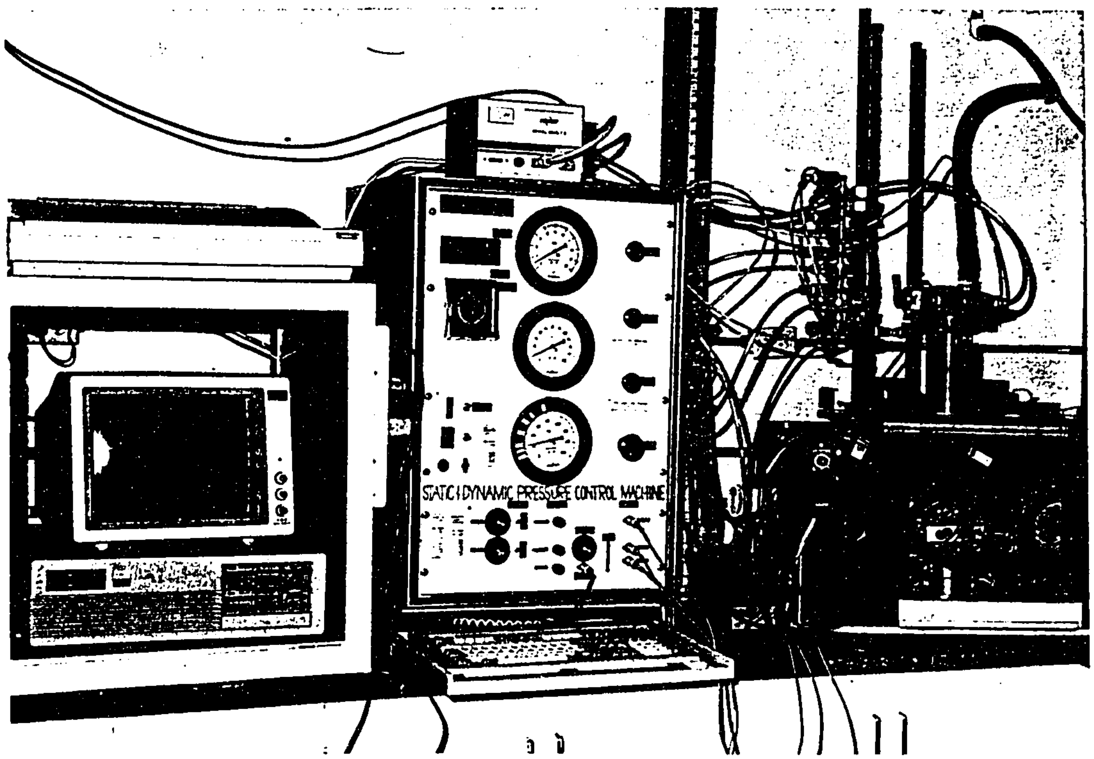


Figure 4.1 : Repeated Loading Testing System

Resilient Modulus (MR), Permanent deformation, and Fatigue are the three diametrical tests which are performed using the 'Repeated loading test system'. In this study, the test conditions which were kept constant for the three diametral tests are summarized as follows:

1. Specimens were placed inside the chamber or in an oven for about 6 hours prior to testing to enable the specimens to reach the specified temperature required for testing.

2. A static load of 10 lbs was applied as a seating load to hold the specimen in place.

3. A dynamic load of 100 lbs was used for Resilient modulus test.

4. Dynamic load duration of 0.10 second was fixed. And the load frequency was adjusted at 60 cycles per minute (1 Hz).

5. Curved loading plates of 1/2 inch wide and 2.5 inch long were used.

6. Testing temperatures were 45° C and 60° C.

The dynamic load of 100 lb was selected because it was found that it was suitable for testing under the selected test temperatures and for the sensitivity of the recording system. This load (100 lb) is corresponding to 80 psi pressure on the specimen under testing (Using a curved loading plate of 2.5 inch long and 0.5 inch wide). This pressure value represents the standard tire pressure of the standard 18-kip axle with dual wheel.

4 -4 Resilient Modulus (MR) Test

Marshall fabricated specimens from each percentage of lime and hedmanite modified mixes as well as control mix were subjected to a resilient modulus (MR) test at 45° C and under dynamic loading of 100 lbs. A Sinusoidal (haversine) axial compression stress is applied to each specimen at a loading frequency of 0.10 sec. The resulting recoverable axial strain response of the specimen is measured. The value of resilient modulus (MR) in Ksi is calculated according to the equation (2).

The test setup is shown in Figure 4.2 . A diametral yoke was used to measure the horizontal deformation of cylindrical specimen subjected to dynamic vertical loading. The horizontal deformation of cylindrical specimen was measured by two LVDT's while the load was measured using flat load cell. Average amplitude of load and the strain over the last five loading cycles as recorded by the computer attached to the system, are used for the calculation of MR value. The value of dynamic modulus (MR) can be used for both asphalt paving mixture design and asphalt pavement thickness design.

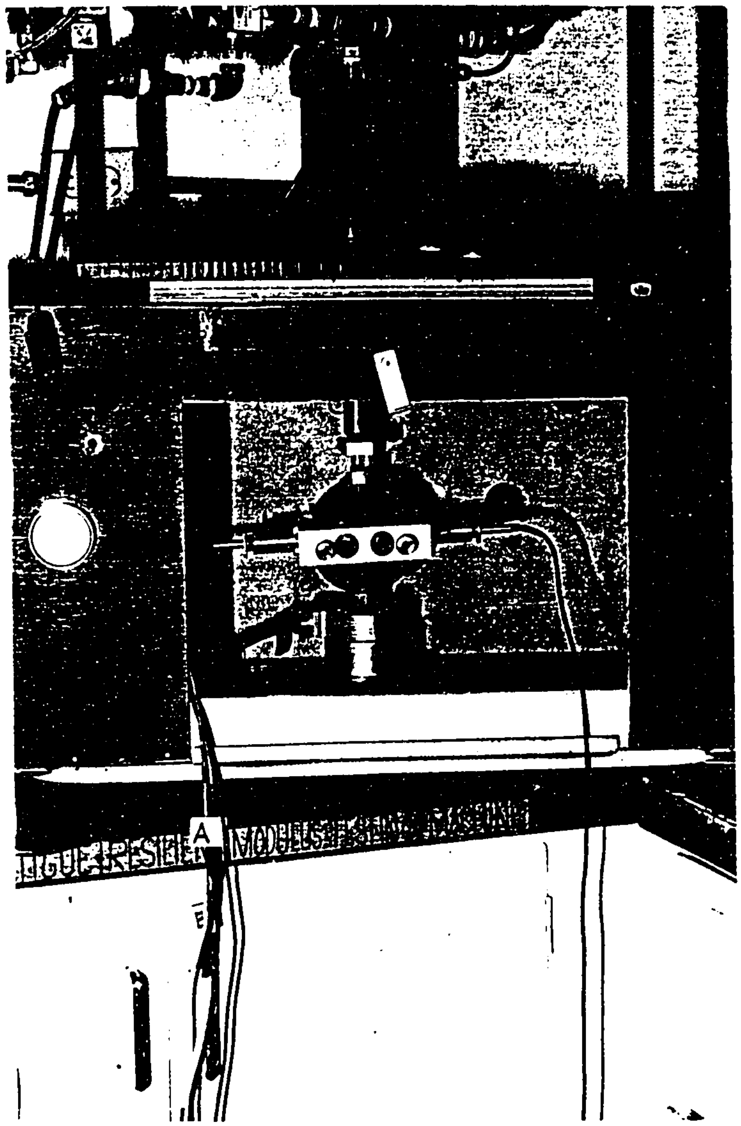


Figure 4.2 : Resilient Modulus Test Setup

4 -4 -1 Test Results

Resilient modulus was determined for both wearing coarse and base coarse mixes at 0%, 1%, 2%, 4%, & 5.5% of Lime or Hedmanite modified mixes. The results of modulus at optimum asphalt content of each mix designed by marshall method are summarized in figure 4.3 to 4.5. Higher values of MR are found with 1%, 2%, & 4% lime modified mixes as compared to hedmanite modified mixes. However at 5.5% the moduli value for hedmanite is much higher than that of lime. It is observed that as the percentage of hedmanite is increased the moduli value increases for both wearing coarse and base coarse mixes, indicating that the mix is becoming stiffer. Also the MR values for base coarse mixes is higher than that of wearing coarse. Comparision of MR between lime and hedmanite modified mixes (Fig.4.5) indicates that, in case of wearing coarse, upto 4% of lime can increase the moduli value and above that percentage it will decrease the moduli tremendously. It may be attributed to the production of dry mix. Mixes with 2% and above hedmanite shows an increase in the MR value as compared to the control mix.

4 -5 Marshall Stability

It was noted from the previous studies that the major problem with local aggregates is the loss of stability, it was reported that the amount of filler was found to be one of the major contributors to stability loss (21). Thus, in this study the marshall stability analysis was performed on modified mixes to know the effect of lime and hedmanite

Figure 4.3 : Resilient Modulus for Lime modified mixes:

Mix type		Resilient Modulus (MR) in ksi @ 45 C.	
		Wearing Course (G1)	Base Course (G2)
		G1L	G2L
Control	0%	135.75	150.80
Lime	1%	142.64	160.80
	2%	160.30	170.16
	4%	168.90	158.12
	5.5%	128.80	147.60

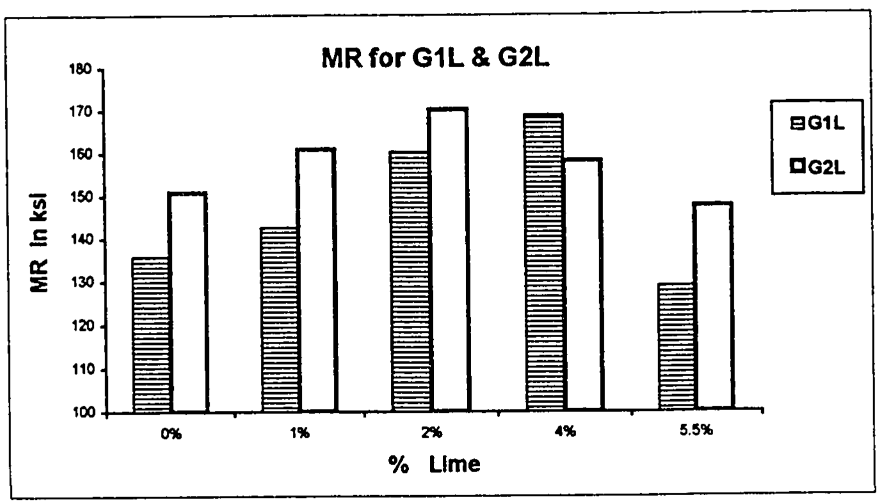


Fig 4.4: Resilient Modulus for Hedmanite modified mixes

Mix Type		Resilient Modulus (MR) in ksi @ 45 C.	
		Wearing Course (G1)	Base Course (G2)
Control	0%	135.75	150.80
Hedmanite	1%	132.10	144.04
	2%	141.80	151.40
	4%	148.60	162.50
	5.5%	154.80	174.80

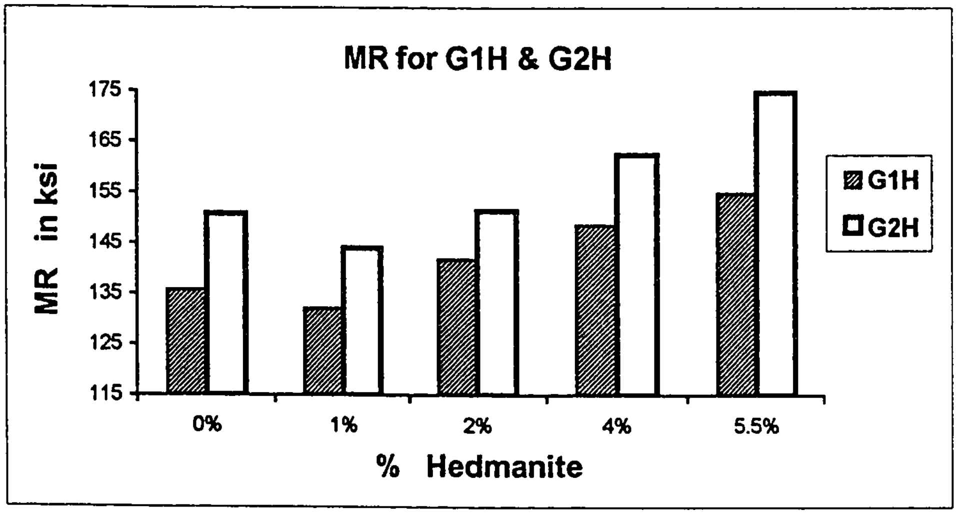
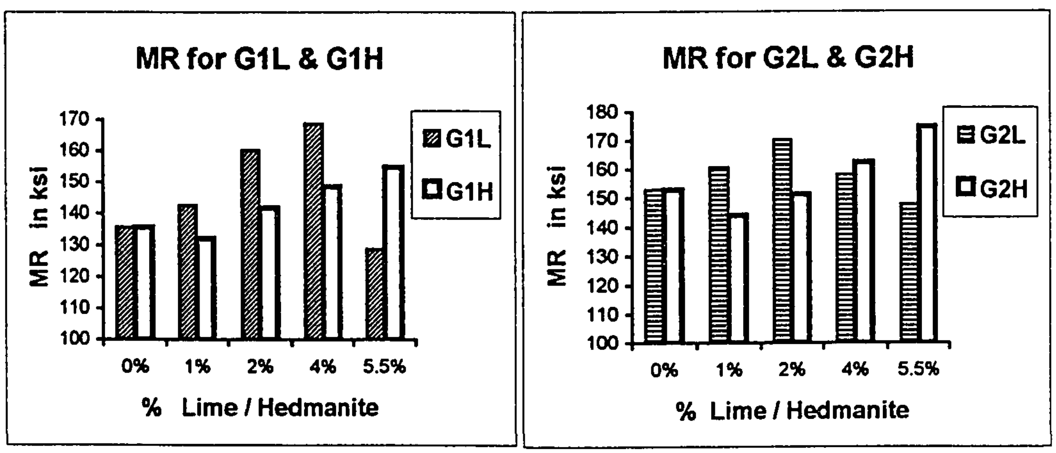


Fig 4.5 : Comparision for MR between Lime & Hedmanite mixes

Mix Type		Resilient Modulus (MR) in ksi @ 45 C.			
		Wearing Course (G1)		Base Course (G2)	
		G1L	G1H	G2L	G2H
Control	0%	135.75	135.75	152.80	152.80
Lime or Hedmanite	1%	142.64	132.10	160.80	144.04
	2%	160.30	141.80	170.16	151.40
	4%	168.90	148.60	158.12	162.50
	5.5%	128.80	154.80	147.60	174.80



on the stability loss characteristic of the AC mixtures.

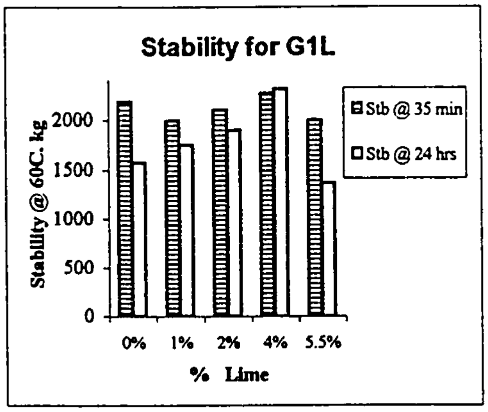
4 -5 -1 Test Results

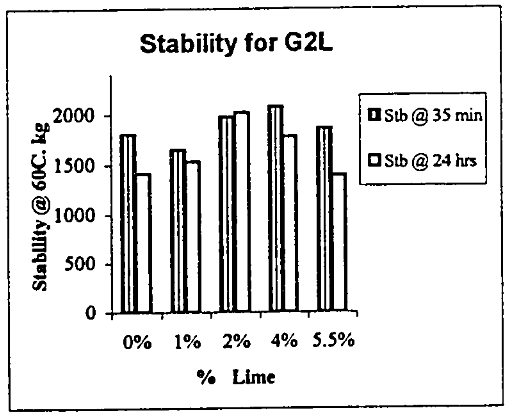
In order to determine the stability loss of the modified mixes, marshall stability analysis at 60° C after 35 minute and 24 hours of immersion in water was performed. This has led to the results shown in Figure 4.6 & 4.7. for lime and hedmanite modified mixes respectively. And the comparision between them is shown in Fig. 4.8.

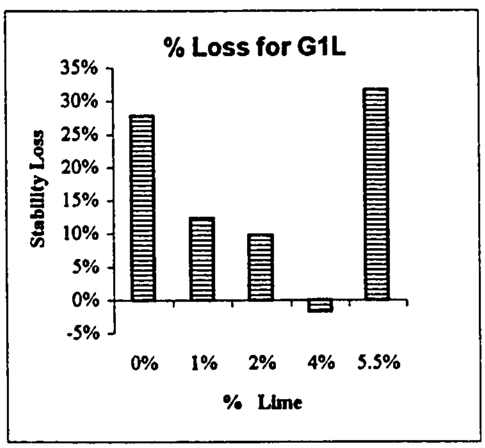
It was observed that upto 4% of lime can reduced the stability loss tremendously and can even prevent it. In fact, some of the specimens (at 4% lime in WC, and 2% lime in BC) showed higher stability after 24 hours immersion in water than that of 30 minute, indicating a gain in stability. This can be attributed to the cementicious property attained by lime when reacted with water [29]. Whereas, at 5.5% lime there is 4% increase in the stability loss for both wearing coarse (WC) and base coarse (BC)mixes, compared to the control mix. This may be due to the excess of lime which has created a dry mix having improper coating of asphalt over the aggregates. In the case of hedmanite modified mixes, there is a reduction in stability loss for 1%, 2% & 4% addition of hedmanite in wearing coarse and 1%, 2% in base coarse, as compared to the control mix. Whereas, when crushed stone filler is replaced completely by hedmanite (5.5%) as filler, there is an increase in stability loss in comparison to control mix (0%). A comparison of hedmanite modified mixes with control mix indicates a drop in stability loss upto 4% addition of hedmanite as compared to control mix, and an increase at 5.5%.

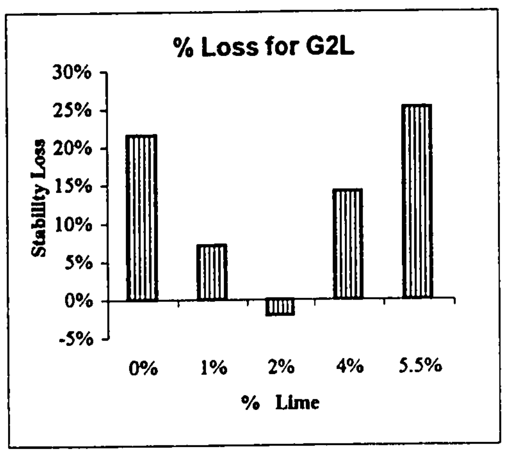
Fig 4.6 :Marshall Stability for Lime modified mixes:

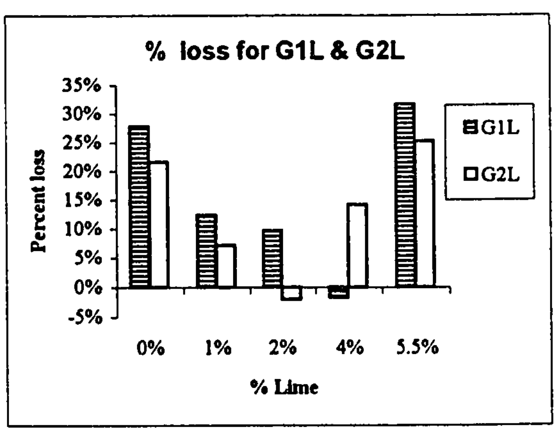
Mix type		Marshal Stability (kg) @ 60 C, 35 Min.		Marshal Stability (kg) @ 60 C, 24 Hrs.		Percent Loss	
		G1	G2	G1	G2	G1	G2
Control	0%	2181.98	1802.9	1575.39	1413.47	27.8%	21.6%
Lime	1%	1993.7	1653.05	1748.47	1535.68	12.3%	7.1%
	2%	2100.6	1975.2	1895.6	2017	9.7%	-2.1%
	4%	2263.7	2070.5	2304.5	1778.3	-1.8%	14.1%
	5.5%	1996.7	1861.8	1365.7	1392.63	31.6%	25.2%







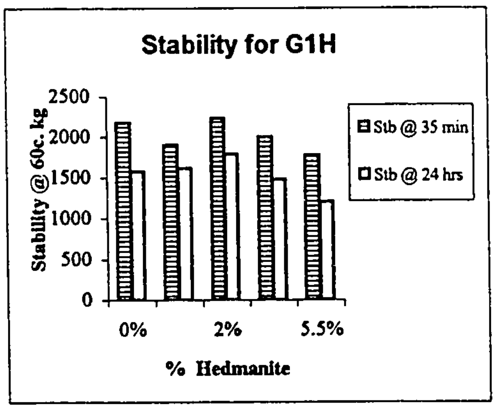


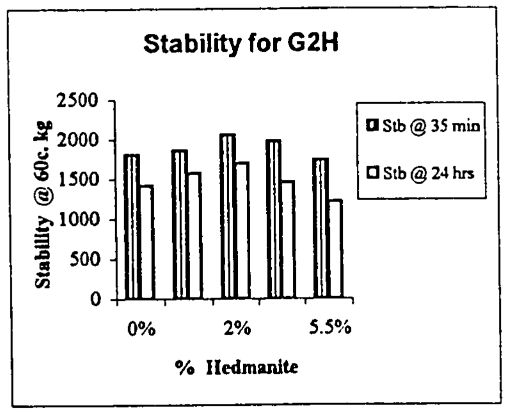


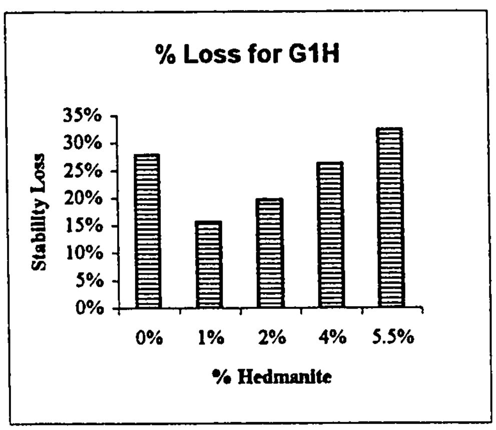
* G1L - Wearing Course.
* G2L - Base Course.

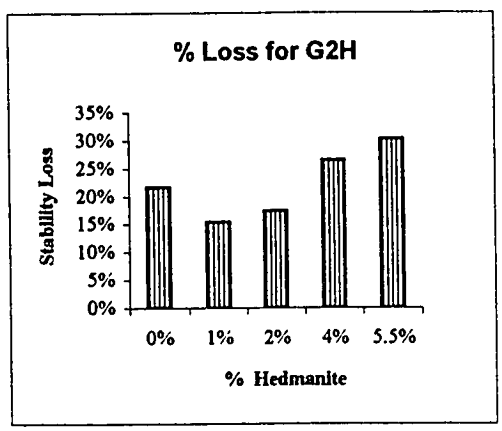
Fig 4.7 :Marshall Stability for Hedmanite modified mixes

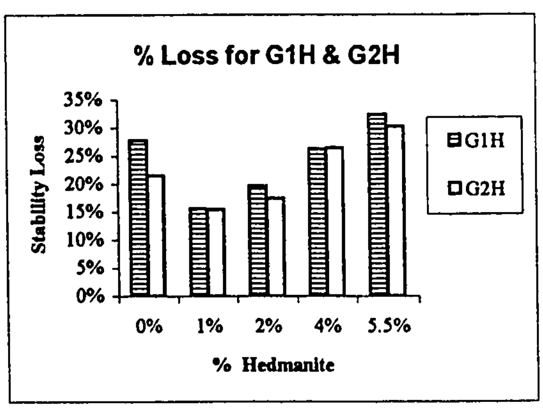
Mix type		Marshal Stability (kg) @ 60 C, 35 Min.		Marshal Stability (kg) @ 60 C, 24 Hrs.		Percent Loss	
		G1	G2	G1	G2	G1H	G2H
Control	0%	2181.98	1802.9	1575.39	1413.47	27.8%	21.6%
Hedmanite	1%	1907.1	1859.6	1609.6	1573.2	15.6%	15.4%
	2%	2232.6	2056.8	1792.8	1698.9	19.7%	17.4%
	4%	1999.6	1973.1	1472.3	1452.3	26.25%	26.4%
	5.5%	1774.5	1735.15	1200.4	1211.13	32.32%	30.2%







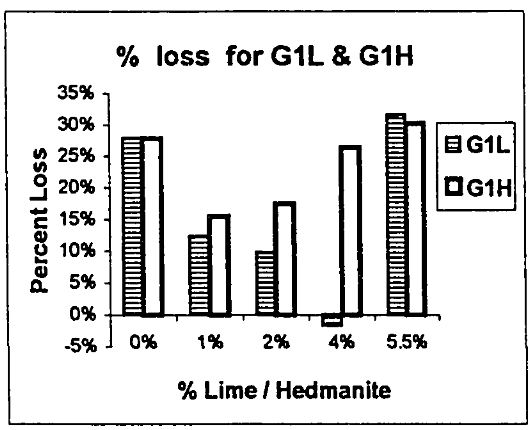


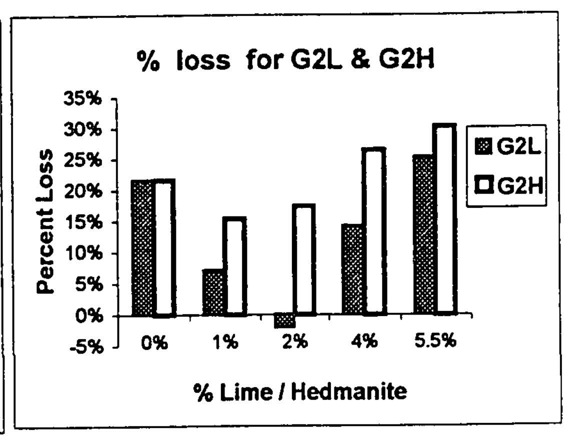


* G1H - Wearing Course.
* G2H - Base Course.

Fig 4.8 : Comparision for Stability Loss between Lime & Hedmanite mixes

Mix Type		% Loss in Marshall Stability.			
		Wearing Course (G1)		Base Course (G2)	
		G1L	G1H	G2L	G2H
Control	0%	27.80%	27.80%	21.60%	21.60%
Lime or Hedmanite	1%	12.30%	15.40%	7.10%	15.40%
	2%	9.70%	17.40%	-2.10%	17.40%
	4%	-1.80%	26.40%	14.10%	26.40%
	5.5%	31.60%	30.20%	25.20%	30.20%





4-6 Split Tensile Strength

Specimens prepared by Marshall mix design method for optimum asphalt content and varying range of lime and hedmanite as a replacement to conventional filler, were subjected to failure in the indirect tensile test. The test involved loading the specimen with a compressive load acting parallel to and along vertical diametrical plane through 0.5 inch (13 mm) wide stainless steel strips which are curved at interface with the specimen. Specimen failed by splitting along the vertical diameter (Fig. 4.9). Split tensile strength was determined by the following equation.

$$S_T = 2P_{max} / \pi h D \quad \text{...} \quad (6)$$

Where: S_T = Split tensile strength

P_{max} = Load at failure, lbs

D = Diameter of sample (4 inches)

h = Sample thickness, inches

The strain rate of 2 inches (50.8 mm) per minute was used in test because it could be easily performed on the Marshall testing machine.

In order to determine the percent loss in tensile strength of the mixes. Specimens were tested after 2 hours (25°C) and 24 hrs (60°C) + 2 hrs (25°C) of immersion in water.

Difference in tensile strength of 2 hrs and 24 hrs samples divided by the initial 2 hrs strength is reported as the 'Percent loss' in split tensile strength.

4 -6 -1 Results

The results of split tensile test are shown in Fig. 4.10 and Fig. 4.11 for lime and hedmanite modified mixes respectively. It is observed that upto 4% of lime in wearing coarse and 2% in base coarse, can enhance the split tensile strength of the asphalt concrete (AC) mixes. In comparision to the tensile strength loss of the control mix, there is 15% reduction in tensile strength loss when 4% lime is added as a part of crushed stone filler substitute in WC mix. Similarly at 2% lime in BC mix the tensile strength loss is decreased by 12%.

Whereas, in hedmanite modified mixes there is a tremendous loss in strength as compared to the control mix. It can be noted that as the percentage of hedmanite is increased from 1% to 5.5%, the amount of strength loss has increased for both WC and BC mixes. It was also observed that the two hour, (25° C) strength is reasonably good, therefore it indicate that the hedmanite modified mixes are loosing the strength after 24 hour immersion in water, It may be due to the expansion or collapse of hedmanite with water. An illustrative comparision of split tensile strength between lime and hedmanite is depicted in figure 4.12.

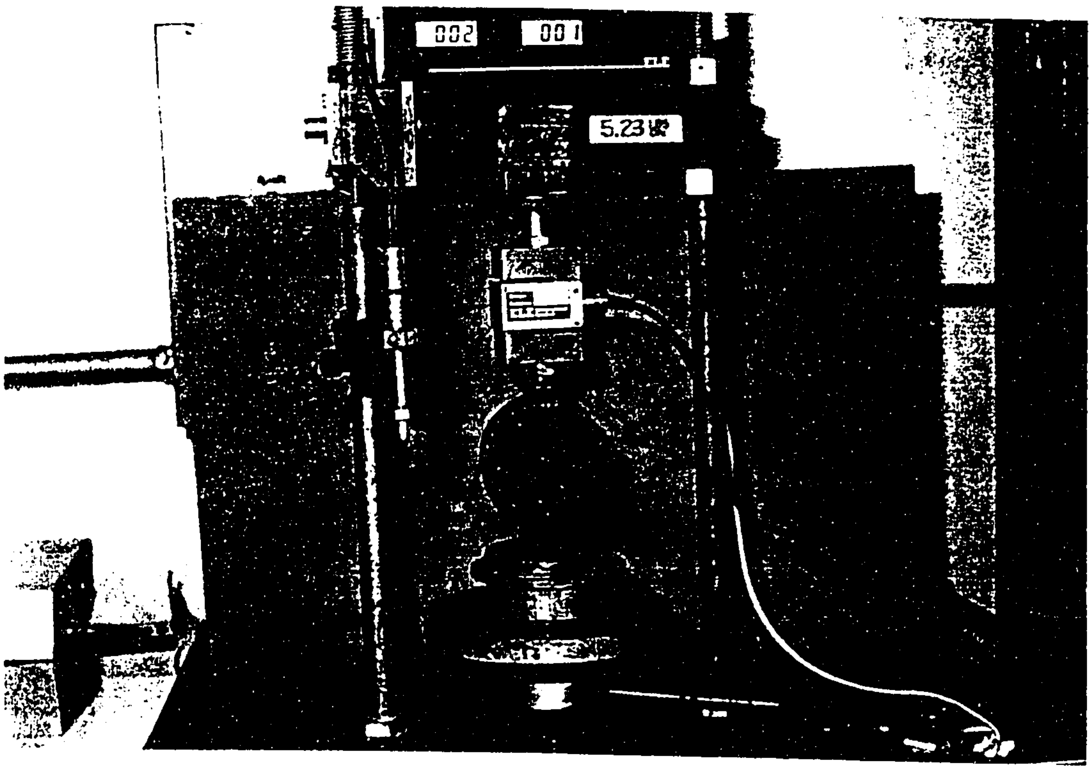
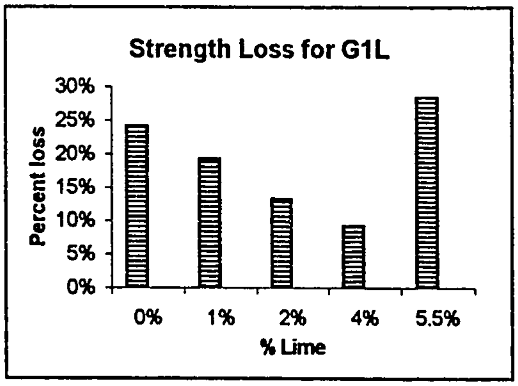
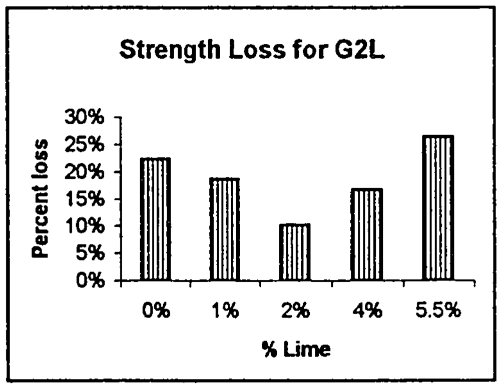


Figure 4.9 : Setup for Indirect Tensile Test.

Fig 4.10 : Split Tensile Strength for Lime modified mixes:

Mix Type		Split Tensile strg (psi) @ 25 C, 2 hrs.		Split Tensile strg (psi) @ 60 C, 24 hrs.		Percent Loss	
		G1	G2	G1	G2	G1	G2
Control	0%	148.18	111.85	112.5	86.9	24.1%	22.3%
Lime	1%	141.4	132.89	114.2	105.5	19.20%	18.60%
	2%	167.1	130.81	145.1	117.6	13.17%	10.10%
	4%	148.58	118.6	134.9	98.6	9.20%	16.60%
	5.5%	120.7	93.9	86.4	69.2	28.40%	26.30%





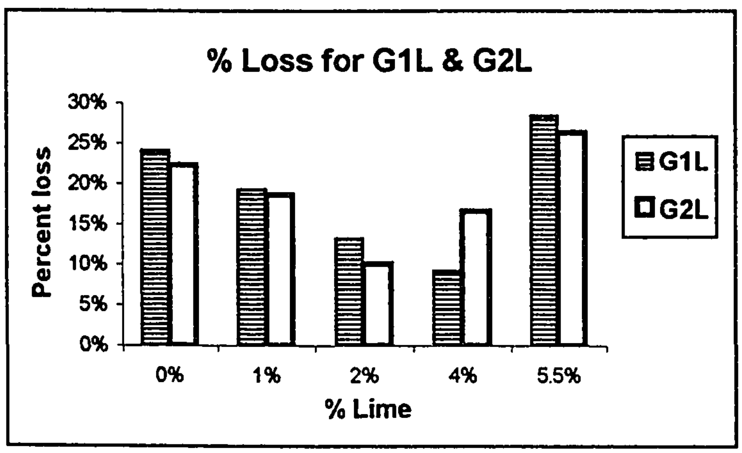
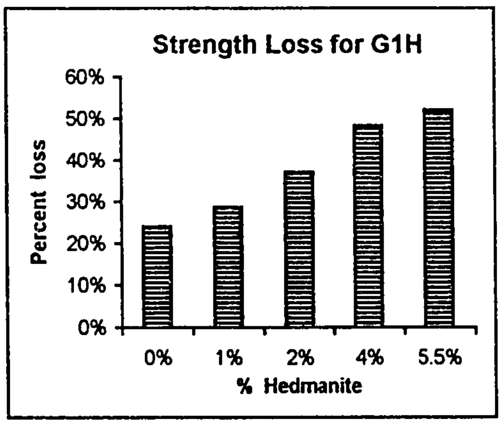
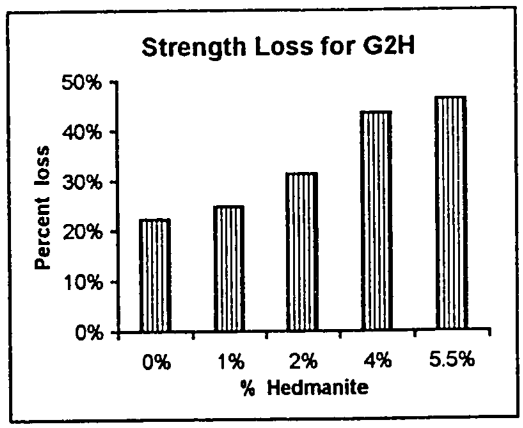


Fig 4.11 : Split Tensile Strength for Hedmanite modified mixes

Mix type		Split Tensile strg (psi) @ 25c for 2 hrs.		Split Tensile strg (psi) @ 60c for 24 hrs.		Percent Loss	
		G1	G2	G1	G2	G1	G2
Control	0%	148.18	111.85	112.5	86.9	24.1%	22.3%
Hedmanite	1%	138.1	122.9	98.5	92.4	28.70%	24.80%
	2%	162.6	138.6	102.3	95.1	37.10%	31.40%
	4%	148.8	130.5	76.8	73.7	48.40%	43.50%
	5.5%	143.3	124.7	68.7	66.9	52.02%	46.30%





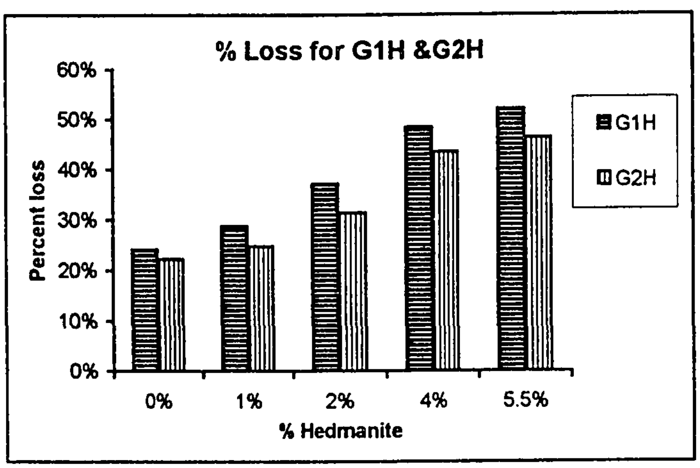
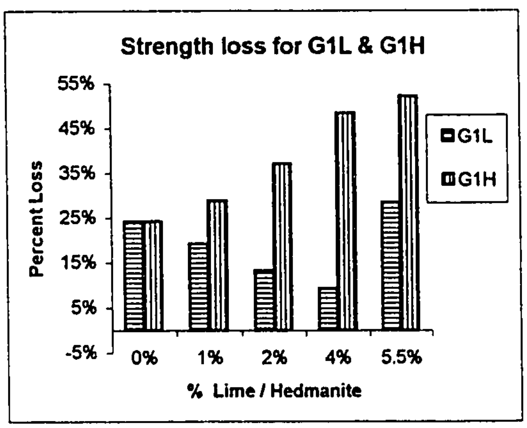
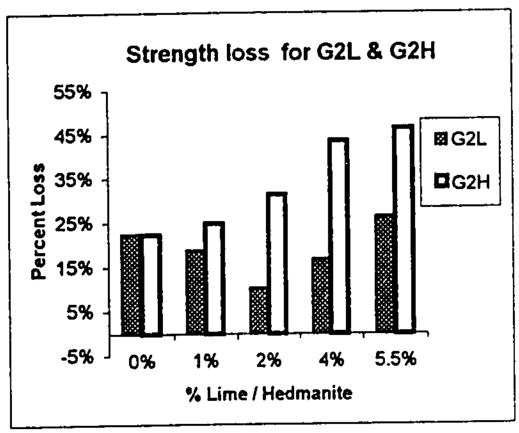


Fig 4.12 : Comparision of Split Tensile Strength between Lime & Hedmanite

Mix Type		% Loss in Split Tensile Strength(STT).			
		Wearing Course (G1)		Base Course (G2)	
		G1L	G1H	G2L	G2H
Control	0%	24.10%	24.10%	22.30%	22.30%
Lime or Hedmanite	1%	19.20%	28.70%	18.60%	24.80%
	2%	13.17%	37.10%	10.10%	31.40%
	4%	9.20%	48.40%	16.60%	43.50%
	5.5%	28.40%	52.02%	26.30%	46.30%





4 -7 Static Creep Test

In this study Creep test was performed as per the Shell procedure, on the specimens prepared by Marshall Mix Design Method, for different filler contents at a temperature of 60°C. Each specimen was tested at a stress level of 60 psi and maintained for 2hrs, one hour loading and one hour unloading. Measured vertical deformations at different loading periods were recorded accurately on a computer using data logger. The data logger was set to take the reading of vertical deformation at every one minute and was connected to a computer where all the data can be stored. The test setup is shown in Figure. 4.13 and 4.14.

4 -7 -1 Results

As discussed in section 2.4.4 (Chapter-2), the creep test is used primarily to determine the linear viscoelastic properties of material. Therefore, the vertical deformation recorded during the test were plotted with respect to time. The curves obtained are shown in figures 4.15 & 4.16, and figure B-1 to B-16 in the Appendix-B for various mix types. From these curves the total deformation , elastic strain, viscoelastic strain and permanent strain were recorded for each mix type, and is reported in table- 4.2.

For Example: In figure 4.15, the total deformation of the sample is 1.1mm, out of which 0.58mm is the elastic strain, 0.17mm constitute viscoelastic part, and the permanent strain is 0.35mm.

The results obtained indicates no specific trend in the linear viscoelastic properties of both wearing coarse and base coarse mixes, having different percentages of lime or hedmanite as a filler substitute. The 4" diameter and 2.5" high specimen (specified by Shell group), may not be sufficient to show the correct behavior of mixture under static loading, for the particular aggregate gradation used in this research. The sample height need to be increased to 8" in order to maintain the height to diameter ratio of about 2.0 [50]. The total deformation, elastic strain, viscoelastic strain and permanent strain are shown on the typical figure for wearing coarse control mix. i.e. Fig. 4.15.

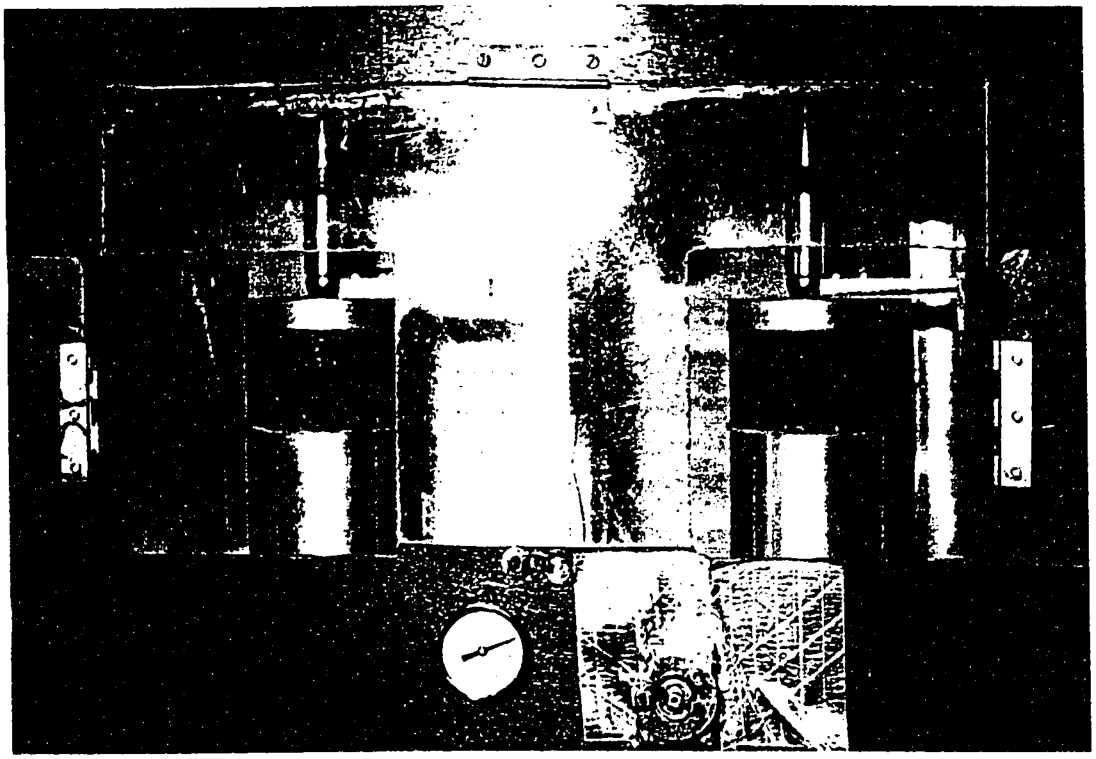


Figure 4.13 : Static Creep Test Chamber

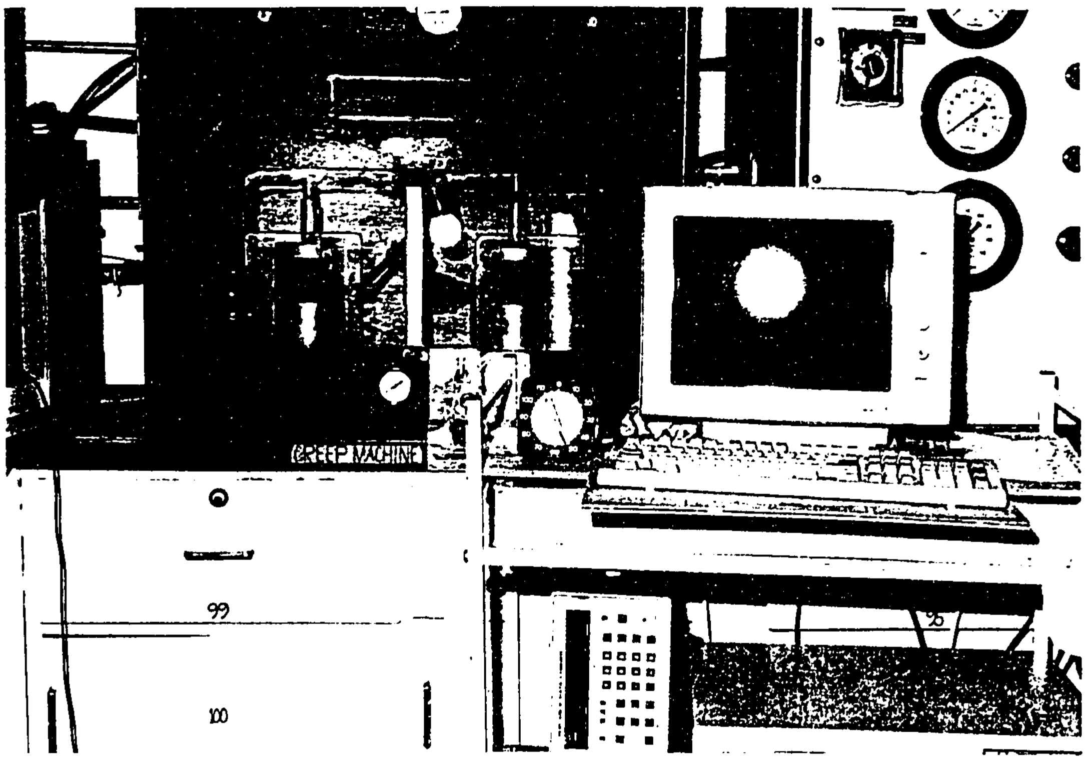


Figure 4.14 : Static Creep Test Setup.

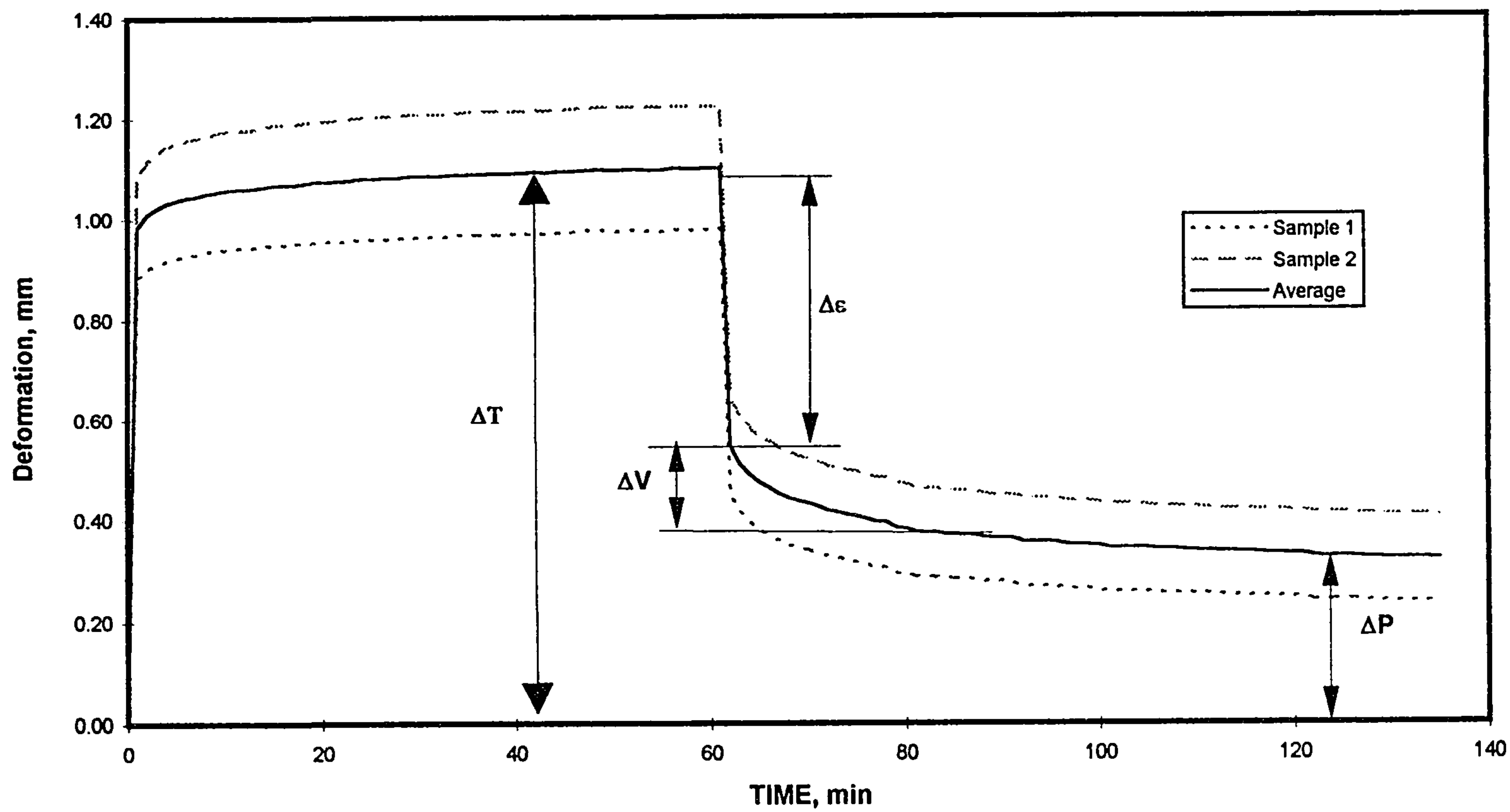


Fig 4.15: Typical Creep Curve for Wearing Coarse Control Mix (G1)

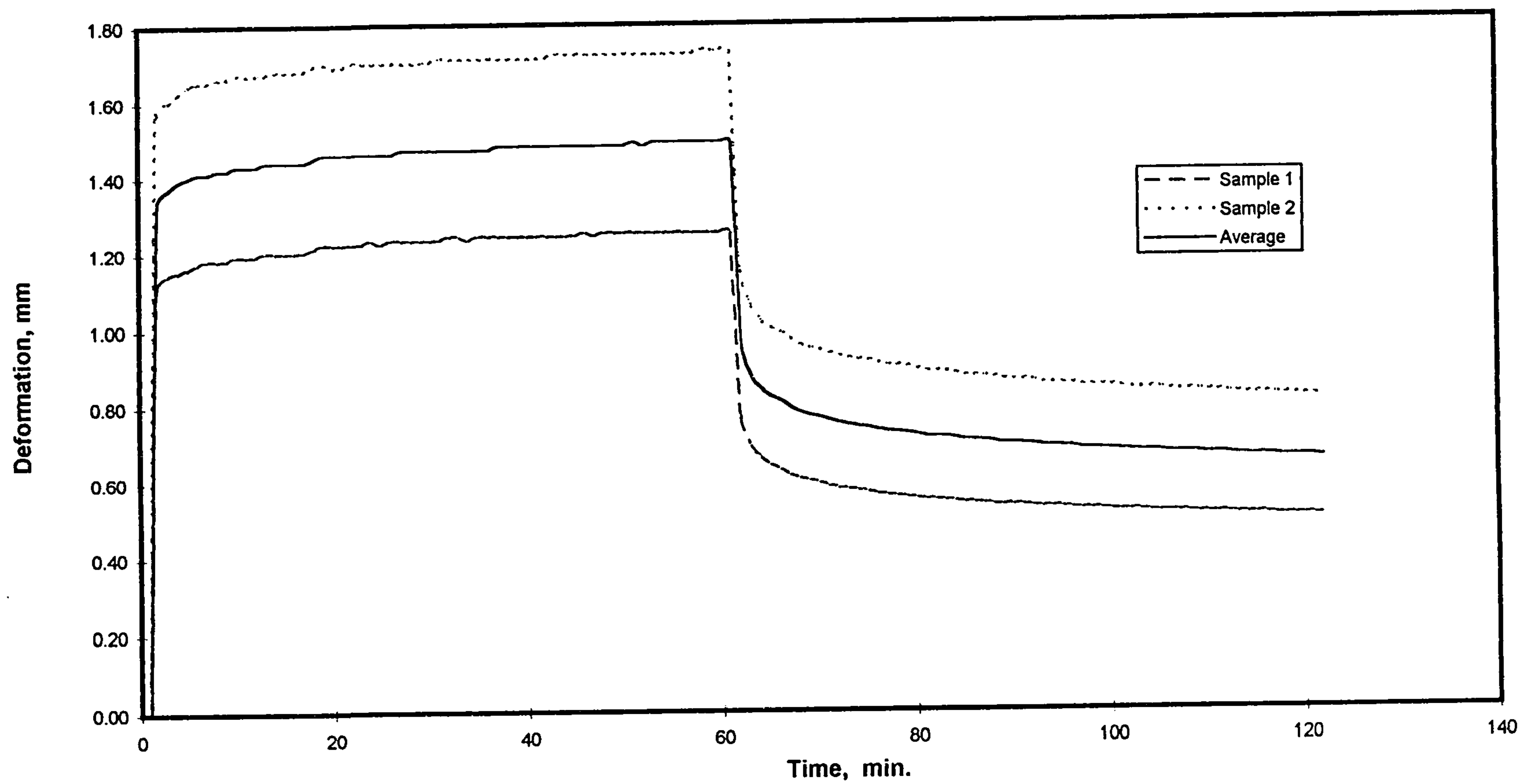


Fig. 4.16 : Creep Curve for Base Coarse Mix (G2)

Table 4.2 : Creep Curves Analysis Results.

Mix Type	Total Deformation ΔT (mm)	Elastic Strain Δε (mm)	Viscoelastic ΔV (mm)	Permanent Strain Δ P (mm)
Wearing Coarse (WC)	1.1	0.58	0.17	0.35
Base Coarse (BC)	1.49	0.51	0.24	0.74
G1L-1%	0.91	0.41	0.12	0.38
G1L-2%	0.8	0.5	0.20	0.10
G1L-4%	1.2	0.66	0.16	0.38
G1L-5.55	0.8	0.47	0.15	0.18
G1H-1%	1.14	0.54	0.16	0.44
G1H-2%	1.15	0.45	0.20	0.50
G1H-4%	1.25	0.51	0.31	0.43
G1H-5.5%	1.16	0.52	0.20	0.44
G2L-1%	0.6	0.36	0.11	0.13
G2L-2%	0.93	0.43	0.18	0.32
G2L-4%	1.0	0.48	0.14	0.38
G2L-5.5%	0.97	0.45	0.21	0.31
G2H-1%	1.6	0.58	0.22	0.8
G2H-2%	1.26	0.45	0.22	0.58
G2H-4%	0.9	0.41	0.12	0.37
G2H-5.5%	1.03	0.40	0.25	0.38

4 -8 *Fatigue and Permanent Deformation*

As discussed in section 2.4.5 (Chapter-2), Fatigue test can be performed under controlled stress or controlled strain mode. In this research, the fatigue test was performed under the constant stress mode, with a constant load during the test. This load was chosen based on some initial horizontal tensile strain level. In this study at least three strain levels were used for each group of specimen.

The resilient modulus test was conducted on the specimens in order to determine the load which induces a desired initial tensile strain in the sample. i.e., in the MR test setup the dynamic load was increased to achieve the desired initial tensile strain required for fatigue test. The strain was monitored from the results shown by the computer attached to the MR setup. When the desired strain level has reached, the load was maintained constant and the specimen is removed from the MR test assembly and kept in the fatigue and permanent deformation setup between the two curved loading plates as shown in Fig. 4.17. A seating load of 10 lb was then applied to keep the specimen in place, and the fatigue test was started by selecting the switch to " fatigue position" on the repeated loading test device.

The vertical permanent deformation was recorded using a data logger and LVDT'S, which inturn is connected to a computer where the permanent deformation at various time intervals was stored. The complete setup is shown in Fig. 4.18. The data logger was set to take permanent deformation reading at every 10 seconds for the first

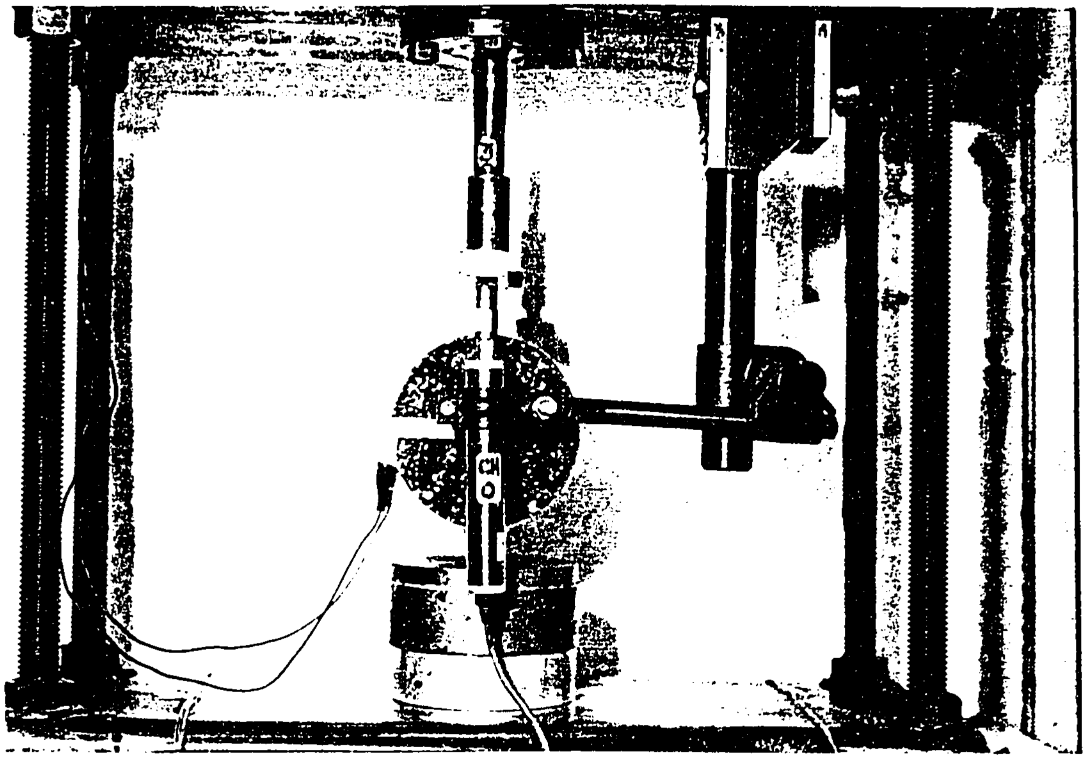


Fig. 4.17 : Fatigue and Permanent Deformation Specimen Setup.

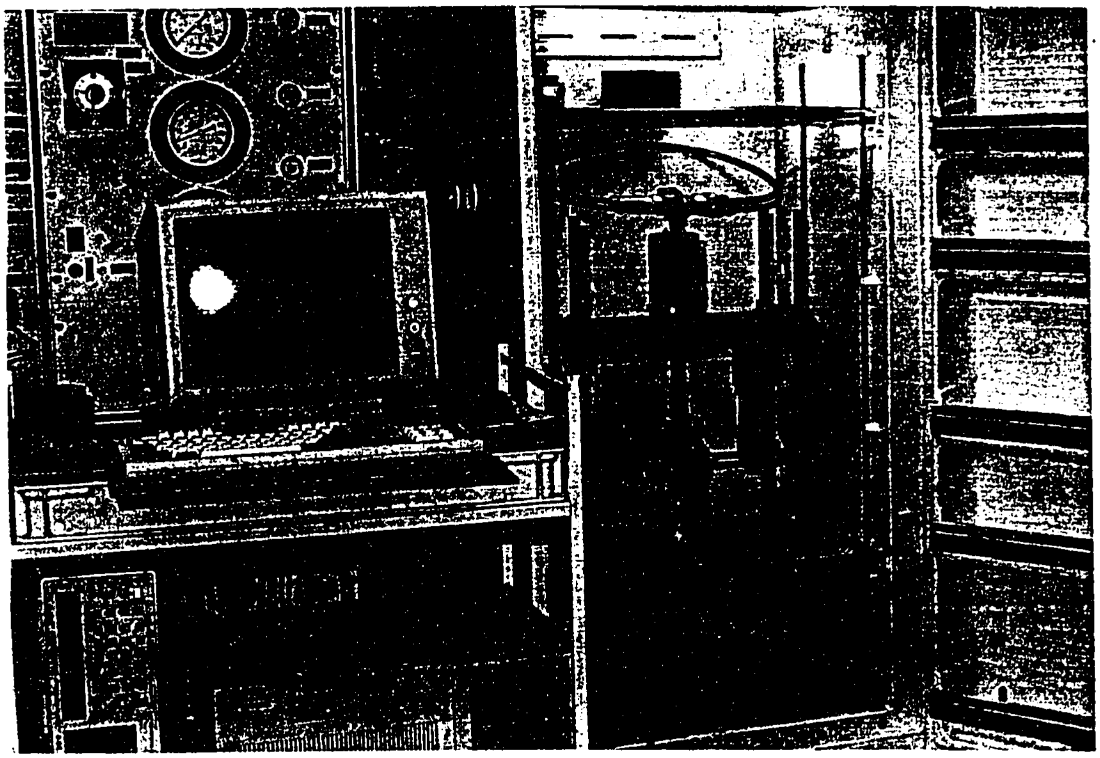


Fig.4.18: Fatigue & Permanent deformation test setup.

200 load repetitions, then at every 10 seconds for the next 1000 repetitions, and finally at every 2 minutes till the specimen fails. The system than stops automatically when the specimen fails, by the electric line cut-off and the number of load repetition to failure was recorded from the system counter.

The permanent deformation data recorded on the computer after each specified numbers of load repetitions were utilized for rutting characterization of the mixes. The permanent strain (ε_p) is calculated using the following equation (46).

$$\varepsilon_p = 5.9055 * 10^{-3} * Yt \qquad \text{.................................. (7)}$$

Where, ε_p = accumulated permanent strain.

Y_t = Total Vertical deformation in mm.

4 -8 -1 Test Results

Fatigue test results at 45°C and 60°C for wearing coarse mixes are shown in Fig.4.19 and Figs. C-1 to C-7 and that for base coarse mixes are shown in Fig.4.20 and Figs. C-8 to C-14 . These results show normal linear relationship between logarithm of applied initial tensile strain and the logarithm of fatigue life. The fatigue data obtained through a data logger and computer were analyzed by running a regression analysis to determine fatigue relationship parameters.

Test results indicates that the slope of regression line S, increases with higher test temperature, It was observed that the fatigue life tends to decrease with increasing temperature. The fatigue life for wearing coarse mixes is higher than that for base

coarse mixes. It may be due to higher asphalt content and more fine aggregates used in Wearing coarse mixes. It was also observed that with the increase in percentage of lime or Hedmanite (as a substitute to mineral filler) the fatigue life of the mixes has increased for both wearing coarse and base coarses, in comparison to the control mix. All results shows a high goodness of fit with a high coefficient of determination with R-Square approaching unity. The Solpe & Intercept values obtained for various mix types at the two testing temperatures are gathered in Table 4.3.

Rutting or Permanent Deformation:

The permanent deformation results for the control mix at 45°C and 60°C are shown in Figs. 4.21 and 4.22 respectively. Wearing coarse and Base coarse mixes modified with lime and hedmanite are shown in Fig C-19 to Fig C-22. at temperatures 45°C and 60°C. These results indicate that a straight line relationship exists between the logarithm of number of repetitions and the logarithm of the permanent strain. The permanent strain is obtained by converting the permanent deformation measured during the fatigue test using the equation-7 shown above.

In order to determine the parameters I (Intercept) and S (Slope), regression analysis was done on the experimental data. It was found that the experimental data were fitting the permanent deformation very satisfactorily. Similar to fatigue tests, values of the Coefficient of determination R-Square were very high. It was also observed that permanent deformation (Rutting) tends to increase with increase in temperature. The evaluation of curves indicates that hedmanite modified mixes tends to show more permanent deformation compared to lime mixes, which is concluded from the slope of

the straight line relation. The lower the slope value for a mix, the lower is the rutting susceptibility for that mix. These slope and intercept values of rutting curves for each group of mixes are shown in the Table 4.4.

4 -9 Statistical Analysis :

The effect of Hedmanite and Lime as filler material in the asphalt concrete mix, were analysed statistically using the data obtained from the different tests performed on modified mixes. The experimental design involves two factors 'Material type' and 'Percent material added' as shown in Table 4.1. Both material type and percentage added in the mix are tested statistically taking the results of each fundamental test at a time, for the null hypothesis "H_o : The data obtained has equal means". Null hypothesis is rejected at 95% confidence level if F_{cal} is greater than $F_{critical}$. Indicating that the data do not support the null hypothesis. The hypothesis are tested using a two-factor or three-factor analysis of variance (ANOVA) with replicates. The results are as discussed below.

Results of Statistical Analysis:

The analysis of resilient modulus (MR) data by ANOVA technique, shows that both material type and percent added have significant effect on MR in wearing coarse mixes, whereas the percent added has a more pronounced effect in base coarse mixes. Table D1 & D2 presents the results of anova analysis for resilient modulus (MR).

Statistical analysis on the results of Marshall Stability test after 35 minutes and 24 hours, reveals that the hypothesis " different percent added have equal means" can be rejected with a probability of 99.99% in wearing coarse and base coarse mixes. Indicating that there is an effect of the filler material and percent in the asphalt concrete mixes. The results in detail are shown in tables D3 & D4 for 35 minutes stability, and that of 24 hr stability in tables D5 & D6.

The results of statistical analysis for Split Tensile Strength after 2 hr and 24 hours are shown in Tables D7 - D10 for both wearing coarse and base coarse mixes. And that of Static Creep is presented in Table D11 . The "F" test results in these tables show that the percentage of different materials added as filler, has a significant effect on the Split tensile strength and Creep values, with a significance of about 99.99%.

A 3-Way analysis was performed on Fatigue and Rutting data, taking material type, percent material added as first two factor and initial strain (for fatigue) & number of repititions (for rutting) as the third factor. The analysis of variance for Fatigue and Rutting data are shown in Tables D12 - D13 and Tables D14 - D15 respectively. Large value of F (calculated) imply that the data do not support the null hypothesis, so the main effect of material type and percent added are significant. Furthermore, there is a significant interaction between material type and percentage as indicated by the results of different tests.

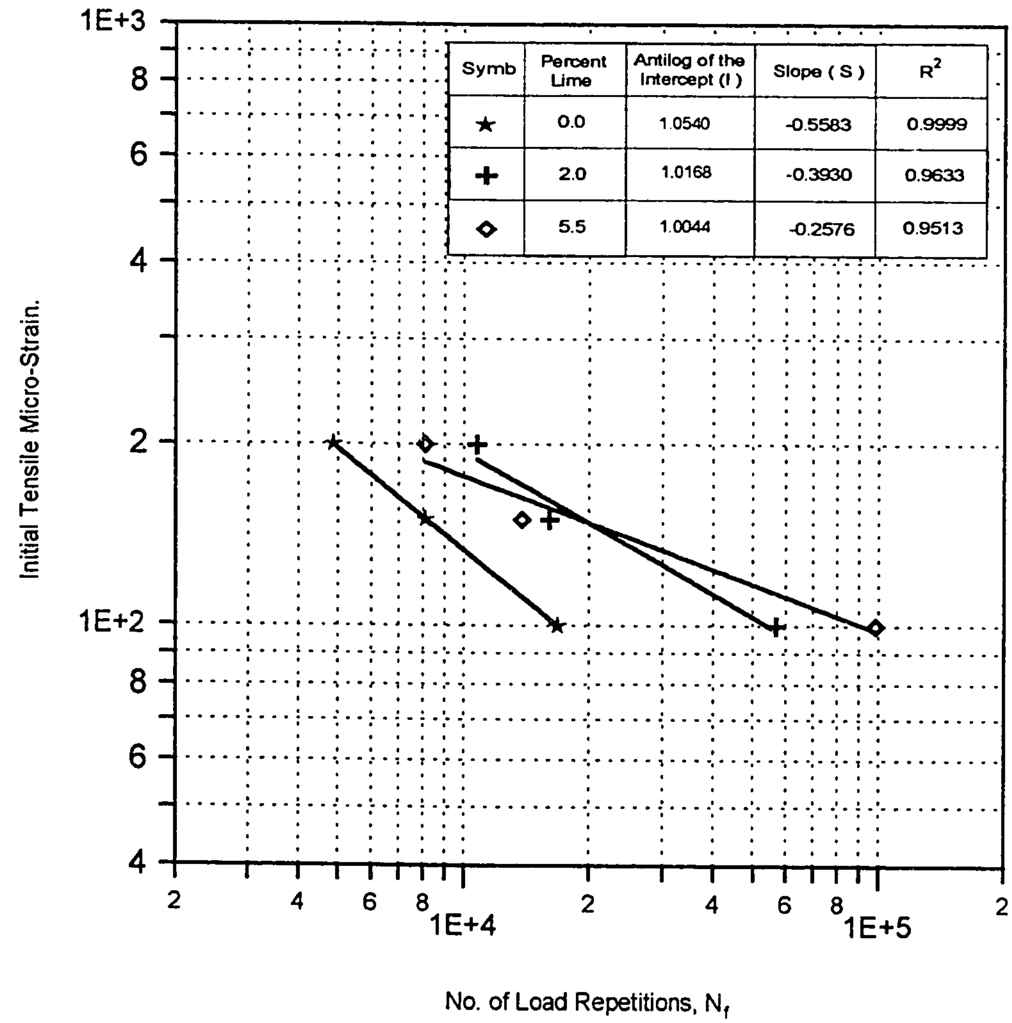


Fig. 4.19 : Fatigue Curve for Lime Modified Wearing Coarse Mixes at 45° C.

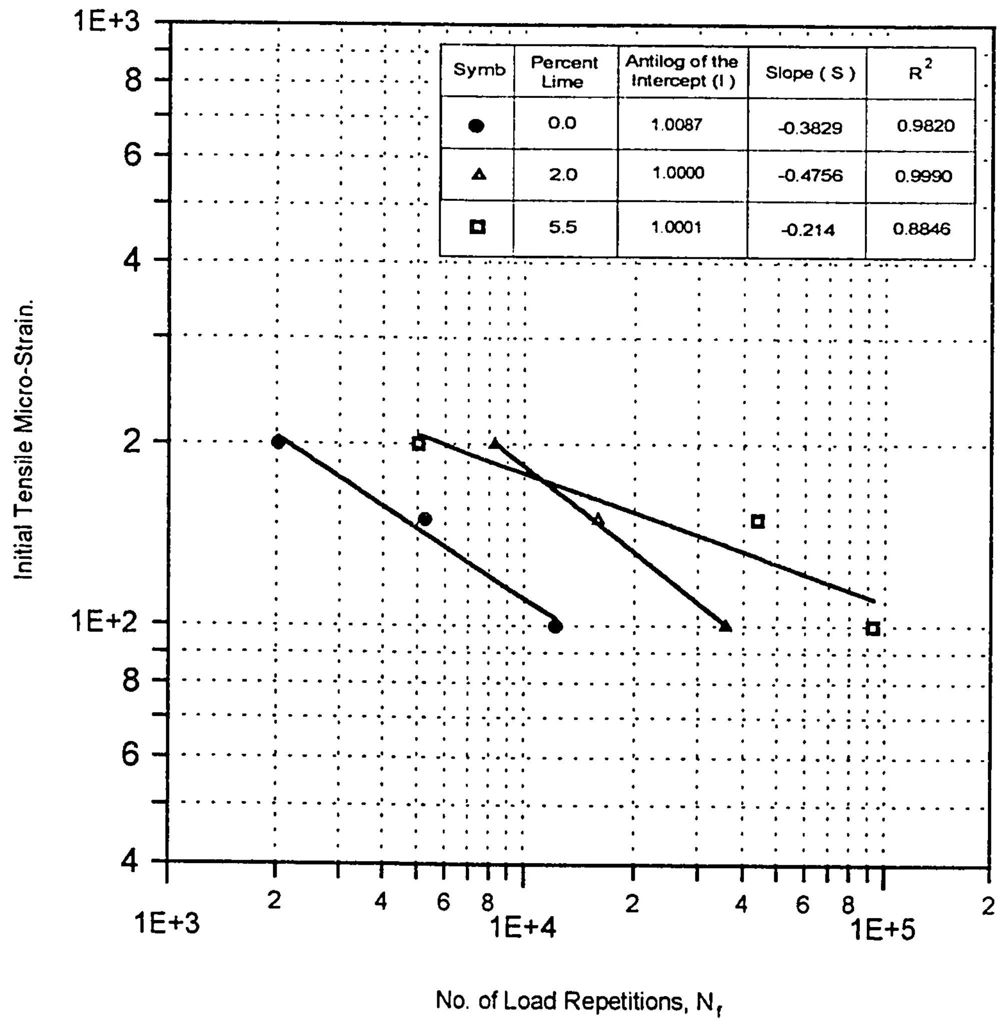


Fig. 4.20 : Fatigue Curve for Lime Modified Base Coarse Mixes at 45° C.

Table 4.3 : Results of Fatigue Curves

Mix Type	Temparature					
	45^0 C			60^0 C		
	Slope	Antilog of Intercept	R^2	Slope	Antilog of Intercept	R^2
G1 (WC)	-0.5583	1.054	0.999	-0.3619	1.005	0.964
G1H-2%	-0.8335	2.064	0.994	-0.5478	1.030	0.999
G1L-2%	-0.3930	1.017	0.963	-0.3052	1.004	0.986
G1H-5.5%	-0.5788	1.097	0.982	-0.2294	1.003	0.955
G1L-5.5%	-0.2576	1.017	0.951	-0.1368	1.001	0.860
G2 (BC)	-0.3829	1.0087	0.982	-0.3903	1.0086	0.947
G2H-2%	-0.5943	1.0596	0.981	-0.3844	1.009	0.989
G2L-2%	-0.4918	1.0406	0.994	-0.5298	1.058	0.970
G2H-5.5%	-0.4823	1.026	0.995	-0.6993	1.277	0.978
G2L-5.5%	-0.1589	1.002	0.885	-0.3650	1.012	0.972

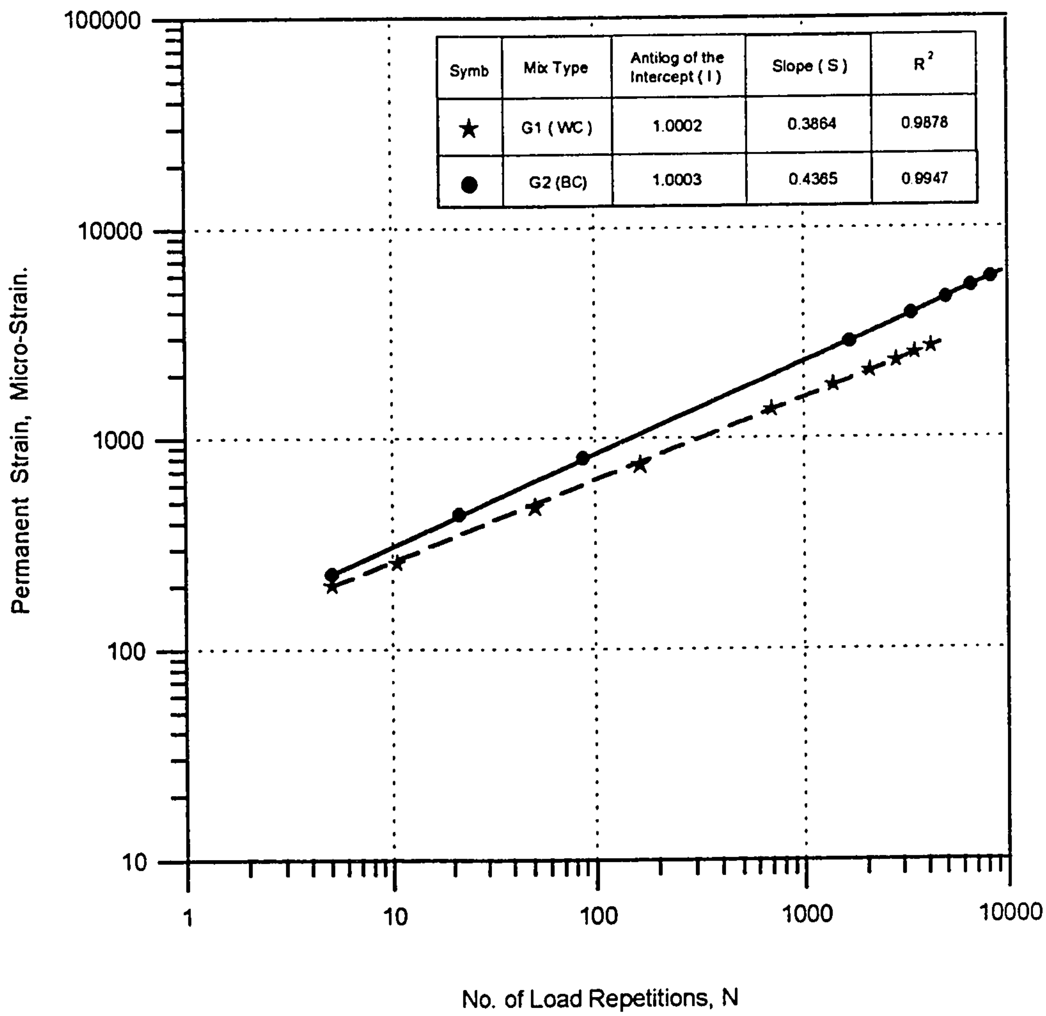


Fig. 4.21 : Rutting Curves for Control mix at 45°C.

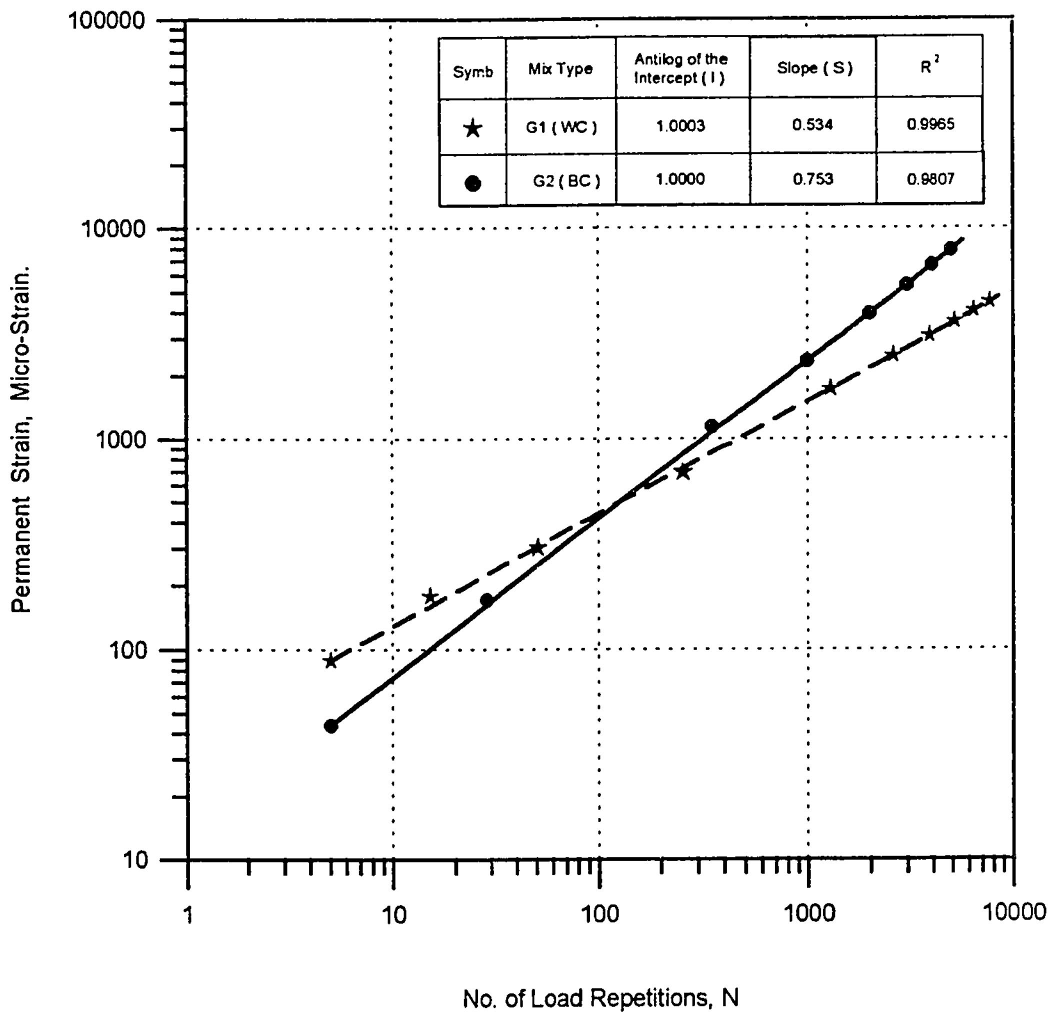


Fig. 4.22 : Rutting Curves for Control Mix at 60°C.

Table 4.4 : Results of Rutting Curves

Mix Type	Temparature					
	45° C			60° C		
	Slope	Antilog of Intercept	R^2	Slope	Antilog of Intercept	R^2
G1 (WC)	0.3864	1.001	0.987	0.534	1.0002	0.981
G1H-2%	0.4097	1.002	0.994	0.485	1.0001	0.989
G1L-2%	0.3408	1.003	0.988	0.636	1.0011	0.971
G1H-5.5%	0.3465	1.005	0.964	0.5735	1.0001	0.991
G1L-5.5%	0.096	1.0016	0.971	0.1779	1.0011	0.971
G2 (BC)	0.4365	1.0002	0.971	0.752	1.0001	0.996
G2H-2%	0.295	1.0001	0.986	0.334	1.0017	0.986
G2L-2%	0.247	1.0006	0.996	0.313	1.002	0.991
G2H-5.5%	0.3653	1.0003	0.992	0.345	1.002	0.976
G2L-5.5%	0.0962	1.0004	0.995	0.324	1.0002	0.986

4-10 Summary

This chapter discusses the behaviour of control mix as well as modified mixtures designed by Marshall method, under dynamic and static loading. The resilient modulus test at 45°C, and Marshall stability, Split tensile strength, Static creep tests were conducted at 60° C. And finally Fatigue and Permanent deformation characterization tests were done at two high temperatures occurring mostly in gulf region. i.e., 45°C and 60°C. The test results indicates that certain percentages of both lime and hedmanite are effective in improving the resilient modulus of the mixtures. While the Marshall stability loss and split tensile strength loss is higher in hedmanite modified mixes than that of lime modified mixtures. Creep test shows no specific trend for the modified mixes with different percentages of lime or hedmanite for wearing coarse (WC) and base coarse (BC) gradations. Fatigue and permanent deformation tests reveals that both lime and hedmanite modified mixes has improved the fatigue life and permanent deformation of the wearing coarse and base coarse mixes as compared to the control mix. It was found that lime modified mixes shows better resistance to fatigue and rutting than the hedmanite modified mixes. These results are supported by the statistical analysis performed on the tests data.

Chapter 5

Conclusions and Recommendations

The primary objective of this research was to evaluate the effect of the incorporation of a newly available material "Hedmanite" as a filler substitute in the asphalt concrete (AC) mixtures, and to compare its behavior with the mixes having crushed stone or lime as filler. Mixes were designed by Marshall method and evaluated for fundamental engineering properties such as Resilient modulus, Stability, Split tensile strength, Creep, Fatigue and Permanent deformation. In order to come up with the results, showing the suitability of this material as a filler in the local AC mixes.

On the basis of tests performed, and under the applied test conditions. The following conclusions were drawn.

5-1 Conclusions

Based on the laboratory evaluation results of this research, following conclusions were drawn:

1. Filler content has a significant affect on the asphalt concrete mix properties. As the incorporation of different fillers has resulted in varying fundamental properties.

   i) Increase in the percentage of hedmanite as a filler substitute, has resulted in the increase in MR for both wearing coarse and base coarse mixes.

   ii) A gain in stability loss is observed in case of 4% lime as filler for the wearing coarse mix.

   iii) Reduction in split tensile strength has occurred as the percentage of hedmanite is increased in both wearing coarse and base coarse mixes.

1. The MR values of hedmanite modified mixes was found to increase with the increase in percentage of hedmanite (added as a filler substitute), for both wearing coarse and base coarse mixes. Whereas, In lime modified mixes with 4% addition of lime (In wearing coarse) and 2% (In base coarse), the resilient moduli value has increased by 20% & 5% for WC & BC respectively, after that the moduli decreases.

2. Lime modified mixes shows high resistance to the effect of water than that of hedmanite modified mixes. Since, In comparison to the control mix, there is 15% reduction in the loss of tensile strength and 28% reduction in stability loss, When 4% lime is added as a filler substitute in the WC mix. Similarly, at 2% lime in the BC mix, the stability loss has decreased by 22% and tensile strength loss is decreased by 12%.

3. Substitution of Hedmanite as filler, has resulted in an increase in the loss of stability and split tensile strength of the mixes.

4. The analysis of creep data indicates no trend in the results, for the various mixes prepared using different percentages of fillers. Therefore, It can be concluded that for the MOC gradation used, the 2.5 inch height of the specimen, specified by Shell group for creep test is insufficient to show the correct behavior of mixtures under static loading.

5. For the given level of applied tensile strain, an increase in the percentage of Lime or Hedmanite as filler, has resulted in an increase in the fatigue life of the mixes.

6. The fatigue life of both wearing coarse and base coarse mixes was found to decrease with the increase in the testing temperature from 45° C to 60° C.

7. Permanent deformation (Rutting) was found to increase with the increase in testing temperature.

8. Lime modified mixes shows better resistance to rutting then the hedmanite modified mixes or the control mix.

9. Based on the results of lime modified mixes, it may be concluded that good quality mixes can be developed using lime as filler with the local aggregates.

In view of the above facts, and under the testing conditions applied in this study for Stability loss, Resilient modulus (MR), Indirect tensile strength, Fatigue and

Permanent deformation tests . It can be concluded that high quality asphalt concrete mixes can be prepared using lime as a filler than the material hedmanite for the local aggregate. Since stability loss and strength loss is the major problem associated with local aggregates and hedmanite modified mixes shows high stability and strength loss. Therefore its use may not provide the expected benefits.

5 -2 Recommendations

In view of the above conclusions, the following recommendations are made:

1. The results of Hedmanite modified mixes are not very conclusive, therefore further investigation can be made. For example, Hedmanite can be mixed with asphalt to produced a modified asphalt cement. Then the mixes prepared using this modified asphalt cement can be evaluated to see the effect of hedmanite when mixed in asphalt.

2. In this study the hedmanite and lime modified mixes were tested at high temperature range 45° C and 60° C. Therefore, tests at low temperature can be performed in order to determine the effect of these materials as filler under low temperature conditions.

3. For the Unconfined Static creep test, specimens with height greater than 2.5 inches should be used for the local gradations. For example, 8 inch high by 4 inch diameter samples should be tested.

APPENDIX-A

CENTRAL ANALYTICAL AND MATERIALS CHARACTERIZATION LABORATORIES

ANALYSIS REPORT

REQUEST FORM NO	: MCL-527	DATE	: 10-10-94
PROPONENT	: Dr. H.A. Ibrahim	PROJECT	: MS Thesis
SECTION (CAL/MCL)	: SEM LAB	DEPARTMENT	: Civil Eng. Dept.
PROJECT TO BE CHARGED	: MS Thesis	COMP'D DATE	: 11-10-94
ANALYSIS GROUP	: A.I. Mohd	CHECKED BY	: Hani Tawancy Dr. H.M. Tawancy.
INSTRUMENT USED	: SEM/EDS		

BACKGROUND :

Received one (1) stone powder sample for SEM/EDS analysis. The objective was to observe the microstructure and determine the chemical composition.

EXPERIMENTAL PROCEDURE :

A small amount of the sample was spread on a double sided tape stucked to a stub and coated with thin film of gold to prevent charging while viewing and filming. 20KV accelerating voltage was used in the observations.

RESULTS :

Micrographs were taken at 1000X, 5000X and 10000X magnification with each revealing the microscopic structure of the sample. Elemental composition were determined with the EDS and the relative weight percentages were calculated using the Standardless semi-quantitative analysis program with ZAF correction.

CENTRAL ANALYTICAL AND MATERIALS CHARACTERIZATION LABORATORIES

ANALYSIS REPORT

REQUEST FORM NO	: MCL-527	DATE	: 3-10-94
PROPONENT	: Dr H. Ibrahim (Advisor)	PROJECT	: MSc Thesis
SECTION (CAL/MCL)	: X-ray Laboratory	DEPARTMENT:	Civil Engineering
PROJECT TO BE CHARGED	: MSc Thesis	COMP'D DATE:	17-10-94.
ANALYSIS GROUP	: Dr. J. Shirokoff	CHECKED BY	:
INSTRUMENT USED	: X-ray Diffractometer		

SUMMARY AND BACKGROUND

One Hedmanite mineral samples in powdered form was received for analysis by XRD. The objective of this analysis was to identify the different chemical/mineralogical phases (compounds) present in the sample and their relative amounts.

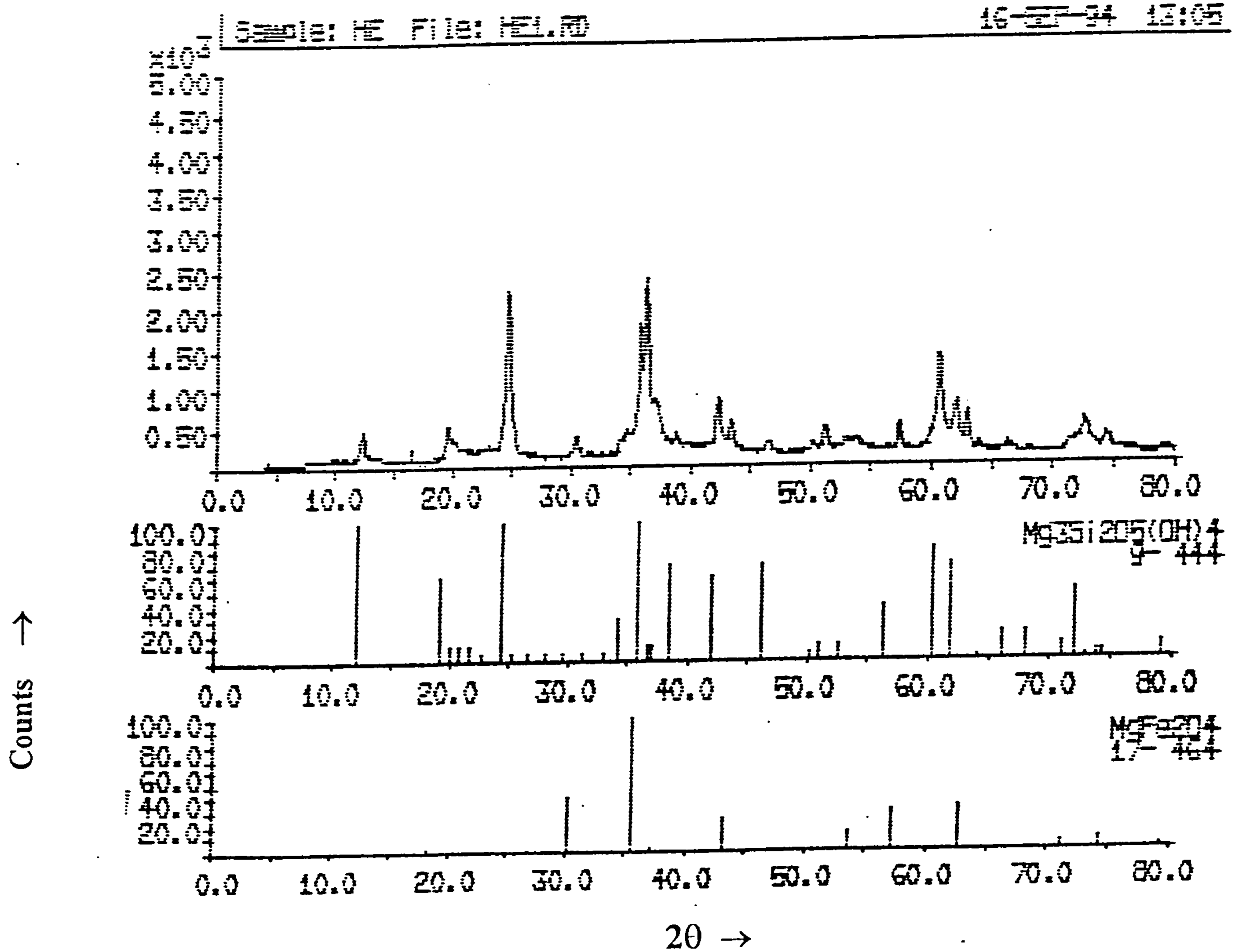
EXPERIMENTAL PROCEDURE :

The samples submitted were of dry powdery consistency. In this powdered form, many grains come into orientation and the quality of the diffraction pattern is greatly improved. The diffraction pattern was generated by a theta-2theta scanning diffractometer.

The operating conditions of XRD analysis were :
Cu broad focus tube at 45 kv and 30 mA.
Auto divergence slit, no scatter slit and receiving slit = 0.2mm.
A monochromator was used.
Scanning speed and interval of data collection was 0.01 degree two theta/sec.
Angle scanned : 4 to 80 degree two theta.

RESULTS AND DISCUSSIONS :

The summary of the results is given below. The number in the middle of the table represents the approximate weight fraction of the phase in percentage. The phase identifications process involves calculating the "Most Likely Match Score" for a given phase based on peak intensity and peak position when compared to a database of standard phases while the weight fraction is calculated by comparing the intensity of the most intense peaks of that phase with standards. The intensities of the diffraction peaks are mainly governed by the amount of material, along with other factors that play an important role. Thus it should be noted that the X-ray diffraction technique is a semi-quantitative analysis technique primarily used on crystalline materials for determining the weight fractions of crystalline phases down to about 1 wt%. The number on the right side of the table is the corresponding JCPDS phase number. The comparision of diffraction pattern with the standard diffraction pattern for different phases established by JCPDS have been attached.



X-ray Diffraction Pattern

340-4

APPENDIX-B

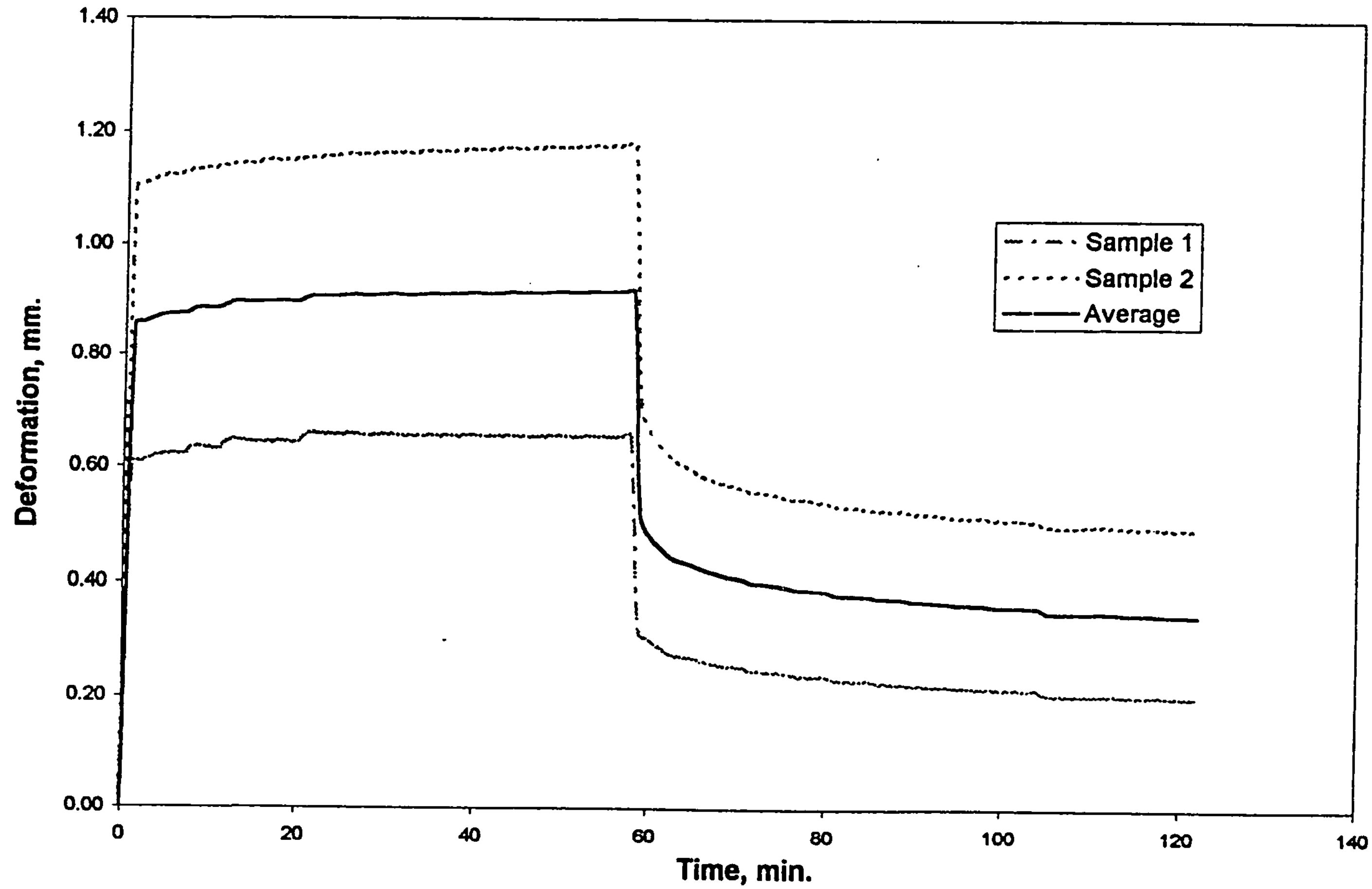


Fig. B1 : Creep Curve for G1L-1%

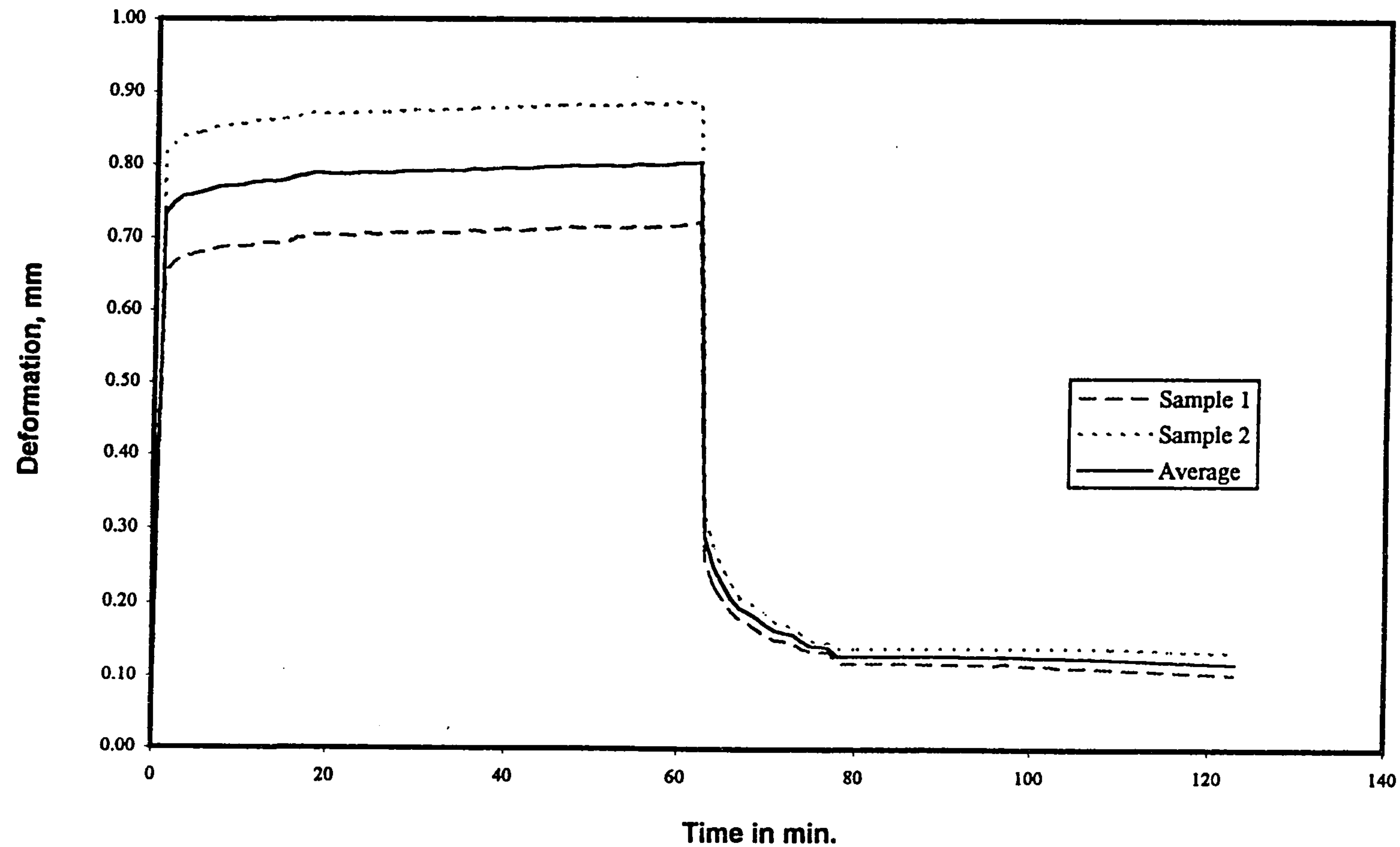


Fig. B2 : Creep Curve for G1L-2%

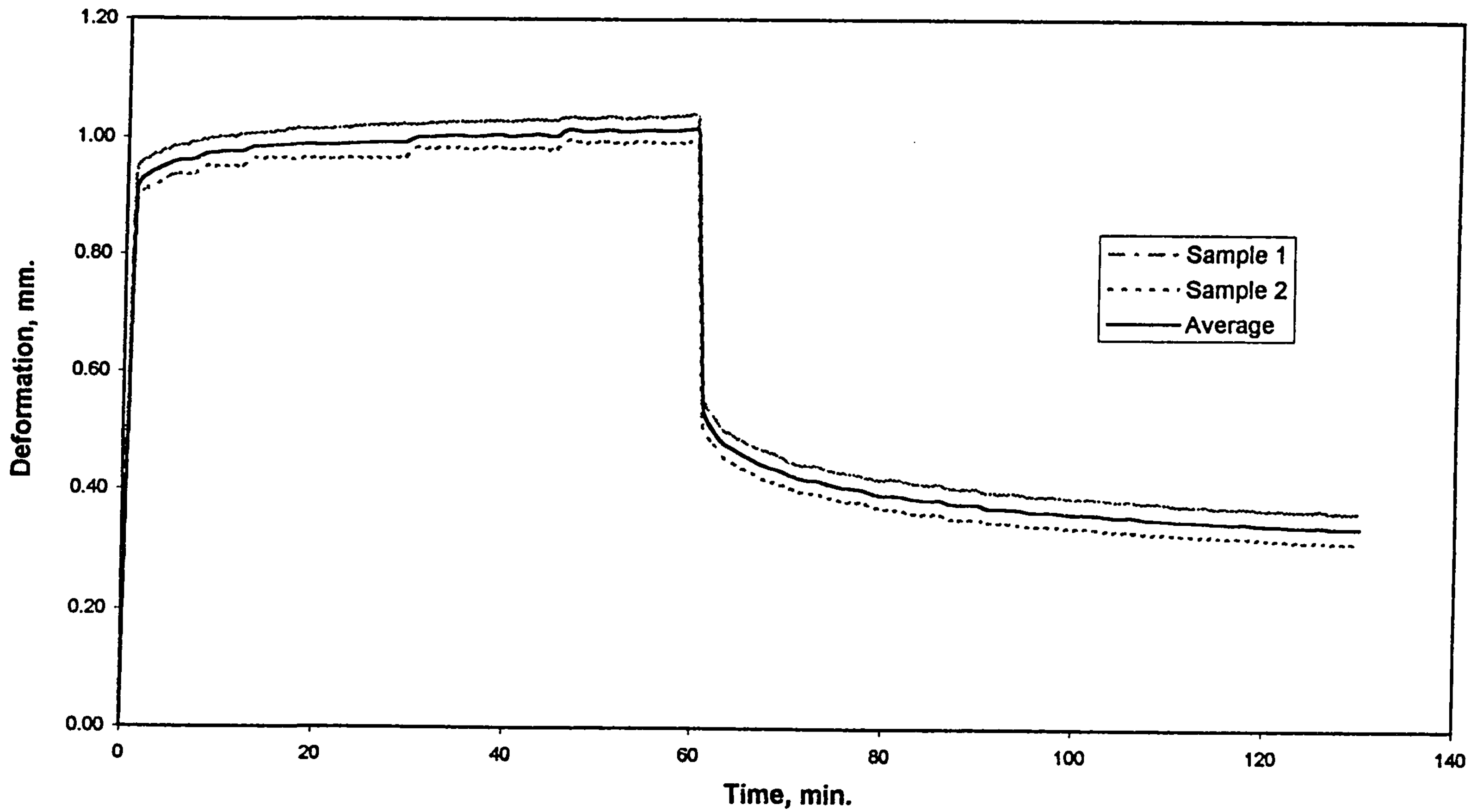


Fig. B3 : Creep Curve for G1L-4%

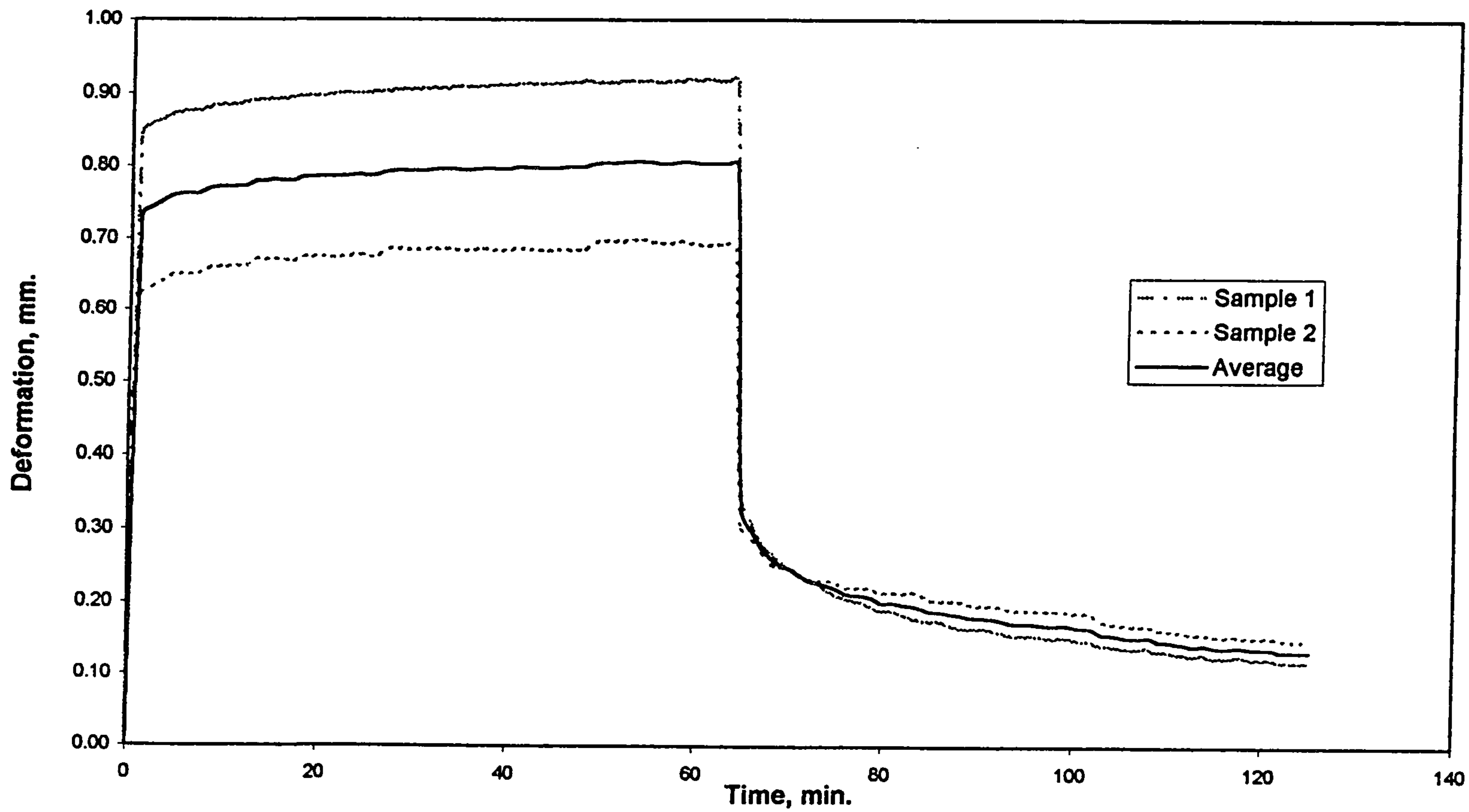


Fig. B4 : Creep Curve for G1L-5.5%

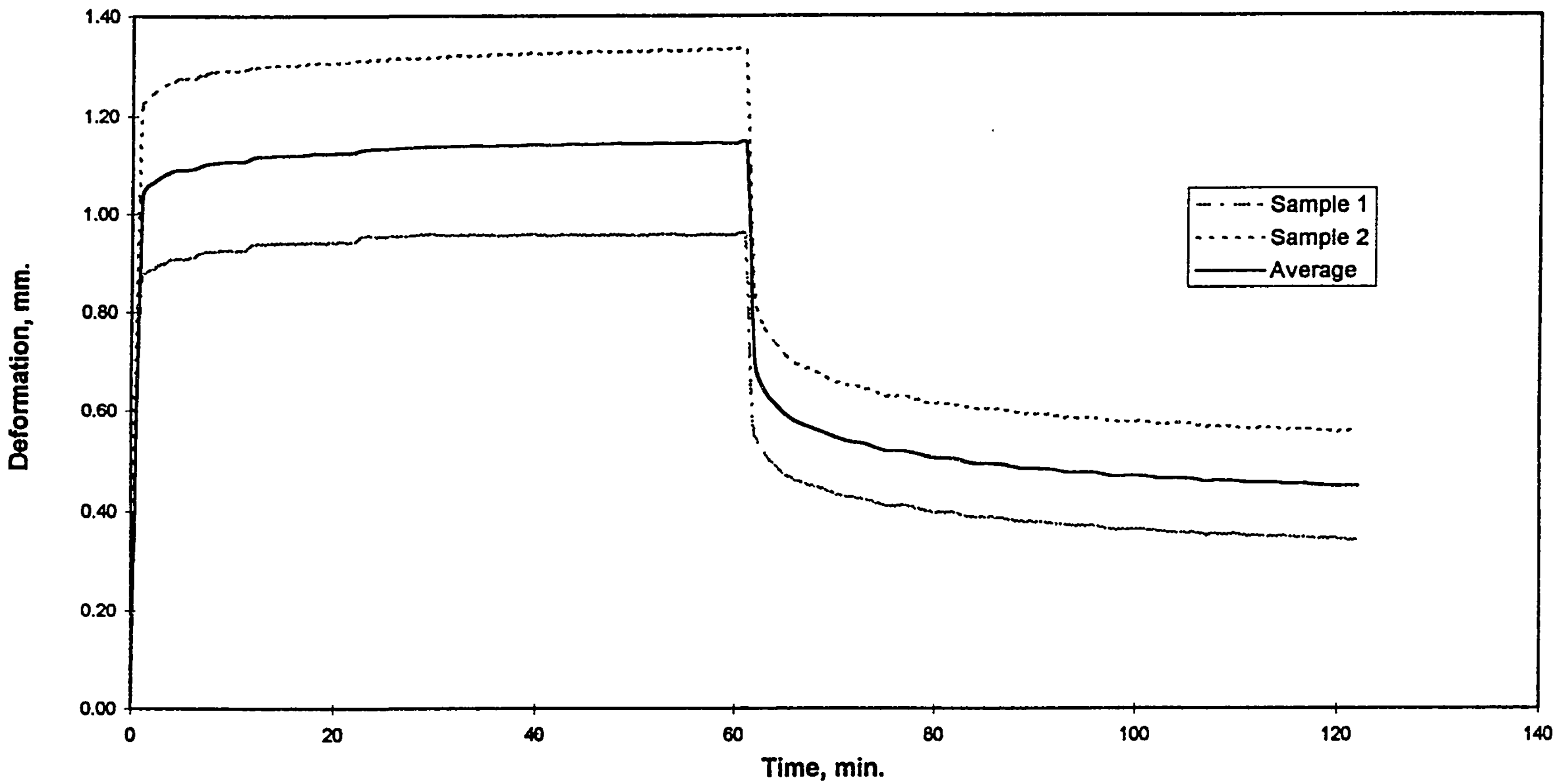


Fig. B5 : Creep Curve for G1H-1%

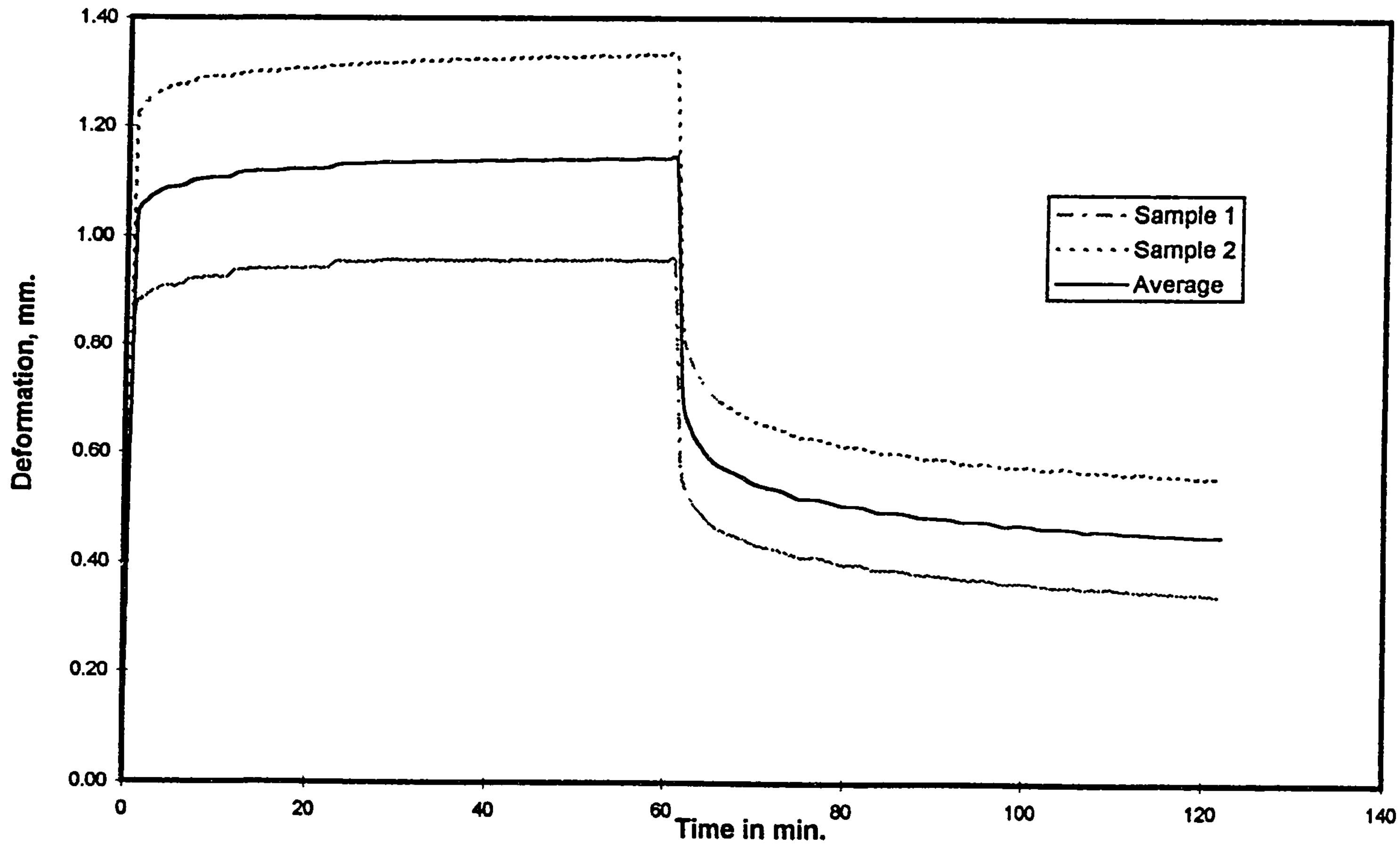


Fig.B6 : Creep Curve for G1H-2%

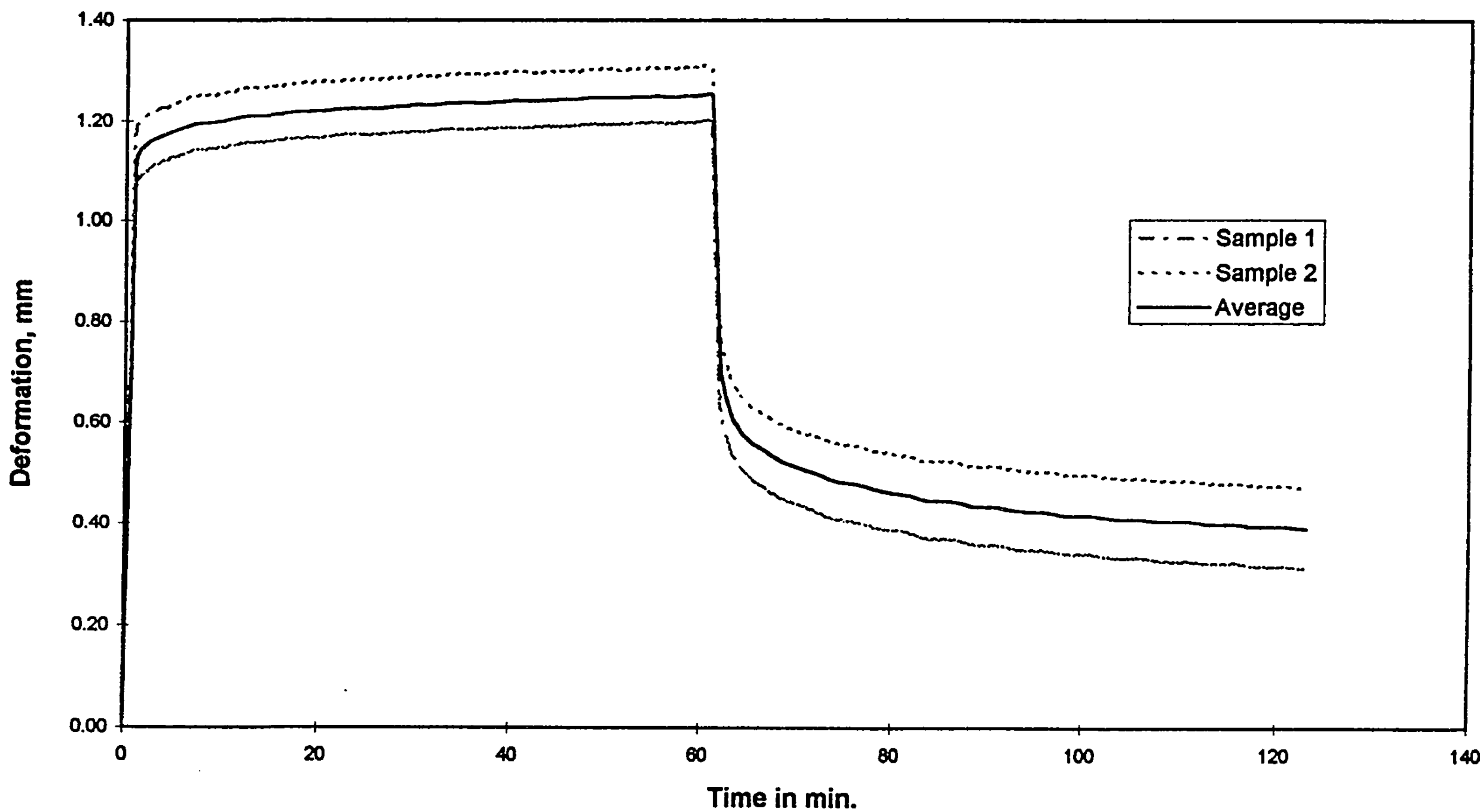


Fig. B7 : Creep Curve for G1H-4%

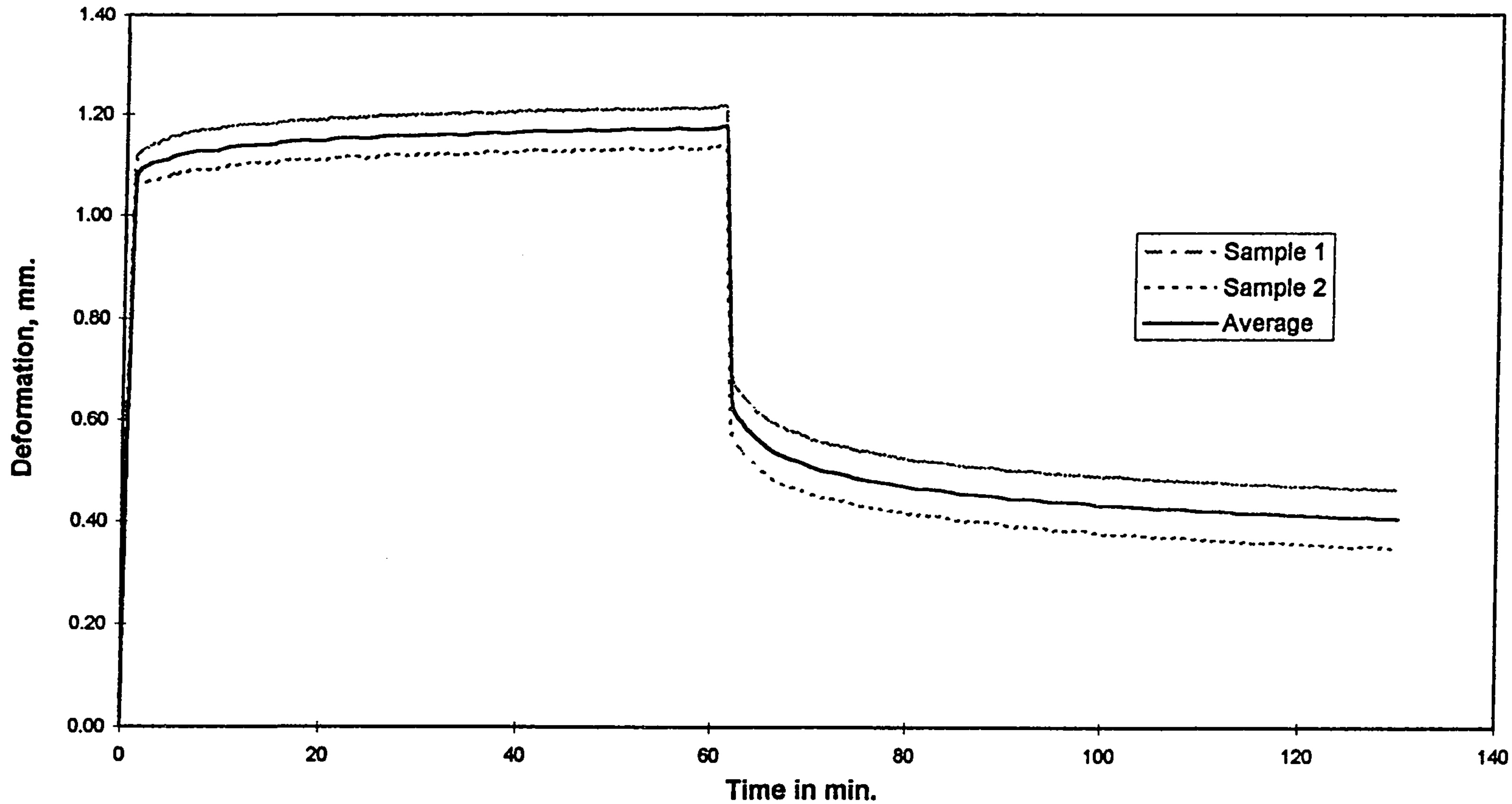


Fig. B8 : Creep Curve for G1H-5.5%

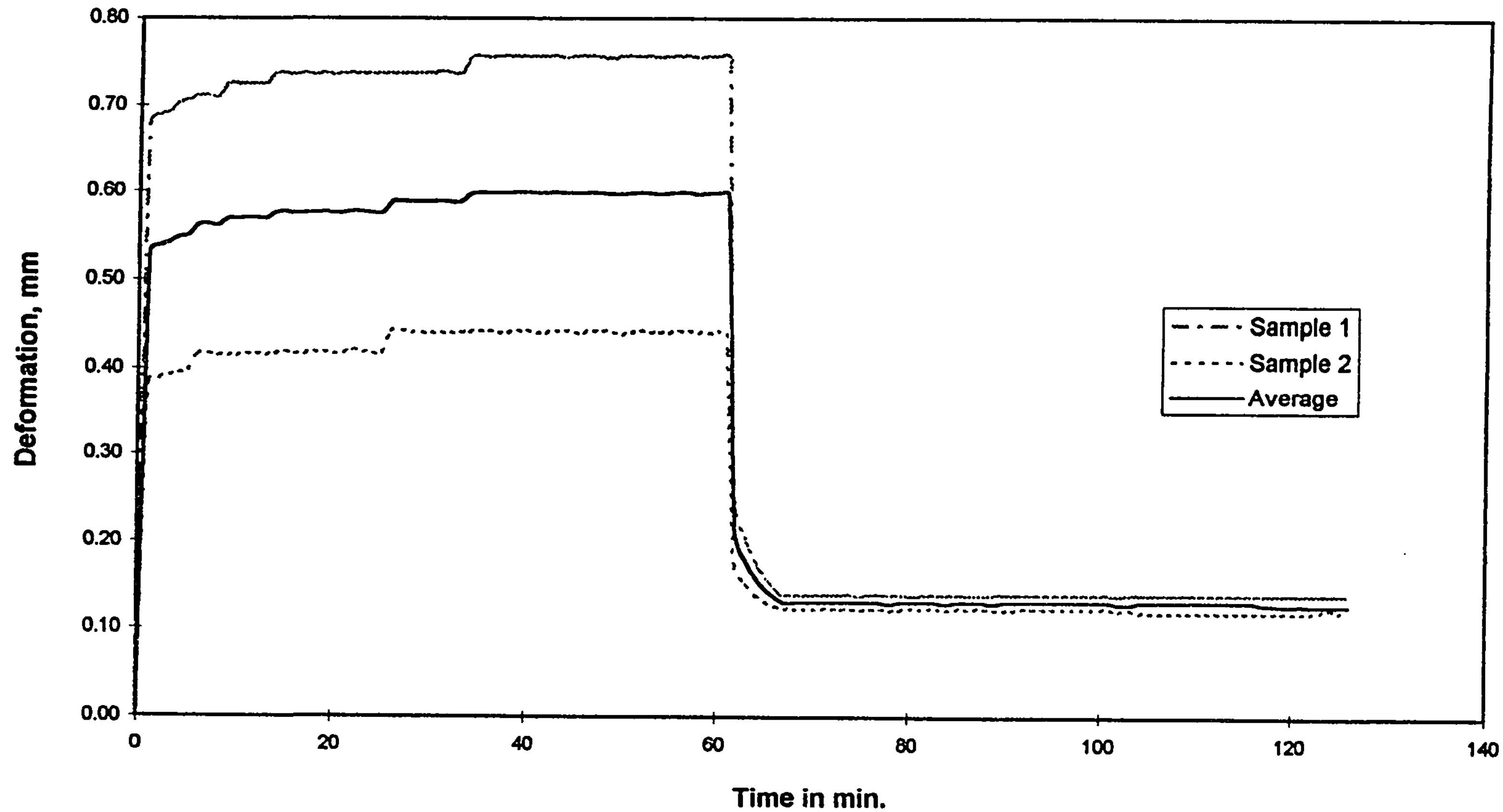


Fig. B9 : Creep Curve for G2L-1%

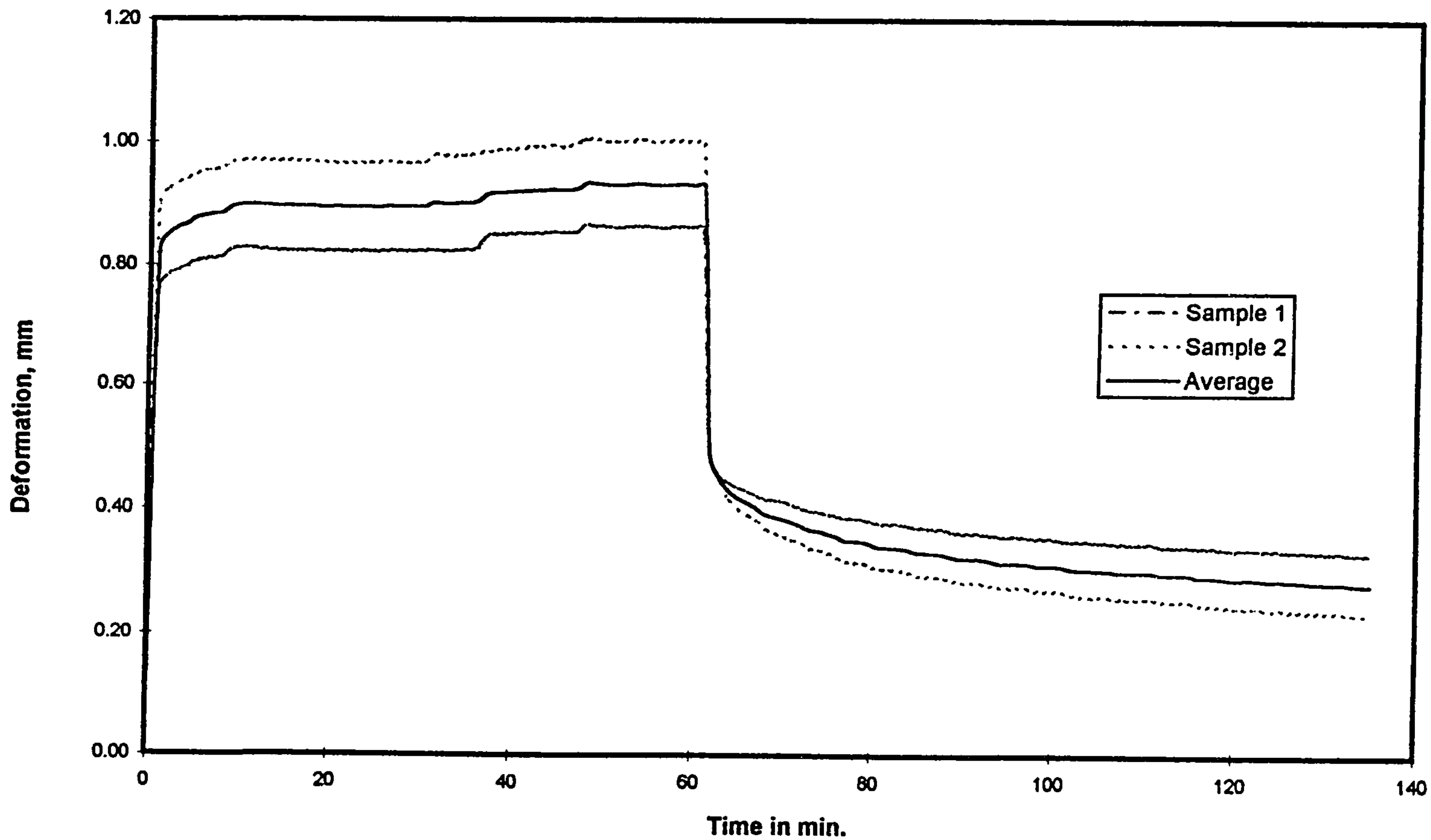


Fig. B10 : Creep Curve for G2L-2%

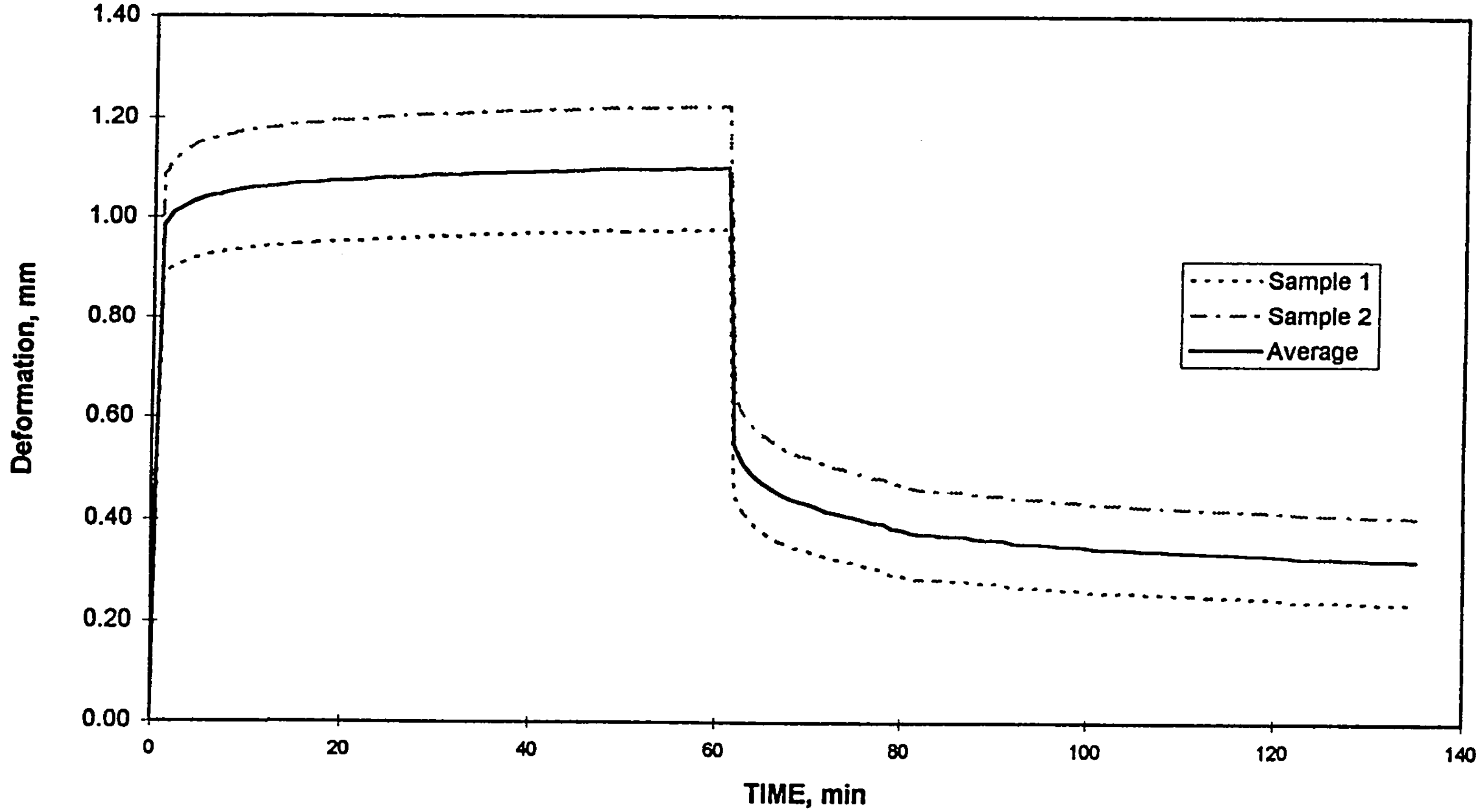


Fig. B11 : Creep Curve for G2L-4%

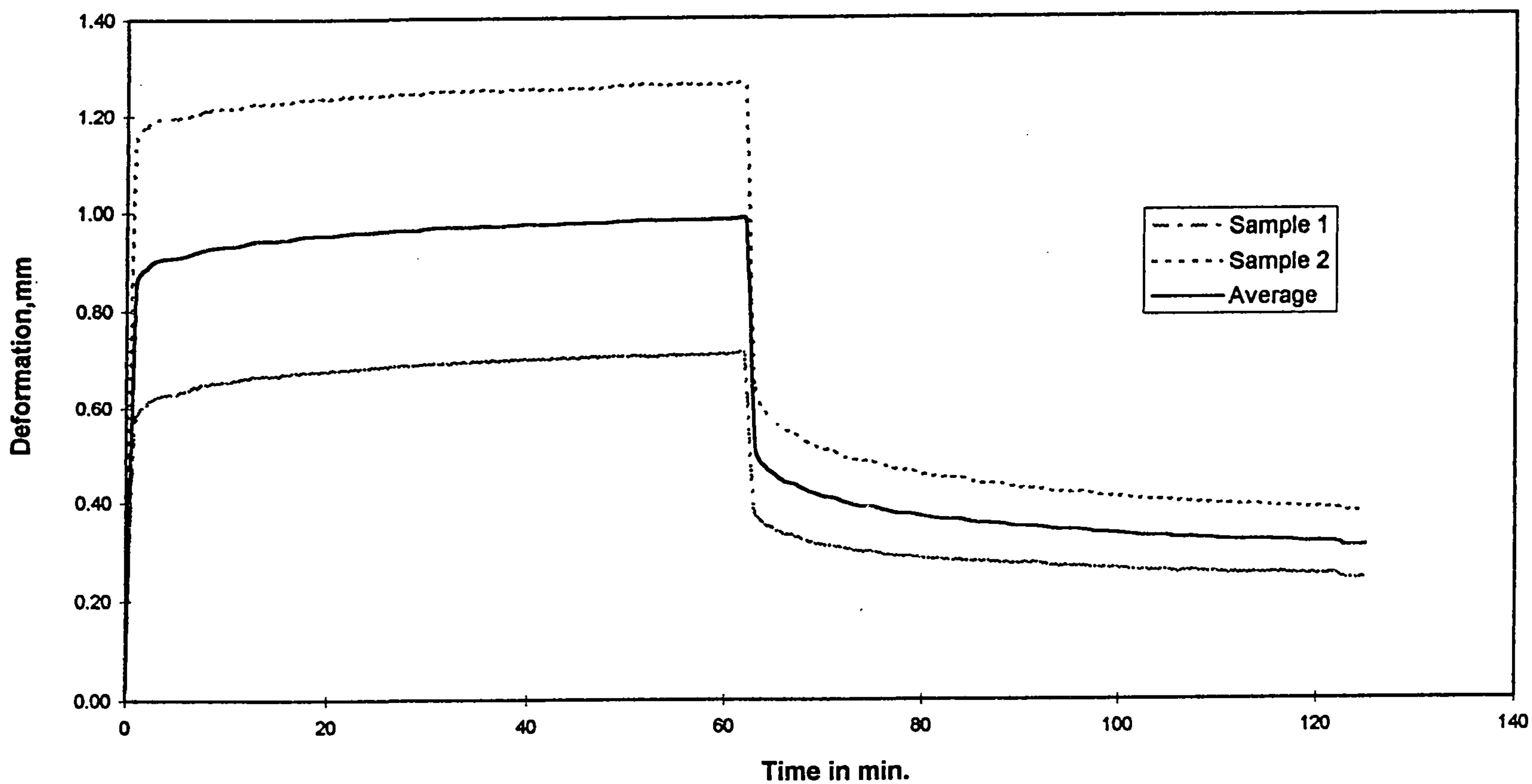


Fig. B12 : Creep Curve for G2L-5.5%

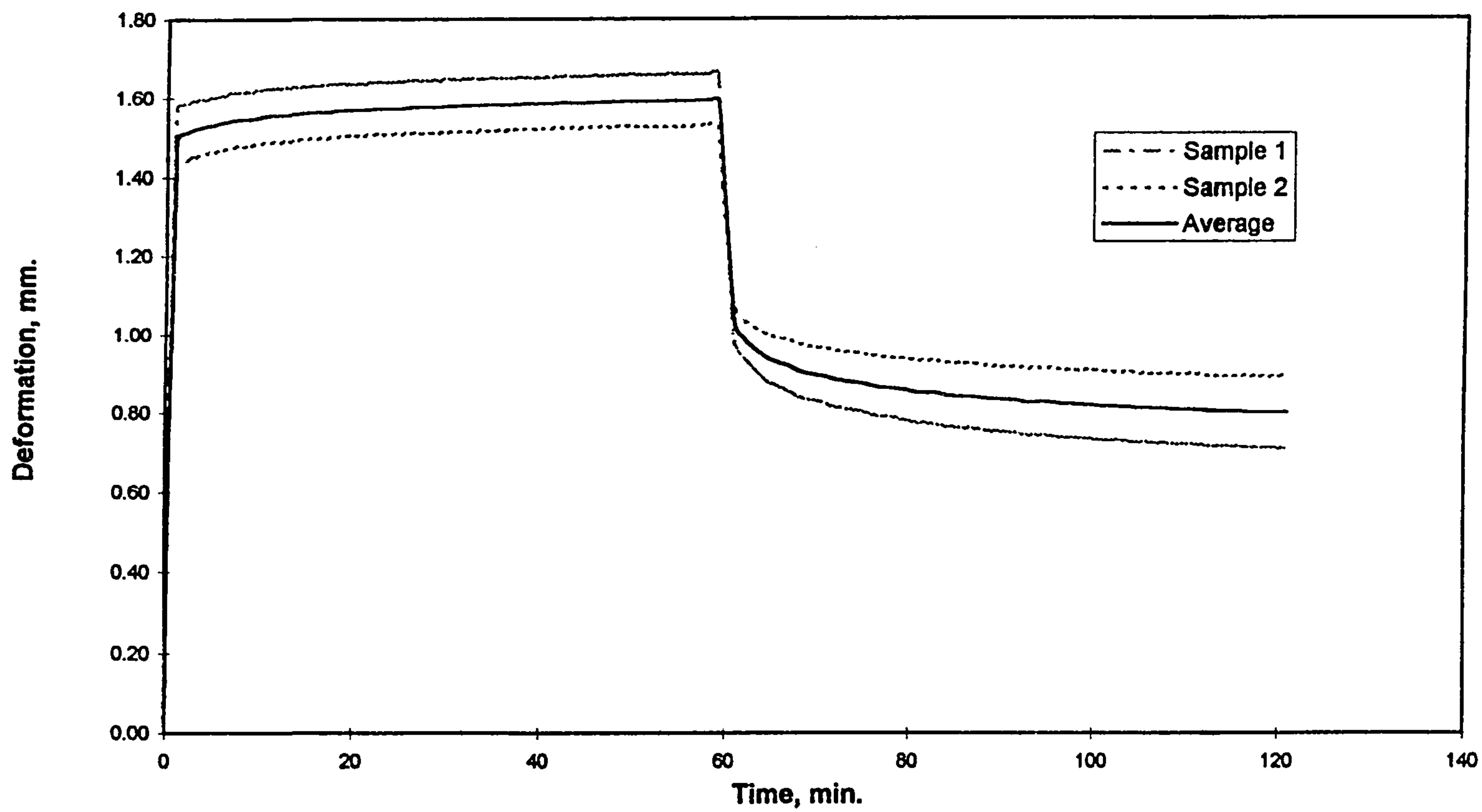


Fig. B13 : Creep Curve for G2H-1%

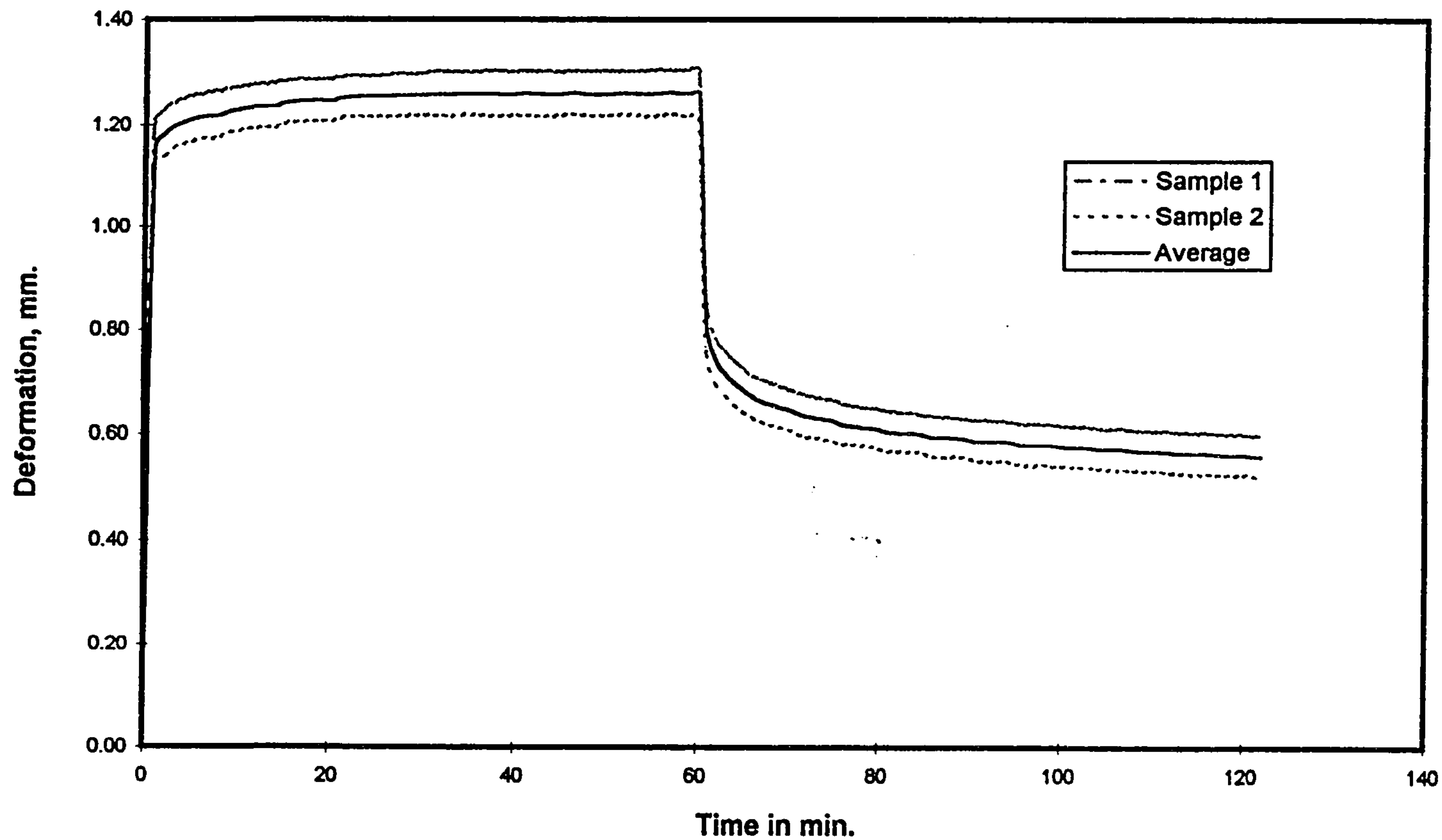


Fig. B14 : Creep Curve for G2H-2%

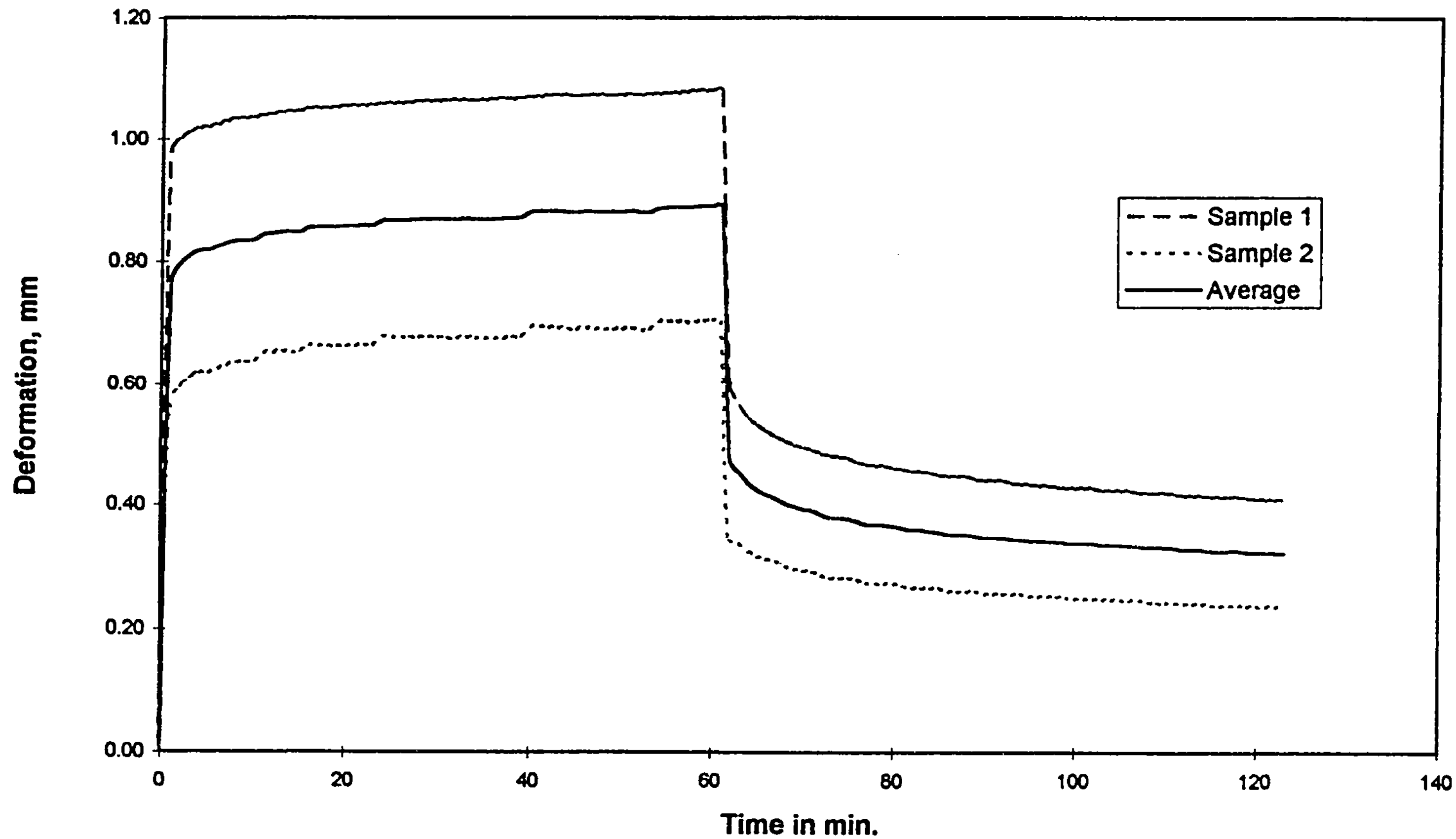


Fig. B15 : Creep Curve for G2H-4%

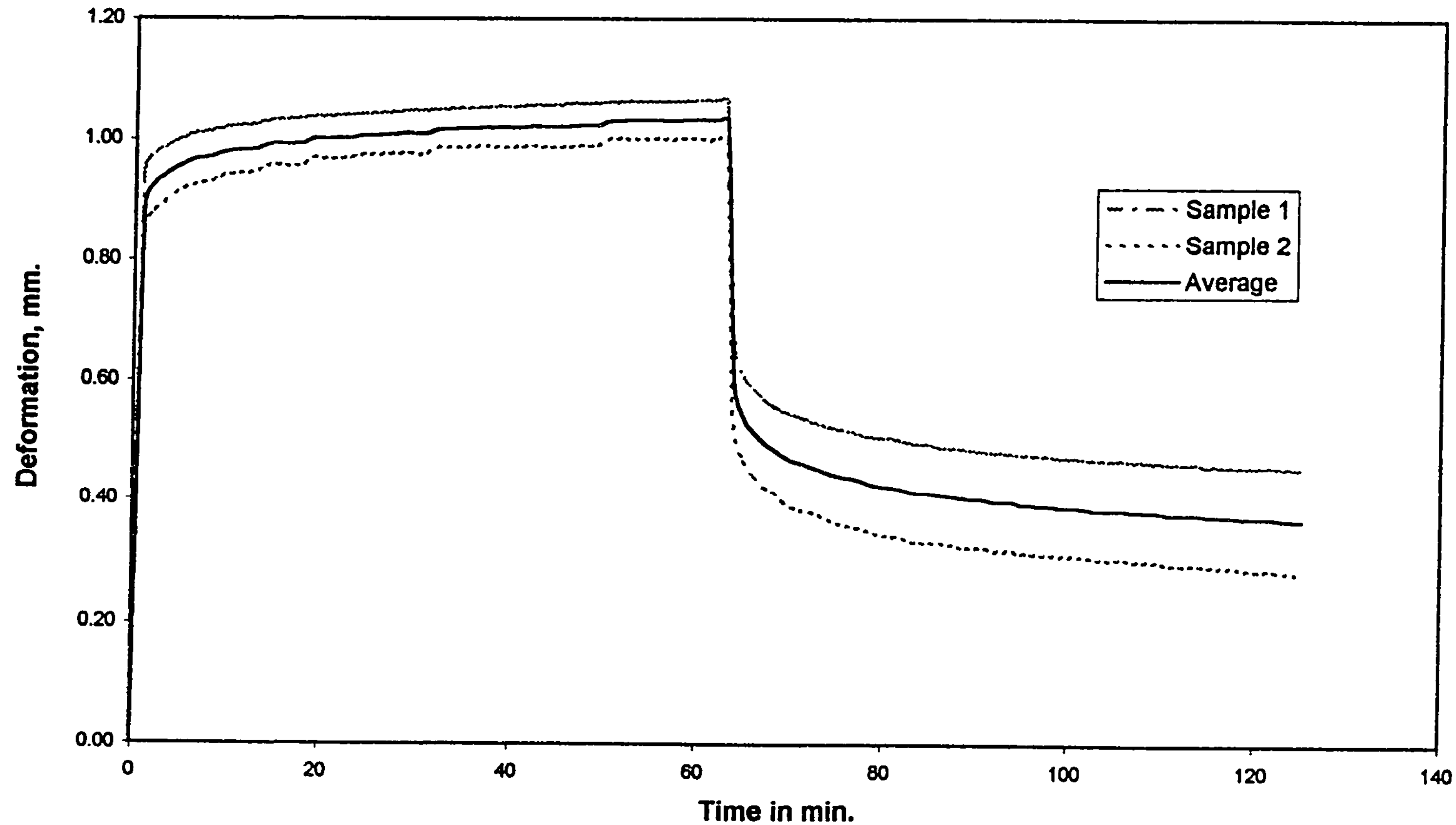


Fig. B16 : Creep Curve for G2H-5.5%

APPENDIX-C

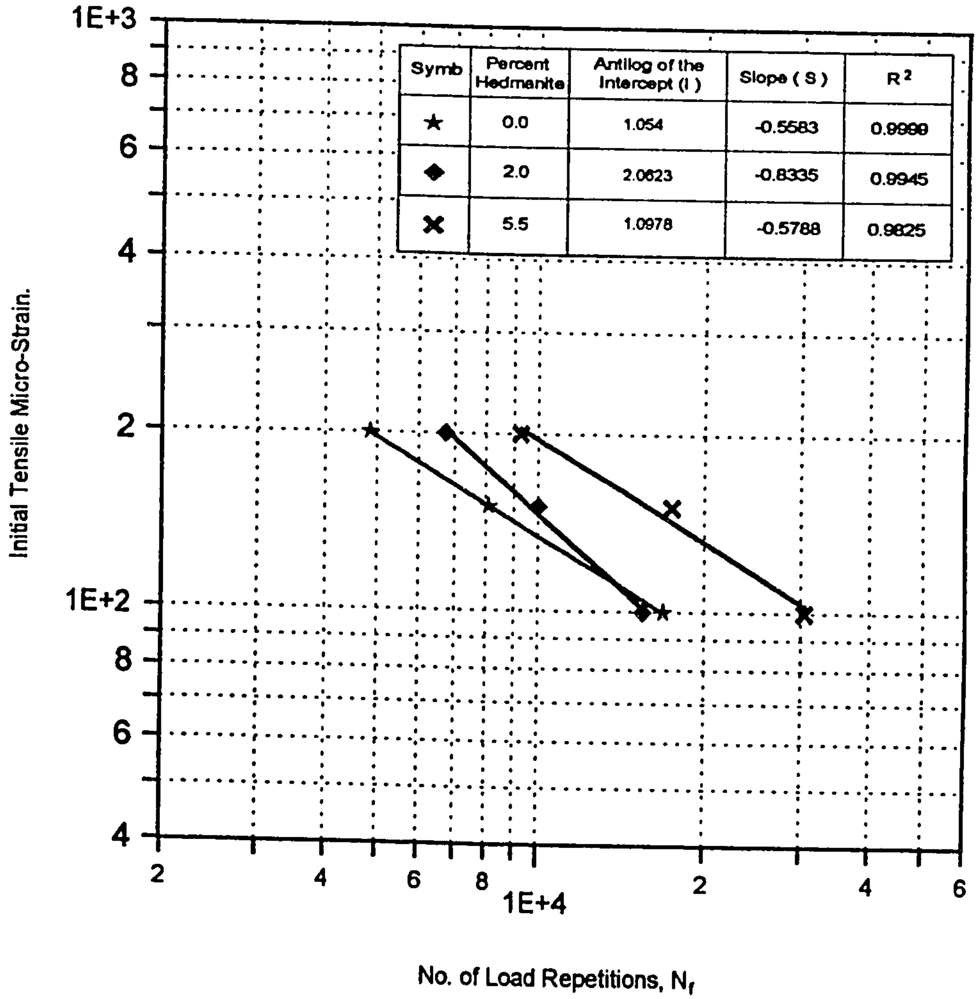


Fig. C1 : Fatigue Curve for Hedmanite Modified Wearing Coarse Mixes at 45°C.

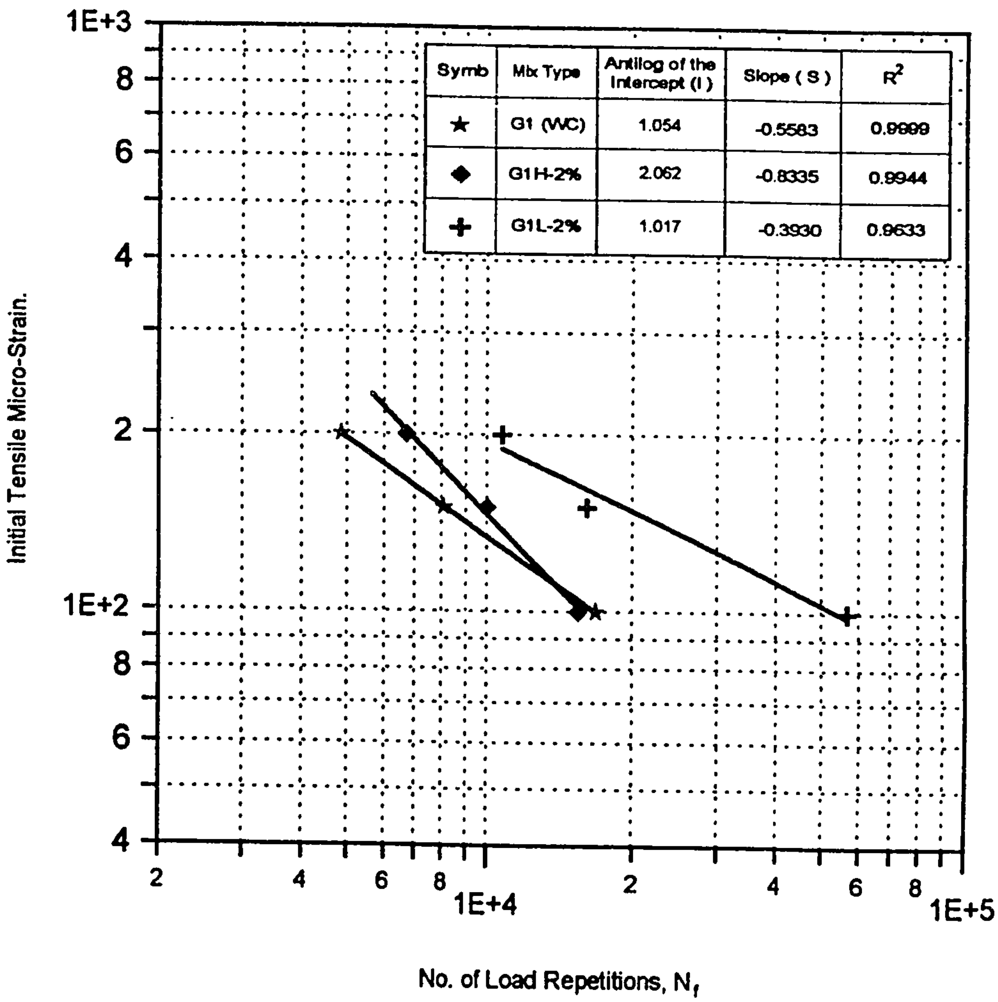


Fig. C2 : Fatigue Curves Comparision for 2-Percent Lime and Hedmanite Modified Wearing Coarse Mixes at 45° C.

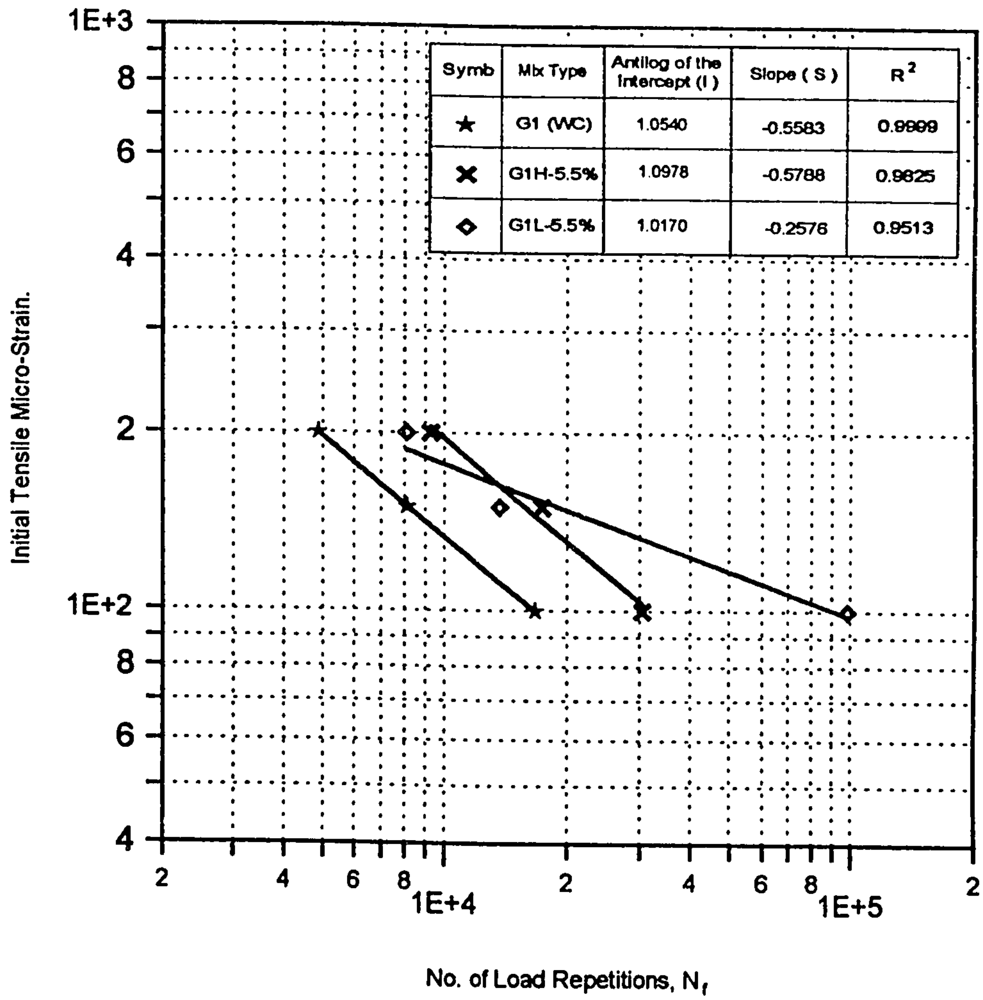
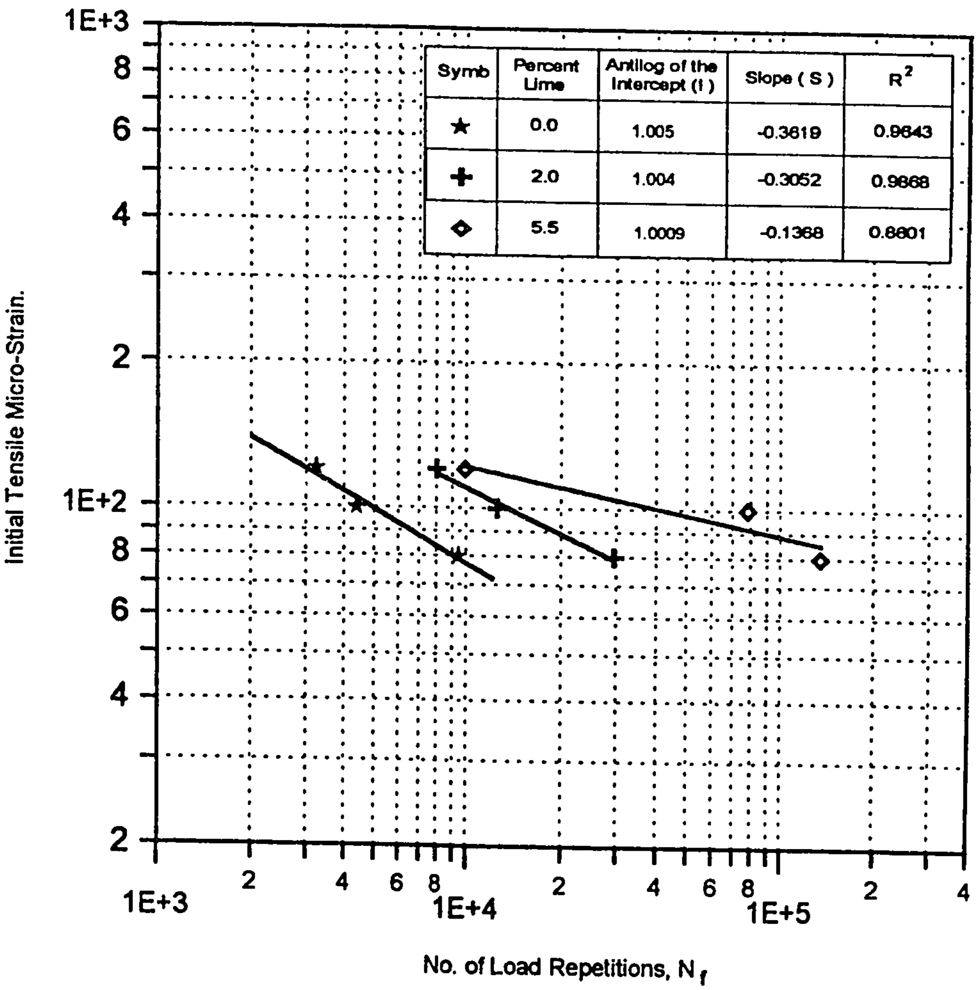
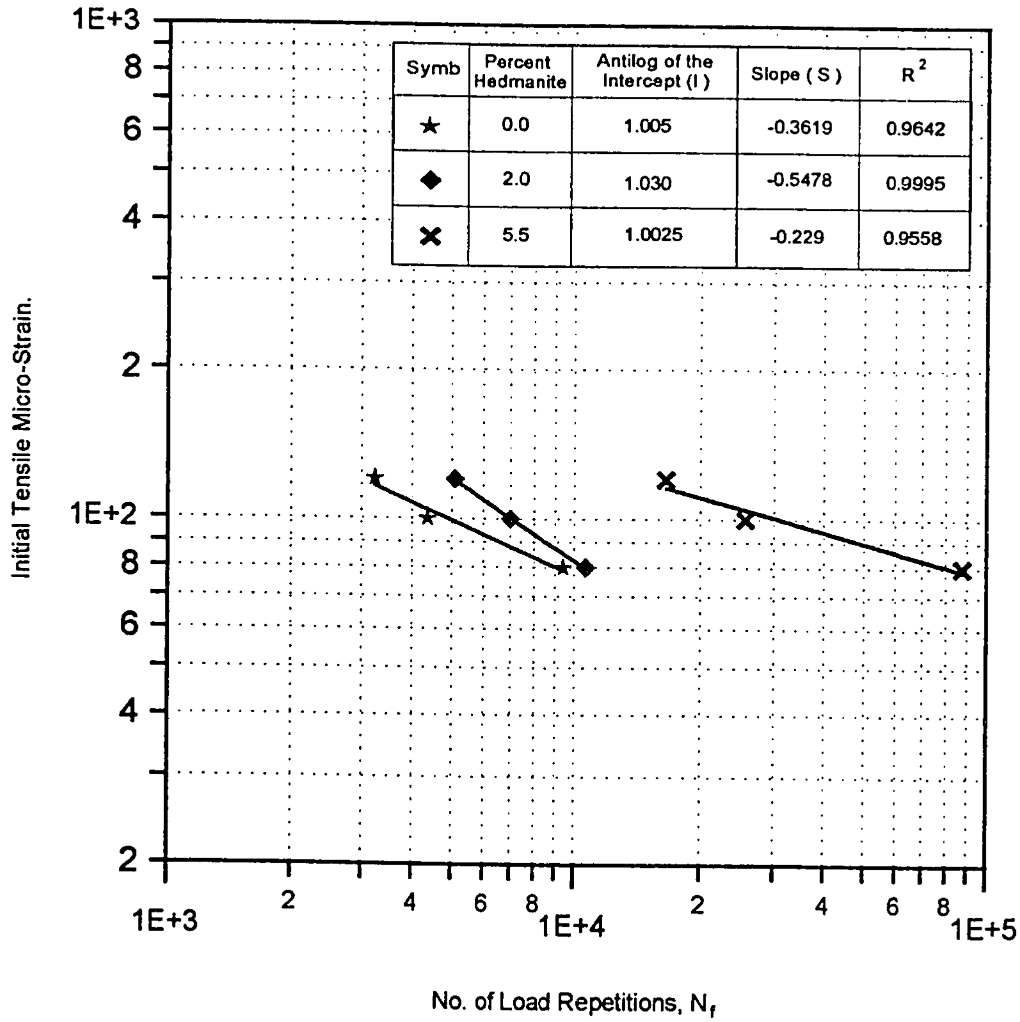


Fig. C3 : Fatigue Curves Comparision for 5.5-Percent Lime and Hedmanite Modified Wearing Coarse Mixes at 45°C.



Symb	Percent Lime	Antilog of the Intercept (I)	Slope (S)	R^2
★	0.0	1.005	-0.3619	0.9643
+	2.0	1.004	-0.3052	0.9868
◇	5.5	1.0009	-0.1368	0.8801

Fig. C4 : Fatigue Curve for Lime Modified Wearing Coarse Mixes at 60 °C.



Symb	Percent Hedmanite	Antilog of the Intercept (I)	Slope (S)	R^2
★	0.0	1.005	-0.3619	0.9642
◆	2.0	1.030	-0.5478	0.9995
✖	5.5	1.0025	-0.229	0.9558

Fig. C5 : Fatigue Curve for Hedmanite Modified Wearing Coarse Mixes at 60 °C.

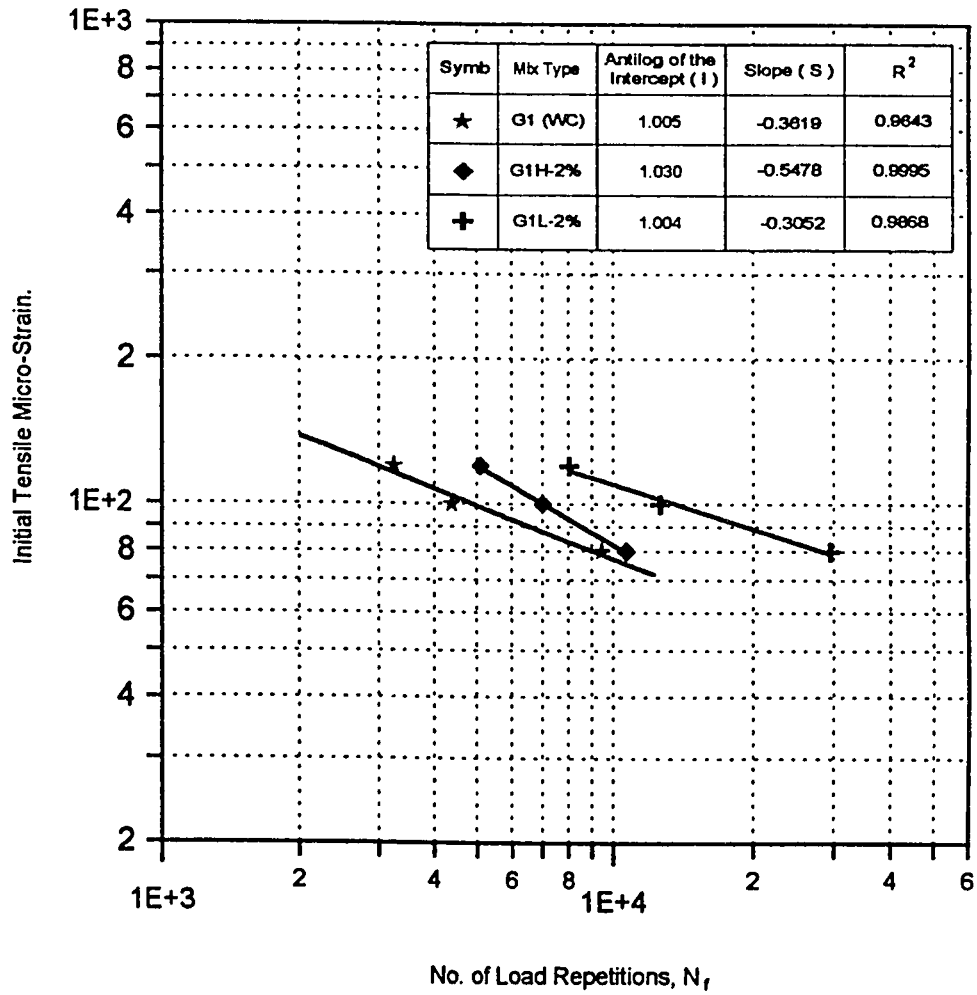


Fig. C6 : Fatigue Curves Comparision for 2-Percent Lime and Hedmanite Modified Wearing Coarse Mixes at 60°C.

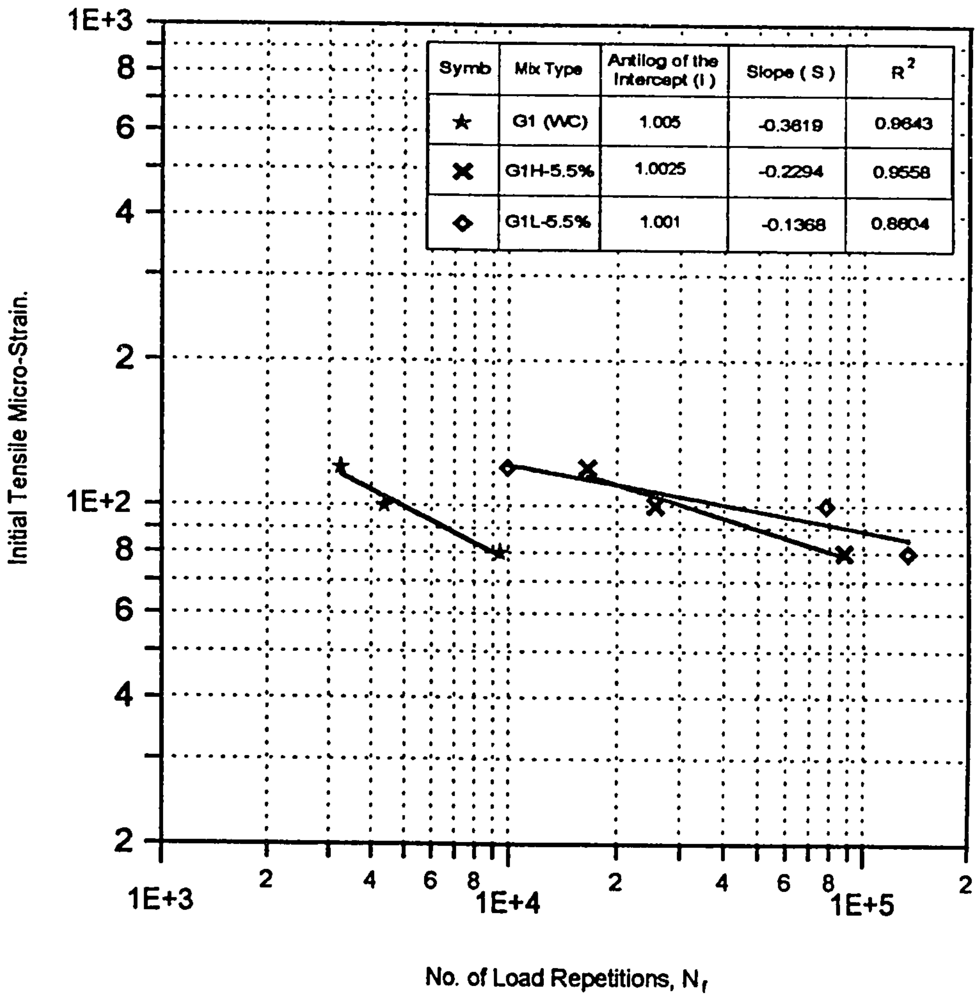


Fig. C7 : Fatigue Curves Comparision for 5.5-Percent Lime and Hedmanite Modified Wearing Coarse Mixes at 60° C.

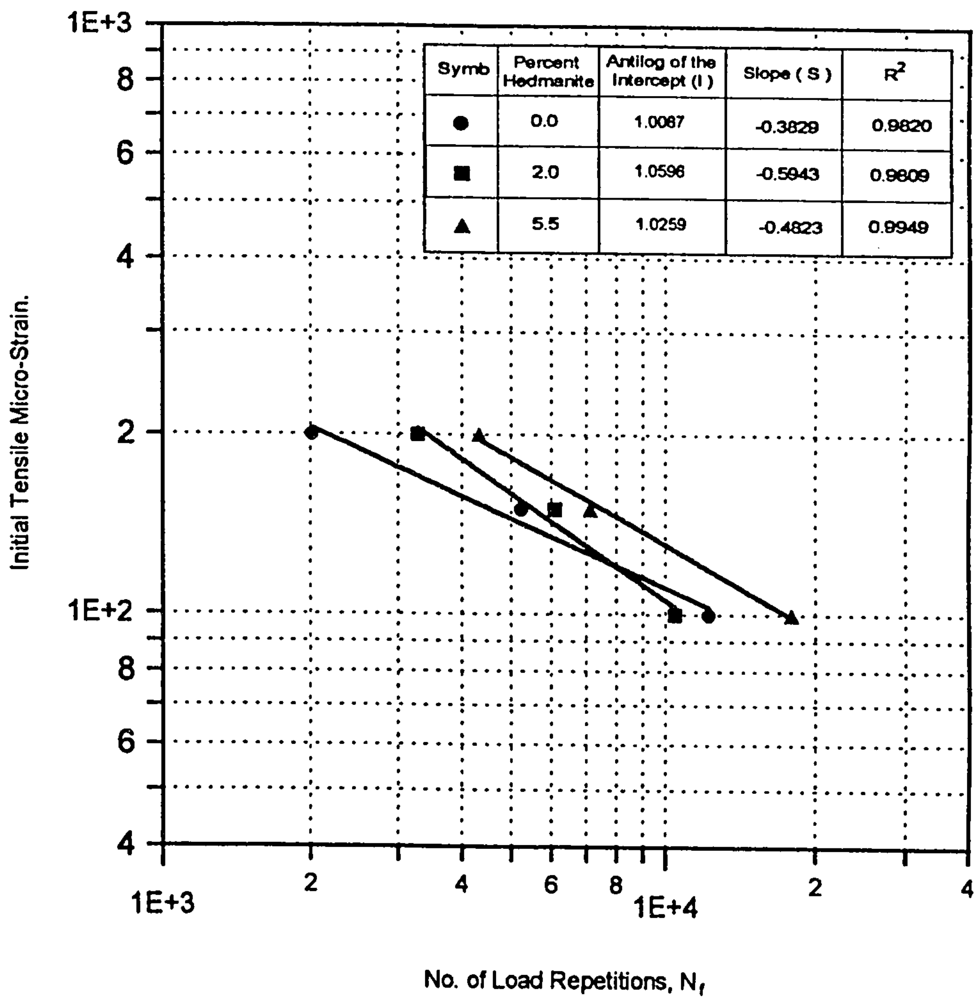
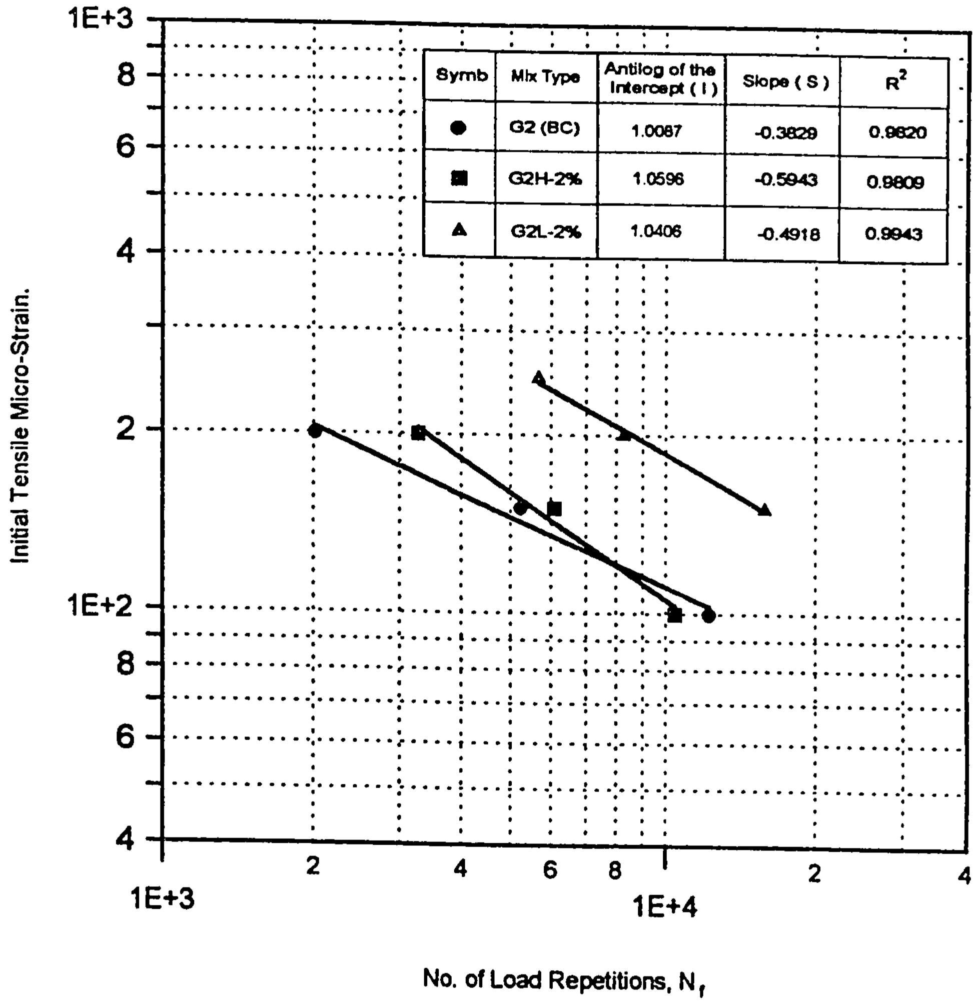
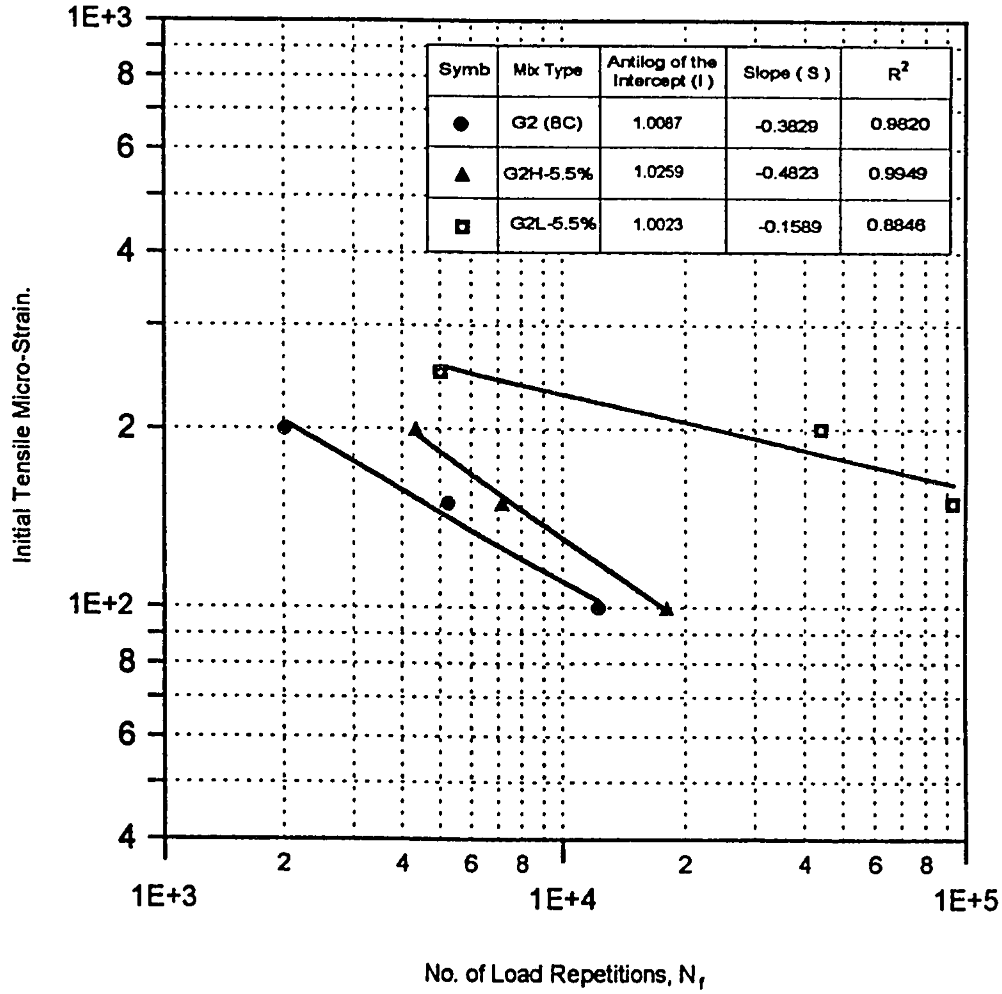


Fig. C8 : Fatigue Curve for Hedmanite Modified Base Coarse Mixes at 45 °C.



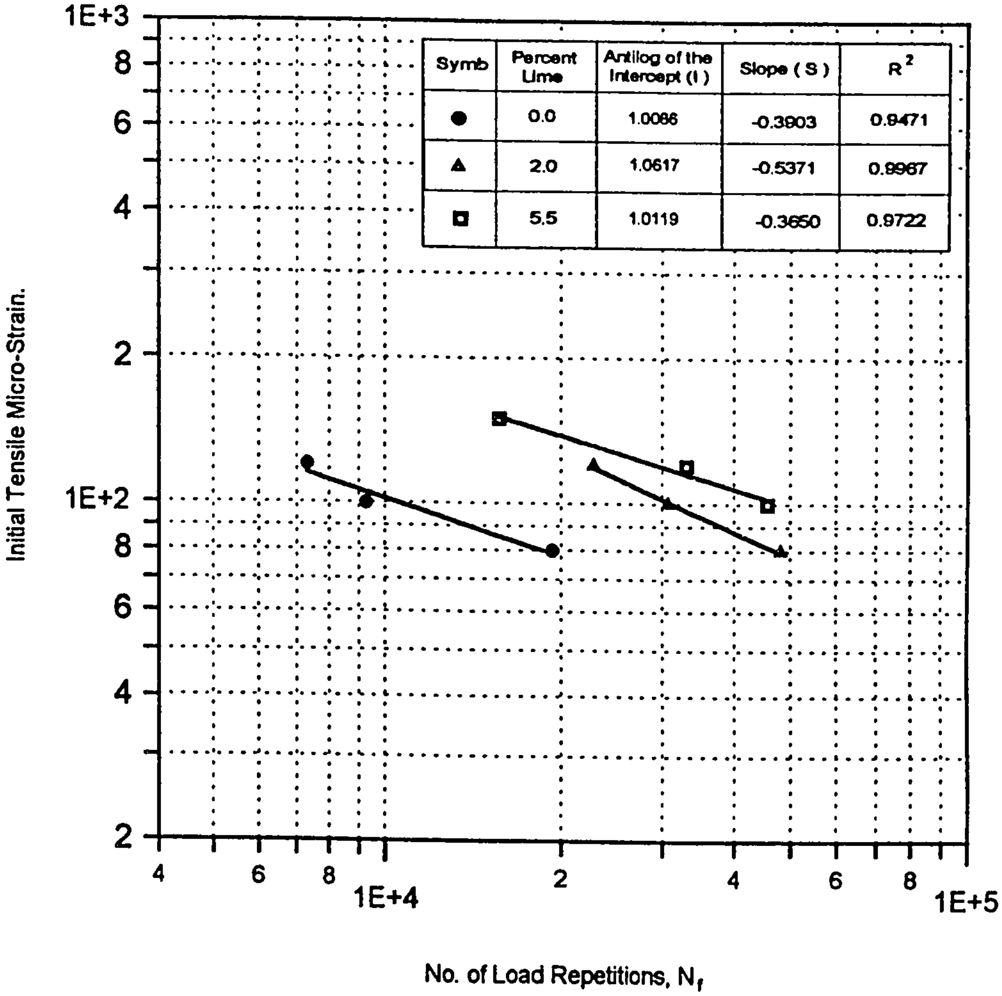
Symb	Mix Type	Antilog of the Intercept (I)	Slope (S)	R^2
●	G2 (BC)	1.0087	-0.3829	0.9820
■	G2H-2%	1.0596	-0.5943	0.9809
▲	G2L-2%	1.0406	-0.4918	0.9943

Fig. C9 : Fatigue Curves Comparision for 2--Percent Lime and Hedmanite Modified Base Coarse Mixes at 45°C.



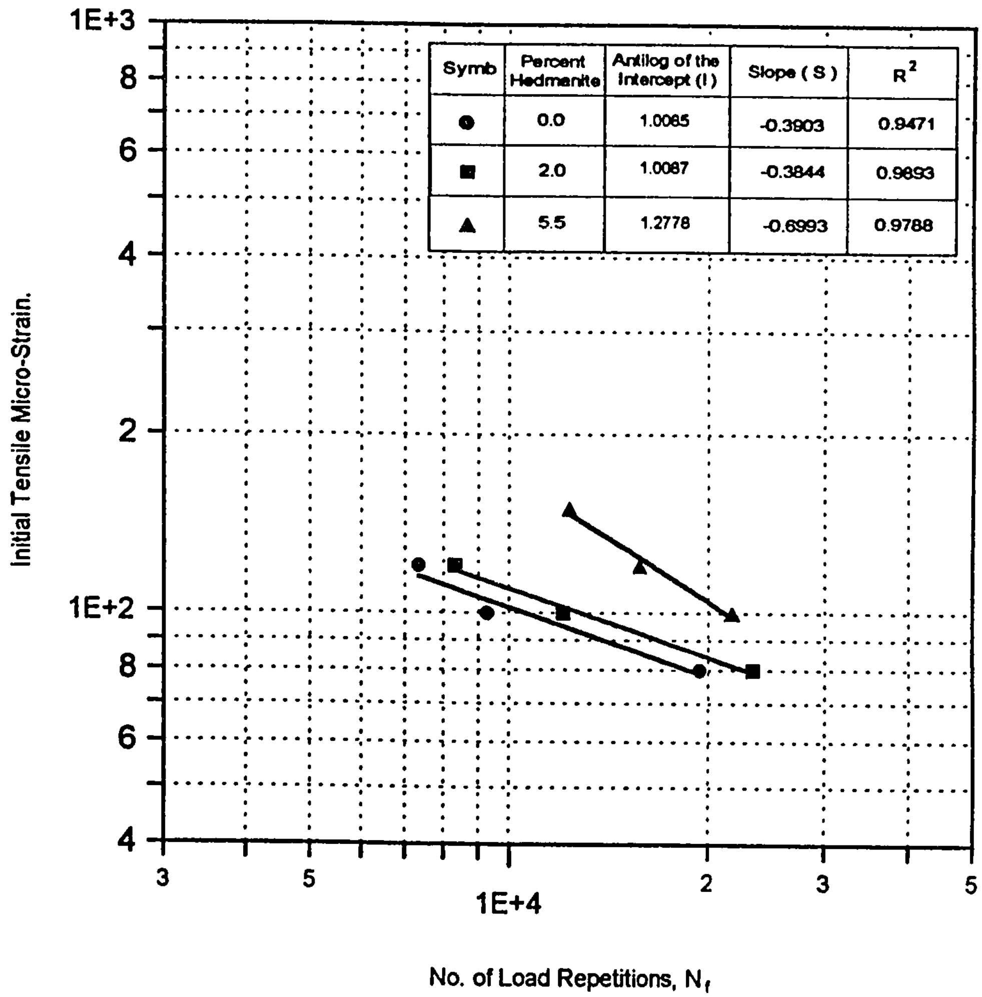
Symb	Mix Type	Antilog of the Intercept (I)	Slope (S)	R^2
●	G2 (BC)	1.0087	-0.3829	0.9820
▲	G2H-5.5%	1.0259	-0.4823	0.9949
◘	G2L-5.5%	1.0023	-0.1589	0.8846

Fig. C10 : Fatigue Curves Comparision between 5.5% Lime and Hedmanite Modified Base Coarse Mixes at 45 °C.



Symb	Percent Lime	Antilog of the Intercept (I)	Slope (S)	R^2
●	0.0	1.0086	-0.3903	0.9471
▲	2.0	1.0617	-0.5371	0.9967
◘	5.5	1.0119	-0.3650	0.9722

Fig. C11 : Fatigue Curve for Lime Modified Base Coarse Mixes at 60°C.



Symb	Percent Hedmanite	Antilog of the Intercept (I)	Slope (S)	R^2
●	0.0	1.0065	-0.3903	0.9471
■	2.0	1.0087	-0.3844	0.9693
▲	5.5	1.2778	-0.6993	0.9788

Fig. C12 : Fatigue Curve for Hedmanite Modified Base Coarse Mixes at 60° C.

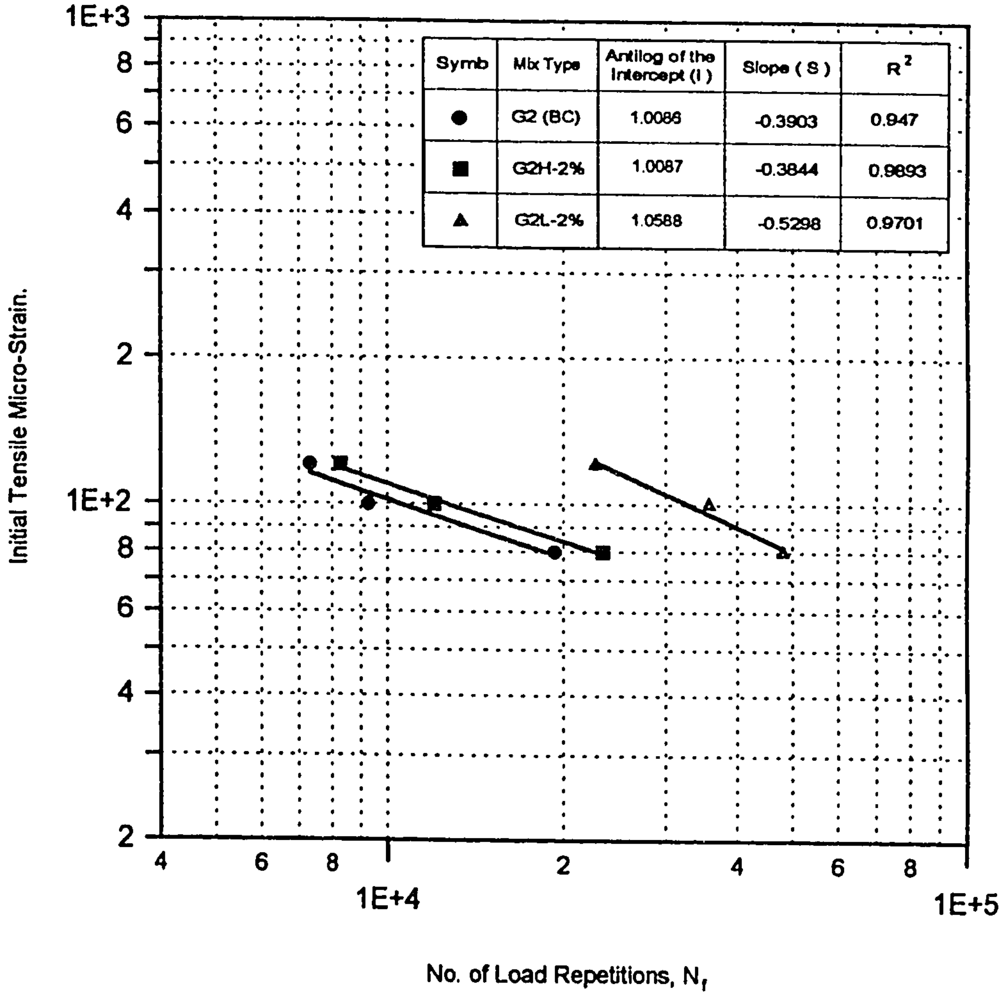
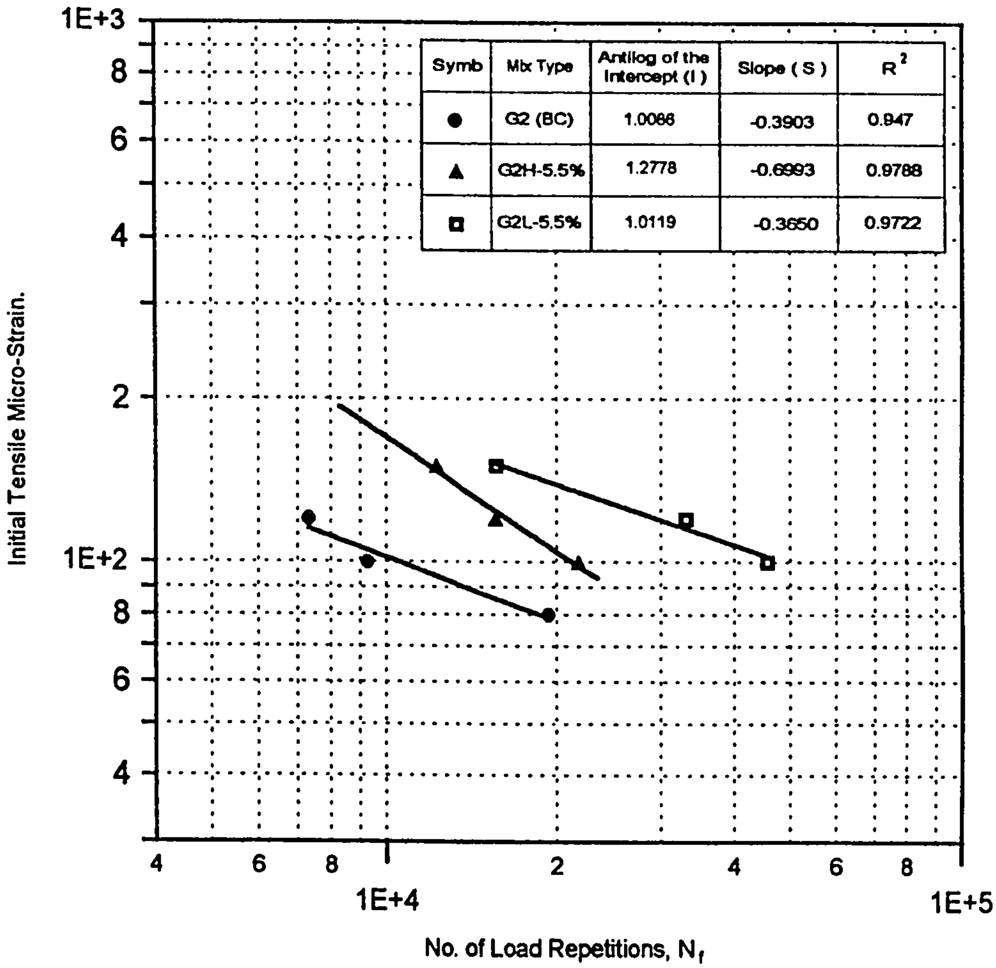


Fig. C13 : Fatigue Curves Comparision for 2-Percent Lime and Hedmanite Modified Base Coarse Mixes at 60° C.



Symb	Mix Type	Antilog of the Intercept (I)	Slope (S)	R^2
●	G2 (BC)	1.0086	-0.3903	0.947
▲	G2H-5.5%	1.2778	-0.6993	0.9788
◘	G2L-5.5%	1.0119	-0.3650	0.9722

Fig. C14 : Fatigue Curves Comparision for 5.5-Percent Lime and Hedmanite Modified Base Coarse Mixes at 60 °C.

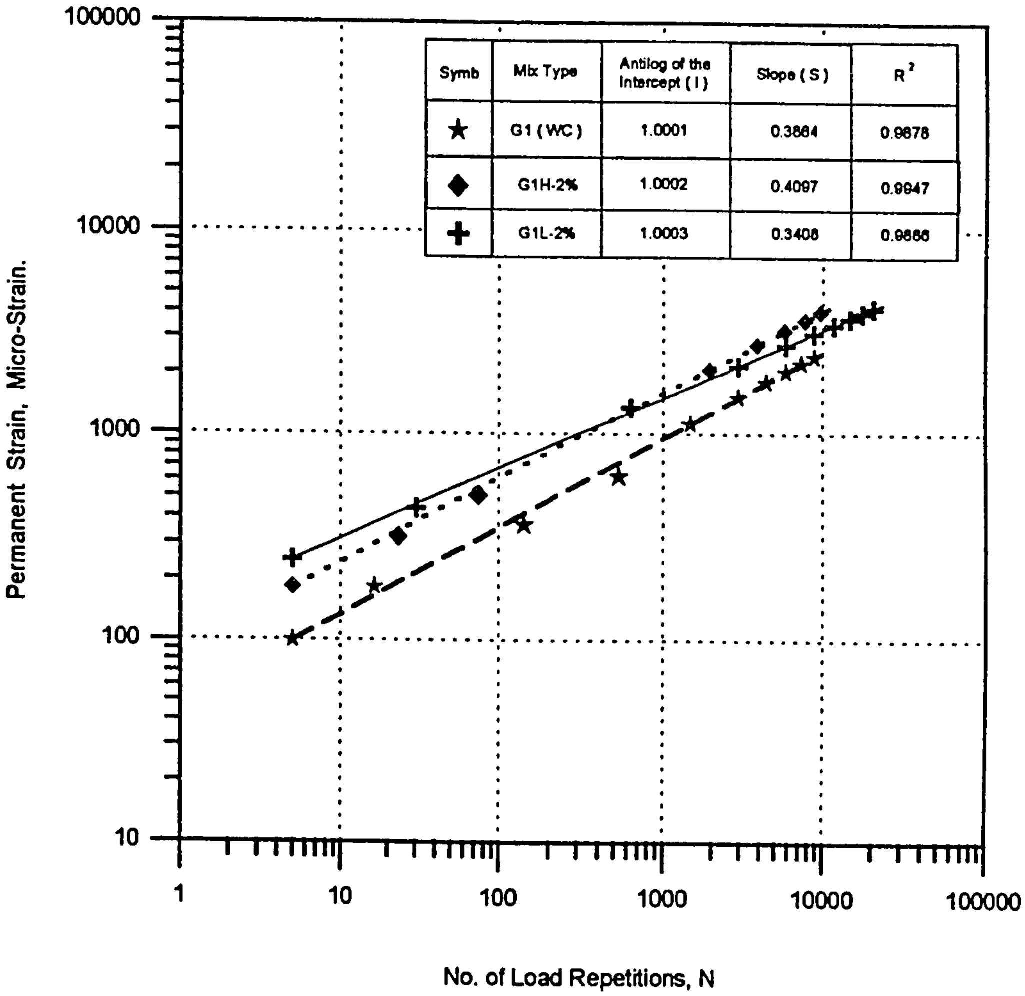


Fig. C15 : Rutting Curves for 2-Percent Lime & Hedmanite Modified Wearing Course mixes at 45°C.

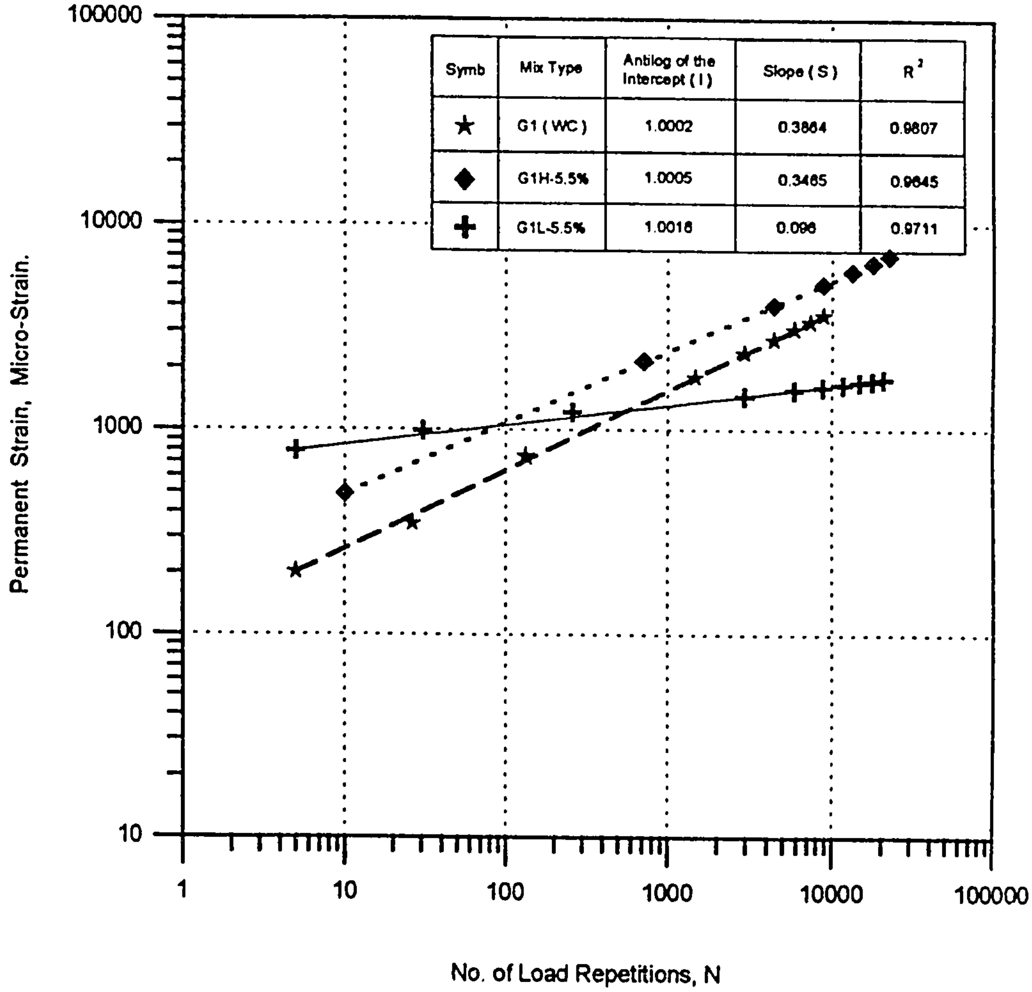


Fig. C16 : Rutting Curves for 5.5 Percent Lime & Hedmanite Modified Wearing Course mixes at 45°C.

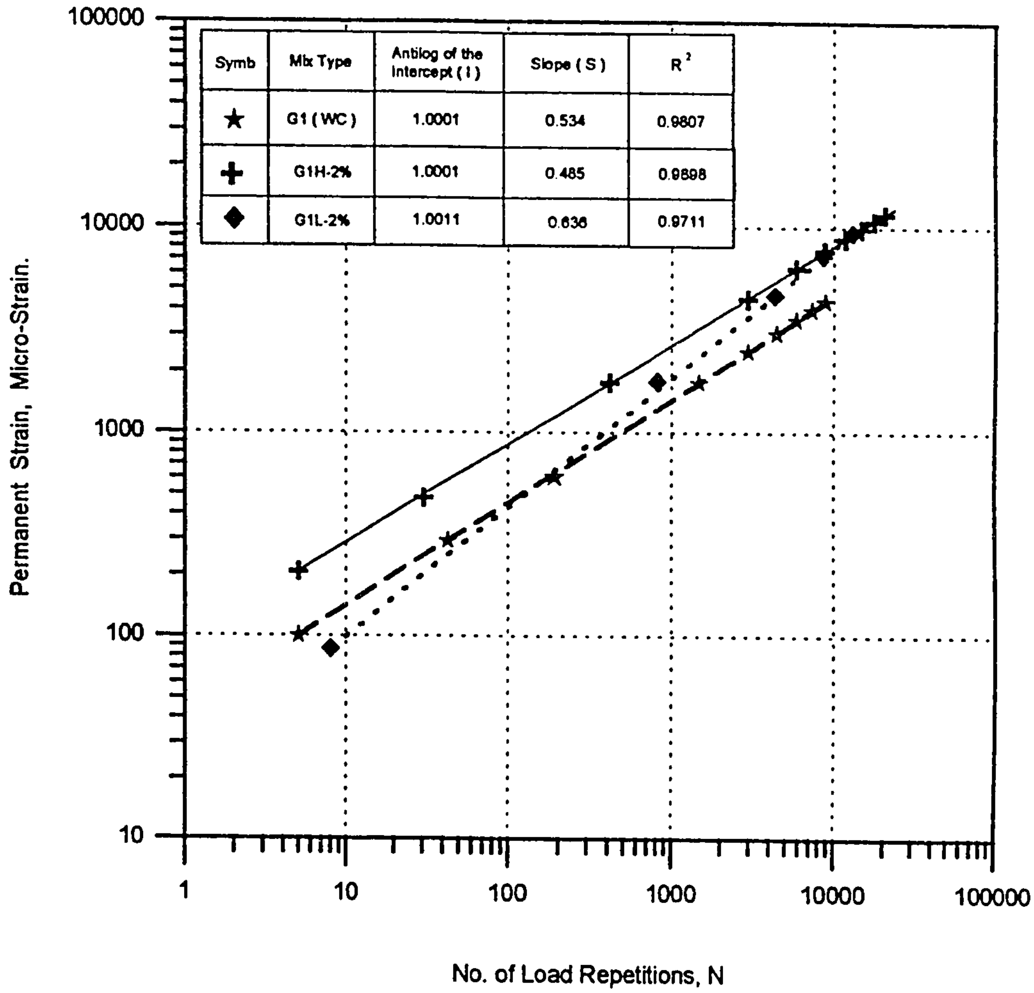
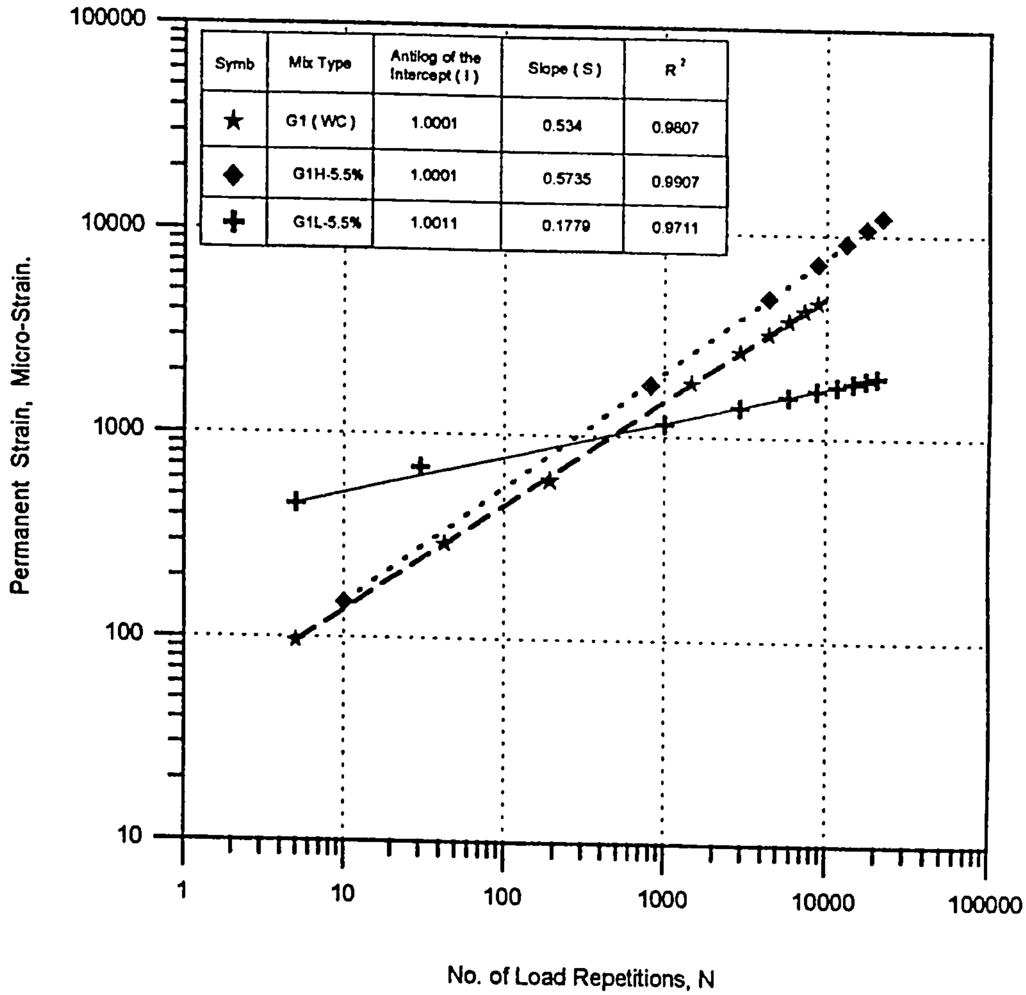


Fig. C17 : Rutting Curves for 2-Percent Lime & Hedmanite Modified Wearing Course mixes at 60°C.



Symb	Mix Type	Antilog of the Intercept (I)	Slope (S)	R^2
★	G1 (WC)	1.0001	0.534	0.9807
◆	G1H-5.5%	1.0001	0.5735	0.9907
✚	G1L-5.5%	1.0011	0.1779	0.9711

Fig. C18 : Rutting Curves for 5.5 Percent Lime & Hedmanite Modified Wearing Course mixes at 60°C.

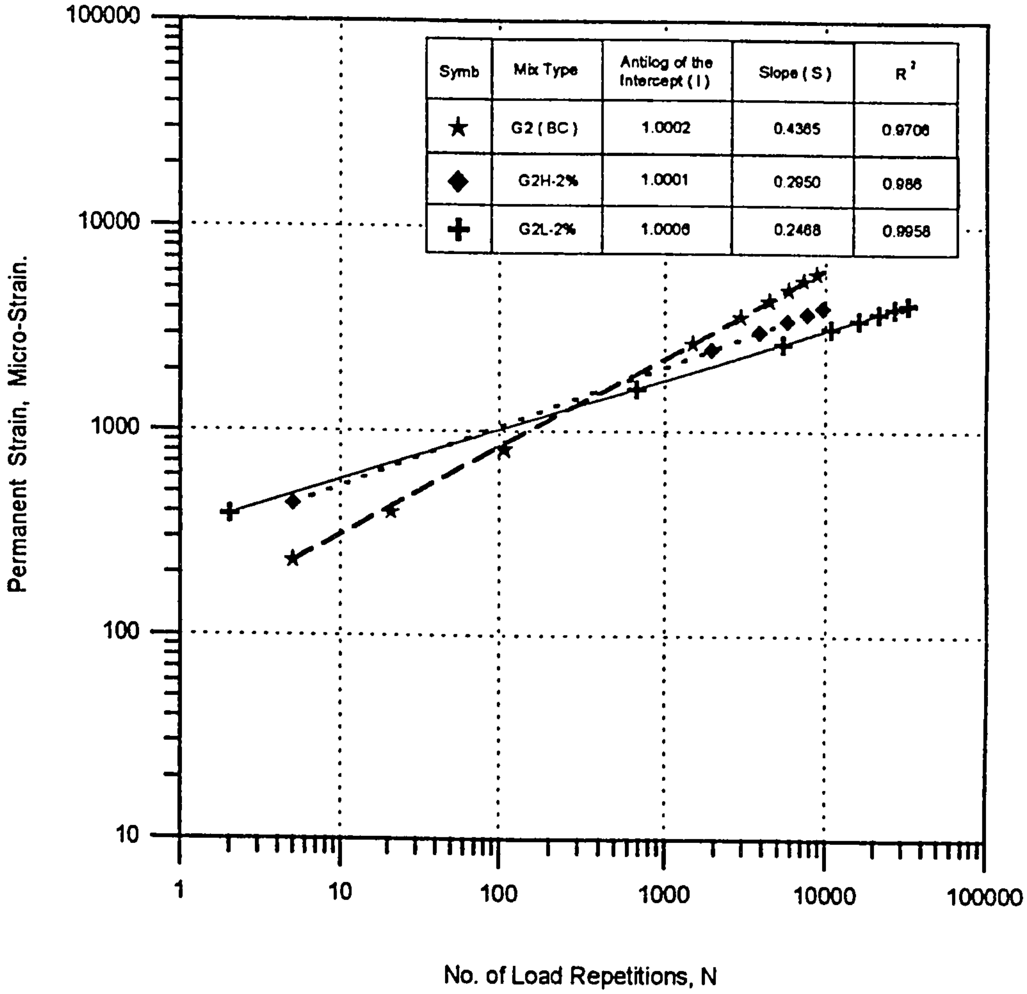


Fig. C19 : Rutting Curves for 2-Percent Lime & Hedmanite Modified Base Course mixes at 45°C.

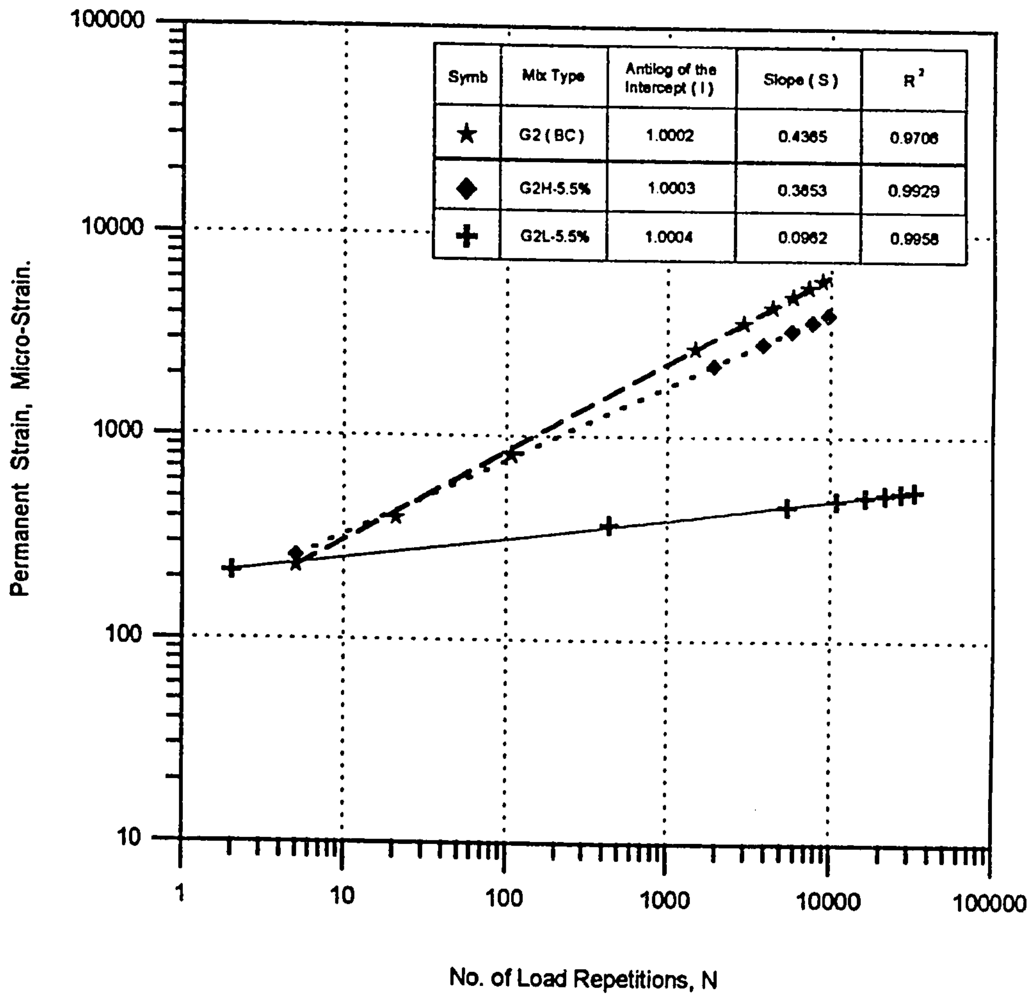


Fig. C20 : Rutting Curves for 5.5 Percent Lime & Hedmanite Modified Base Course mixes at 45°C.

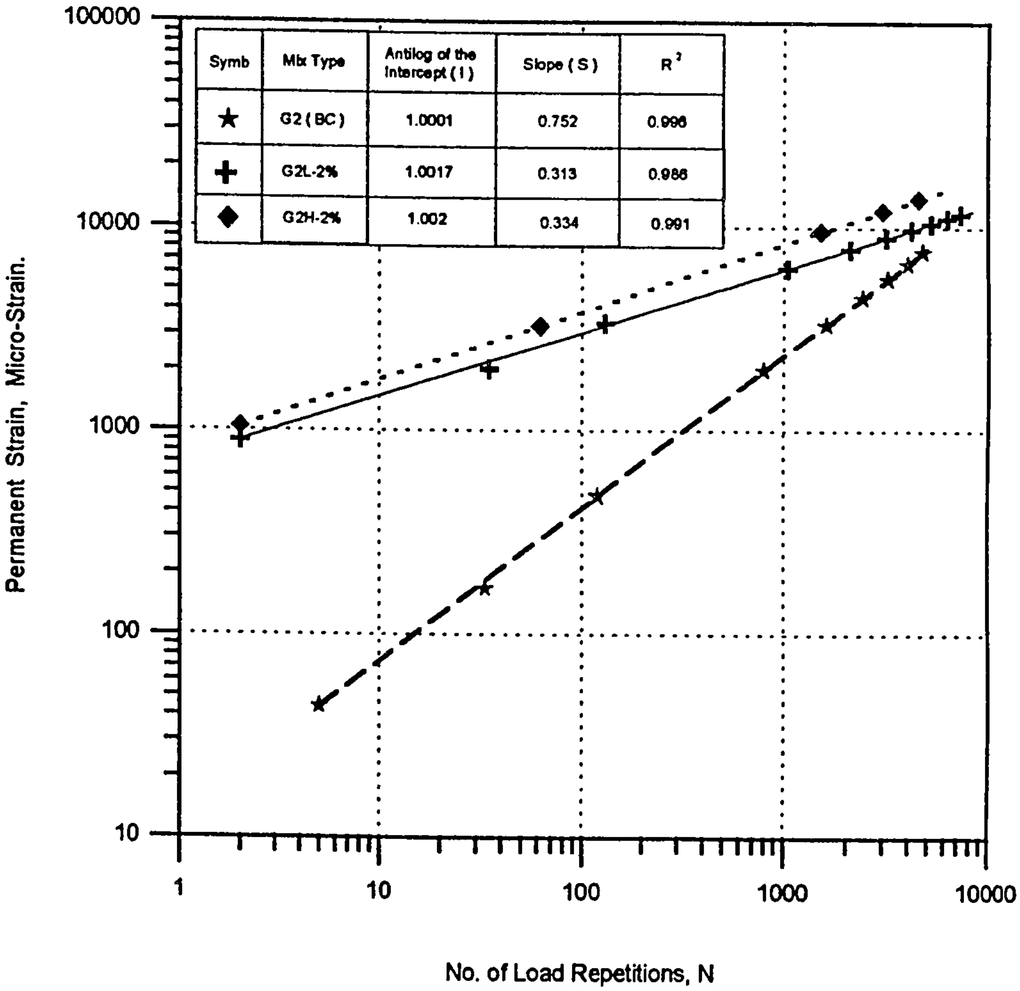


Fig. C21 : Rutting Curves for 2-Percent Lime & Hedmanite Modified Base Course mixes at 60°C.

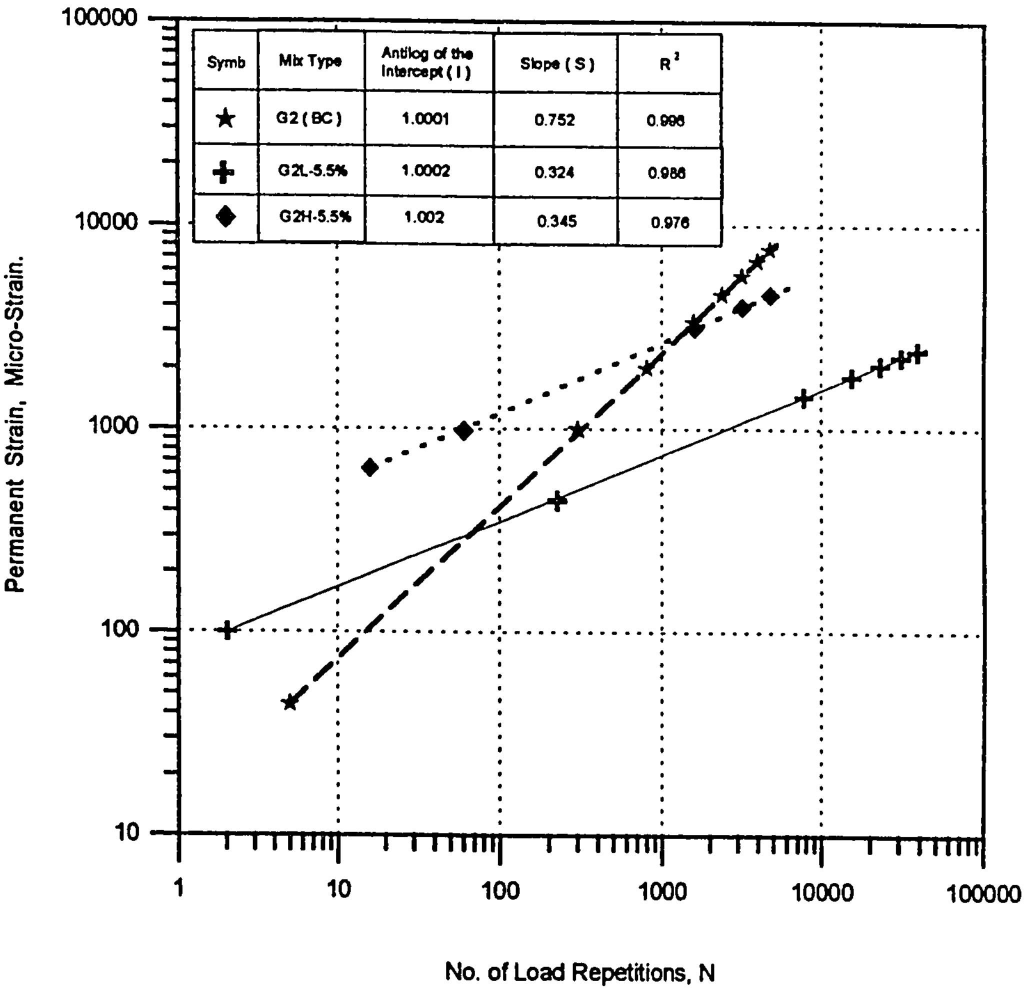


Fig. C22 : Rutting Curves for 5.5 Percent Lime & Hedmanite Modified Base Course mixes at 60°C.

APPENDIX - D

Statistical Analysis

Table D1 : MR data (in ksi) for Wearing Coarse mixes:

		Percent			
Material Type	0.0%	1.0%	2.0%	4.0%	5.5%
G1H	156.4	122	124.8	134.4	143.7
	143.6	140.3	139.6	144.8	152.4
	139.7	126.9	132.4	145.3	160.3
	125.1	116.6	152.8	137.4	156.9
	121.5	135.4	151.5	152.6	149.2
	128.2	151.4	149.7	167.4	166.3
G1L	156.4	128.7	165.7	166.4	137.3
	143.6	139.4	166.8	157.1	131.6
	139.7	154.5	156.5	173.3	126.7
	125.1	162.8	169.5	169.9	121.3
	121.5	140.1	150.3	167.2	132.5
	128.2	130.3	153	179.5	123.4

Anova: Two-Factor With Replication for MR (WC).

SUMMARY	0.0%	1.0%	2.0%	4.0%	5.5%	Total
G1H						
Count	6	6	6	6	6	30
Sum	814.5	792.6	850.8	881.9	928.8	4268.6
Average	135.75	132.1	141.8	146.9833	154.8	711.433333
Variance	175.427	163.984	131.94	141.2337	65.448	678.032667
G1L						
Count	6	6	6	6	6	30
Sum	814.5	855.84	961.8	1013.4	772.8	4418.34
Average	135.75	142.64	160.3	168.9	128.8	736.39
Variance	175.427	181.9296	64.756	56.22	36.72	515.0526
Total						
Count	12	12	12	12	12	
Sum	1629	1648.44	1812.6	1895.3	1701.6	
Average	271.5	274.74	302.1	315.8833	283.6	
Variance	350.854	345.9136	196.696	197.4537	102.168	

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Material type	373.7011	1	373.7011	3.132225	0.0828566	4.03431955
Percentages	4294.461	4	1073.615	8.998646	1.485E-05	2.55717936
Interaction	4455.345	4	1113.836	9.335763	1.026E-05	2.55717936
Within	5965.426	50	119.3085			
Total	15088.93	59				

Table D2 : MR data (in ksi) for Base Coarse mixes:						
		Percent				
Material Type	0.0%	1.0%	2.0%	4.0%	5.5%	
G2H	145.6	139.8	145.45	158	157.3	
	157.8	133.6	153	163.2	164.3	
	149.6	161.4	136.7	143.4	183.2	
	153.3	139.8	150.1	169.7	187.2	
	146.7	151.4	169.1	171.8	176.8	
	150.9	138.24	154.1	168.9	180	
G2L	145.6	151.8	162.3	146.3	133.9	
	157.8	180.3	163.8	160.7	152.6	
	149.6	148.7	180.2	169.3	149.1	
	153.3	152.7	172.3	165.6	162.3	
	146.7	159.2	177.2	150.6	137.1	
	150.9	172.1	165.16	156.22	150.6	
Anova: Two-Factor With Replication for (BC)						
SUMMARY	0.0%	1.0%	2.0%	4.0%	5.5%	Total
G2H						
Count	6	6	6	6	6	30
Sum	903.9	864.24	908.45	975	1048.8	4700.39
Average	150.65	144.04	151.4083	162.5	174.8	783.39833
Variance	20.083	106.8256	115.2644	112.968	134.372	489.51302
G2L						
Count	6	6	6	6	6	30
Sum	903.9	964.8	1020.96	948.72	885.6	4723.98
Average	150.65	160.8	170.16	158.12	147.6	787.33
Variance	20.083	160.704	56.4344	77.4944	110.056	424.7718
Total						
Count	12	12	12	12	12	
Sum	1807.8	1829.04	1929.41	1923.72	1934.4	
Average	301.3	304.84	321.5683	320.62	322.4	
Variance	40.166	267.5296	171.6988	190.4624	244.428	
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Material	9.274802	1	9.274802	0.101443	0.7514312	4.0343195
Percent	1250.261	4	312.5652	3.418685	0.0151057	2.5571794
Interaction	4165.366	4	1041.342	11.38968	1.199E-06	2.5571794
Within	4571.424	50	91.42848			
Total	9996.326	59				

Table D3 : Marshall Stability (kg) @ 35 minutes for Wearing Coarse mixes.						
		Percent				
Material Type	0.0%	1.0%	2.0%	4.0%	5.5%	
G1H	2166.3	1922.6	2245.5	2022.1	1793.3	
	2237.1	1896.5	2209.4	1984.7	1747.8	
	2142.54	1902.2	2242.9	1992	1782.4	
G1L	2166.3	1944.6	2125.2	2237.6	1986.9	
	2237.1	2038.3	2066.7	2291.4	1998.6	
	2142.54	1998.2	2109.9	2262.1	2004.6	
Anova: Two-Factor With Replication for Marshall Stability @ 35 min (WC).						
SUMMARY	0.0%	1.0%	2.0%	4.0%	5.5%	Total
G1H						
Count	3	3	3	3	3	15
Sum	6545.94	5721.3	6697.8	5998.8	5323.5	30287.34
Average	2181.98	1907.1	2232.6	1999.6	1774.5	10095.78
Variance	2419.7952	188.31	405.37	393.01	564.37	3970.855
G1L						
Count	3	3	3	3	3	15
Sum	6545.94	5981.1	6301.8	6791.1	5990.1	31610.04
Average	2181.98	1993.7	2100.6	2263.7	1996.7	10536.68
Variance	2419.7952	2210.11	920.43	725.53	81.03	6356.895
Total						
Count	6	6	6	6	6	
Sum	13091.88	11702.4	12999.6	12789.9	11313.6	
Average	4363.96	3900.8	4333.2	4263.3	3771.2	
Variance	4839.5904	2398.42	1325.8	1118.54	645.4	
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Material Tupe	58317.843	1	58317.84	56.46713	3.02E-07	4.35125
Percent	442507.44	4	110626.9	107.1161	3.28E-13	2.866081
Interaction	157749.97	4	39437.49	38.18595	4.26E-09	2.866081
Within	20655.501	20	1032.775			
Total	679230.76	29				

Table D4 : Marshall Stability (kg) @ 35 minutes for Base Coarse mixes.						
		Percent				
Material Type	0.0%	1.0%	2.0%	4.0%	5.5%	
G2H	1779.1	1885.6	2029.8	1937.2	1749.1	
	1857.3	1842.7	2071.4	1968.8	1717.8	
	1772.3	1850.5	2069.2	2013.3	1738.55	
G2L	1779.1	1707.2	1942.1	2082.3	1864.8	
	1857.3	1638.9	1981.4	2055	1848.4	
	1772.3	1613.05	2002.1	2074.2	1872.2	
Anova: Two-Factor With Replication for Marshall Stability @ 35 min (BC)						
SUMMARY	0.0%	1.0%	2.0%	4.0%	5.5%	Total
G2H						
Count	3	3	3	3	3	15
Sum	5408.7	5578.8	6170.4	5919.3	5205.45	28282.65
Average	1802.9	1859.6	2056.8	1973.1	1735.15	9427.55
Variance	2231.08	522.21	547.96	1461.67	253.5925	5016.513
G2L						
Count	3	3	3	3	3	15
Sum	5408.7	4959.15	5925.6	6211.5	5585.4	28090.35
Average	1802.9	1653.05	1975.2	2070.5	1861.8	9363.45
Variance	2231.08	2366.222	928.83	196.59	148.36	5871.083
Total						
Count	6	6	6	6	6	
Sum	10817.4	10537.95	12096	12130.8	10790.85	
Average	3605.8	3512.65	4032	4043.6	3596.95	
Variance	4462.16	2888.432	1476.79	1658.26	401.9525	
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Material	1232.643	1	1232.643	1.132154	0.299995	4.35125
Percent	398912.6	4	99728.15	91.59796	1.45E-12	2.866081
Interaction	111040	4	27760.01	25.49691	1.31E-07	2.866081
Within	21775.19	20	1088.76			
Total	532960.4	29				

Table D5: Marshall Stability (kg) @ 24 hrs for Wearing Coarse mixes						
		Percent				
Material Type	0.0%	1.0%	2.0%	4.0%	5.5%	
G1H	1577.3	1623.4	1804.8	1459.8	1214.4	
	1548.2	1598.2	1789.3	1480.2	1191	
	1600.7	1607.2	1784.3	1476.9	1195.8	
G1L	1577.3	1780.3	1886	2296.5	1370.1	
	1548.2	1770.1	1913.2	2300.5	1358.3	
	1600.7	1695.01	1887.6	2316.5	1368.7	
Anova: Two-Factor With Replication for Marshall Stability @ 24 hrs (WC)						
SUMMARY	0.0%	1.0%	2.0%	4.0%	5.5%	Total
G1H						
Count	3	3	3	3	3	15
Sum	4726.2	4828.8	5378.4	4416.9	3601.2	22951.5
Average	1575.4	1609.6	1792.8	1472.3	1200.4	7650.5
Variance	691.77	163.08	114.25	119.91	152.76	1241.77
G1L						
Count	3	3	3	3	3	15
Sum	4726.2	5245.41	5686.8	6913.5	4097.1	26669.01
Average	1575.4	1748.47	1895.6	2304.5	1365.7	8889.67
Variance	691.77	2169.489	232.96	112	41.56	3247.779
Total						
Count	6	6	6	6	6	
Sum	9452.4	10074.21	11065.2	11330.4	7698.3	
Average	3150.8	3358.07	3688.4	3776.8	2566.1	
Variance	1383.54	2332.569	347.21	231.91	194.32	
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Material	460662.7	1	460662.7	1026.078	1.16E-18	4.35125
Percent	1413168	4	353292	786.921	1.1E-21	2.866081
Interaction	663937.8	4	165984.4	369.713	1.94E-18	2.866081
Within	8979.097	20	448.9549			
Total	2546748	29				

Table D6: Marshall Stability (kg) @ 24 hrs for Base Coarse mixes						
Material Type		Percent				
	0.0%	1.0%	2.0%	4.0%	5.5%	
G2H	1575.4	1609.6	1792.8	1472.3	1200.4	
	1433.6	1588.45	1712.2	1447	1223.3	
	1404.5	1570.1	1687	1465.4	1201.7	
G2L	1575.4	1748.47	1895.6	2304.5	1365.7	
	1433.6	1549.8	2029.7	1764.3	1384.2	
	1404.5	1524.5	1995.6	1793.7	1389.5	

Anova: Two-Factor With Replication for Marshall Stability @ 24 hrs (BC)

SUMMARY	0	0.01	0.02	0.04	0.055	Total
G2H						
Count	3	3	3	3	3	15
Sum	4413.5	4768.15	5192	4384.7	3625.4	22383.75
Average	1471.167	1589.383	1730.667	1461.567	1208.467	7461.25
Variance	8360.143	390.7158	3054.173	171.0433	165.4433	12141.52
G2L						
Count	3	3	3	3	3	15
Sum	4413.5	4822.77	5920.9	5862.5	4139.4	25159.07
Average	1471.167	1607.59	1973.633	1954.167	1379.8	8386.357
Variance	8360.143	15045.4	4857.603	92266.17	156.13	120685.5
Total						
Count	6	6	6	6	6	
Sum	8827	9590.92	11112.9	10247.2	7764.8	
Average	2942.333	3196.973	3704.3	3415.733	2588.267	
Variance	16720.29	15436.12	7911.777	92437.22	321.5733	

ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Material	256746.7	1	256746.7	19.32941	0.000278	4.35125
Percent	1105250	4	276312.6	20.80245	6.77E-07	2.866081
Interaction	240314.5	4	60078.63	4.523075	0.009153	2.866081
Within	265653.9	20	13282.7			
Total	1867966	29				

Table D7: Split Tensile Strength (ksi) @ 2 hour for Wearing Coarse mixes						
			Percent			
Material Type	0%	1%	2%	4%	5.50%	
G1H	140.3	123.9	155.3	160	147.8	
	155.2	146.4	173.2	133.4	129.7	
	149.04	144	159.3	153	152.4	
G1L	140.3	133.8	171.2	139.5	134.9	
	155.2	146.5	153.4	160.1	114.6	
	149.04	143.9	176.7	146.14	112.6	

Anova: Two-Factor With Replication for Split Tensile Strength @ 2hr (WC)

SUMMARY	0%	1%	2%	4%	5.50%	Total
G1H						
Count	3	3	3	3	3	15
Sum	444.54	414.3	487.8	446.4	429.9	2222.94
Average	148.18	138.1	162.6	148.8	143.3	740.98
Variance	56.0572	152.67	88.27	190.12	144.01	631.1272
G1L						
Count	3	3	3	3	3	15
Sum	444.54	424.2	501.3	445.74	362.1	2177.88
Average	148.18	141.4	167.1	148.58	120.7	725.96
Variance	56.0572	45.01	148.33	110.5552	152.23	512.1824
Total						
Count	6	6	6	6	6	
Sum	889.08	838.5	989.1	892.14	792	
Average	296.36	279.5	329.7	297.38	264	
Variance	112.1144	197.68	236.6	300.6752	296.24	

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Material	67.68012	1	67.68012	0.591967	0.450651	4.35125
Percent	3599.79192	4	899.948	7.871429	0.000557	2.866081
Interaction	745.24248	4	186.3106	1.629573	0.205908	2.866081
Within	2286.6192	20	114.331			
Total	6699.33372	29				

Table D8: Split Tensile Strength (ksi) @ 2 hour for Base Coarse mixes						
			Percent			
Material Type	0%	1%	2%	4%	5.50%	
G2H	103.8	132.2	128.2	125.5	131.2	
	117.4	116.1	150	141.9	120.1	
	114.4	120.4	137.6	124.1	122.8	
G2L	103.8	124	137.8	125.4	87.3	
	117.4	141.1	122.7	109.8	101.5	
	117.4	133.57	131.9	120.6	92.9	

Anova: Two-Factor With Replication for Split Tensile Strength @ 2hr (BC)

SUMMARY	0%	1%	2%	4%	5.50%	Total
G2H						
Count	3	3	3	3	3	15
Sum	335.6	368.7	415.8	391.5	374.1	1885.7
Average	111.8667	122.9	138.6	130.5	124.7	628.5667
Variance	51.05333	69.49	119.56	97.96	33.51	371.5733
G2L						
Count	3	3	3	3	3	15
Sum	338.6	398.67	392.4	355.8	281.7	1767.17
Average	112.8667	132.89	130.8	118.6	93.9	589.0567
Variance	61.65333	73.4493	57.91	63.84	51.16	308.0126
Total						
Count	6	6	6	6	6	
Sum	674.2	767.37	808.2	747.3	655.8	
Average	224.7333	255.79	269.4	249.1	218.6	
Variance	112.7067	142.9393	177.47	161.8	84.67	

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Material	468.312	1	468.312	6.891137	0.016221	4.35125
Percent	2738.113	4	684.5283	10.07273	0.000124	2.866081
Interaction	1409.523	4	352.3808	5.185227	0.004951	2.866081
Within	1359.172	20	67.9586			
Total	5975.12	29				

Table D9: Split Tensile Strength (ksi) @ 24 hour for Wearing Coarse mixes						
		Percent				
Material Type	0%	1%	2%	4%	5.50%	
G1H	121.3	108.2	93.3	71.1	75.6	
	107.4	109.7	94.8	91.4	70.1	
	117	77.6	118.8	67.9	60.4	
G1L	121.3	127.8	137.8	123.4	95.7	
	107.4	110	140.5	143.6	77.9	
	117	104.8	157	137.7	85.6	

Anova: Two-Factor With Replication for Split Tensile Strength @ 24 hrs (WC)

SUMMARY	0%	1%	2%	4%	5.50%	Total
G1H						
Count	3	3	3	3	3	15
Sum	345.7	295.5	306.9	230.4	206.1	1384.6
Average	115.233333	98.5	102.3	76.8	68.7	461.53333
Variance	50.6433333	328.17	204.75	162.43	59.23	805.22333
G1L						
Count	3	3	3	3	3	15
Sum	345.7	342.6	435.3	404.7	259.2	1787.5
Average	115.233333	114.2	145.1	134.9	86.4	595.83333
Variance	50.6433333	145.48	108.03	107.89	79.69	491.73333
Total						
Count	6	6	6	6	6	
Sum	691.4	638.1	742.2	635.1	465.3	
Average	230.466667	212.7	247.4	211.7	155.1	
Variance	101.286667	473.65	312.78	270.32	138.92	

ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Material Type	5410.947	1	5410.947	41.72034	2.68E-06	4.35125
Percent	7246.47133	4	1811.618	13.96822	1.35E-05	2.8660807
Interaction	3239.898	4	809.9745	6.245193	0.001979	2.8660807
Within	2593.91333	20	129.6957			
Total	18491.2297	29				

Table D10: Split Tensile Strength (ksi) @ 24 hour for Base Coarse mixes						
		Percent				
Material Type	0%	1%	2%	4%	5.50%	
G2H	96.3	86.5	104.7	60.6	72.5	
	92.5	103.4	87.7	81.8	58.4	
	71.9	87.3	92.9	78.7	69.8	
G2L	96.3	113.6	111.5	108.1	61.3	
	92.5	102.7	126.1	90.5	74.2	
	71.9	100.2	115.2	97.2	72.1	

Anova: Two-Factor With Replication for Split Tensile Strength @ 24 hrs (BC)

SUMMARY	0%	1%	2%	4%	5.50%	Total
G2H						
Count	3	3	3	3	3	15
Sum	260.7	277.2	285.3	221.1	200.7	1245
Average	86.9	92.4	95.1	73.7	66.9	415
Variance	172.36	90.91	75.88	131.11	56.01	526.27
G2L						
Count	3	3	3	3	3	15
Sum	260.7	316.5	352.8	295.8	207.6	1433.4
Average	86.9	105.5	117.6	98.6	69.2	477.8
Variance	172.36	50.77	57.61	78.91	47.91	407.56
Total						
Count	6	6	6	6	6	
Sum	521.4	593.7	638.1	516.9	408.3	
Average	173.8	197.9	212.7	172.3	136.1	
Variance	344.72	141.68	133.49	210.02	103.92	

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Material Type	1183.152	1	1183.152	12.66989	0.001965	4.35125
Percent	5106.408	4	1276.602	13.6706	1.58E-05	2.8660807
Interaction	771.588	4	192.897	2.065654	0.123552	2.8660807
Within	1867.66	20	93.383			
Total	8928.808	29				

Table D11: Static Creep (mm) Data Analysis for Wearing Coarse & Base Coarse						
		Percent				
Material Type	0%	1%	2%	4%	5.50%	
G1H	0.41	0.54	0.6	0.51	0.4	
	0.29	0.34	0.4	0.35	0.48	
G1L	0.41	0.51	0.08	0.4	0.2	
	0.29	0.25	0.13	0.36	0.16	
G2H	0.82	0.88	0.6	0.42	0.42	
	0.66	0.72	0.56	0.32	0.34	
G2L	0.82	0.14	0.37	0.43	0.35	
	0.66	0.12	0.27	0.33	0.27	

Anova: Two-Factor With Replication

ANOVA FOR WEARING COARSE MIX (G1)

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Material	0.117045	1	0.117045	10.93368	0.007925	4.964591
Percent	0.04132	4	0.01033	0.96497	0.467787	3.47805
Interaction	0.11268	4	0.02817	2.631481	0.097821	3.47805
Within	0.10705	10	0.010705			
Total	0.378095	19				

ANOVA FOR BASE COARSE MIX (G2)

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Material	0.19602	1	0.19602	32.24013	0.000204	4.964591
Percent	0.3914	4	0.09785	16.09375	0.000234	3.47805
Interaction	0.32548	4	0.08137	13.38322	0.000504	3.47805
Within	0.0608	10	0.00608			
Total	0.9737	19				

Table D12: Statistical Analysis of Fatigue Data for Wearing Coarse

Material	Percent	Initial Strain @ 45C	Repetitions to Failure @ 45C	Initial Strain @ 60C	Repetitions to Failure @ 45C
G1H	0	100	16800	80	9422
G1H	0	150	8074	100	4369
G1H	0	200	4859	120	3238
G1H	2	100	15435	80	10654
G1H	2	150	10008	100	6996
G1H	2	200	6710	120	5089
G1H	5.5	100	30463	80	87702
G1H	5.5	150	17402	100	25544
G1H	5.5	200	9250	120	16544
G1L	0	100	16800	80	9422
G1L	0	150	8074	100	4369
G1L	0	200	4859	120	3238
G1L	2	100	56895	80	29442
G1L	2	150	16066	100	12607
G1L	2	200	10749	120	8007
G1L	5.5	100	98621	80	134374
G1L	5.5	150	13749	100	78130
G1L	5.5	200	8072	120	9892

Analysis of Variance for Fatigue @ 45C - Type III Sums of Squares

Source of variation	Sum of Squares	d.f.	Mean square	F	Sig. level	F-critical
MAIN EFFECTS						
A:Material	7.3324E08	1	7.3324E08	2.375	.149	4.75
B:Percent	1.1629E09	2	5.8145E08	1.884	.194	3.89
C:Repetitions	3.5143E09	2	1.7572E09	5.692	.0183	3.89
RESIDUAL	3.7042E09	12	3.0868E08			
TOTAL (CORRECTED)	9.1146E09	17				

Analysis of Variance for Fatigue @ 60C - Type III Sums of Squares

Source of variation	Sum of Squares	d.f.	Mean square	F	Sig. level	F-critical
MAIN EFFECTS						
A:Material	7.8987E08	1	7.8987E09	1.345	.2688	4.75
B:Percent	1.0043E09	2	5.0213E09	8.450	.0051	3.89
C:Repetitions	4.7126E09	2	2.3563E09	3.965	.0476	3.89
RESIDUAL	7.1307E09	12	5.9422E08			
TOTAL (CORRECTED)	2.2685E010	17				

All F-ratios are based on the residual mean square error.

Table D13 : Statistical Analysis of Fatigue Data for Base Coarse

Material	Percent	Initial Strain @ 45C	Repetitions to Failure @ 45C	Initial Strain @ 60C	Repetitions to Failure @ 45C
G2H	0	100	12167	80	19325
G2H	0	150	5236	100	9256
G2H	0	200	2009	120	7322
G2H	2	100	10415	80	23391
G2H	2	150	6076	100	12028
G2H	2	200	3266	120	8290
G2H	5.5	100	17882	80	21766
G2H	5.5	150	7130	100	15649
G2H	5.5	200	4322	120	12280
G2L	0	100	12167	80	19325
G2L	0	150	5236	100	9256
G2L	0	200	2009	120	7322
G2L	2	100	35641	80	38080
G2L	2	150	15788	100	25641
G2L	2	200	8274	120	12714
G2L	5.5	100	92826	80	45681
G2L	5.5	150	43867	100	33099
G2L	5.5	200	5011	120	15680

Analysis of Variance for Fatigue @ 45C (BC) - Type III Sums of Squares

Source of variation	Sum of Squares	d.f.	Mean square	F	Sig. level	F-critical
MAIN EFFECTS						
A:Material	1.2889E09	1	1.2889E09	4.402	.0578	4.75
B:Percent	1.5288E09	2	7.6440E08	2.610	.1145	3.89
C:Repetitions	2.0763E09	2	2.9283E08	3.545	.0617	3.89
RESIDUAL	3.5140E09	12	2.9283E08			
TOTAL (CORRECTED)	8.4080E09	17				

Analysis of Variance for Fatigue @ 60C (BC) - Type III Sums of Squares

Source of variation	Sum of Squares	d.f.	Mean square	F	Sig. level	F-critical
MAIN EFFECTS						
A:Material	3.3360E08	1	3.3360E08	10.608	.0069	4.75
B:Percent	4.5264E08	2	2.2632E08	7.196	.0088	3.89
C:Repetitions	9.1326E08	2	4.5663E08	14.52	.0006	3.89
RESIDUAL	3.7739E08	12	31449510			
TOTAL (CORRECTED)	2.0769E09	17				

All F-ratios are based on the residual mean square error.

Table D14: Statistical Analysis of Rutting Data for Wearing Coarse

Material	Percent	Repetitions	P-Defr @ 45C	P-Defr @ 60C
G1H	0	500	673	1015
G1H	0	1000	844	1417
G1H	0	2000	1124	1748
G1H	2	500	1169	1901
G1H	2	1000	1488	2622
G1H	2	2000	1941	3271
G1H	5.5	500	1842	1452
G1H	5.5	1000	2185	1984
G1H	5.5	2000	2700	2787
G1L	0	500	673	1015
G1L	0	1000	844	1417
G1L	0	2000	1124	1748
G1L	2	500	1145	1062
G1L	2	1000	1382	1405
G1L	2	2000	1815	3011
G1L	5.5	500	1275	1015
G1L	5.5	1000	1370	1122
G1L	5.5	2000	1480	1251

Analysis of Variance for Rutting @ 45C - Type III Sums of Squares

Source of variation	Sum of Squares	d.f.	Mean square	F	Sig. level	F-critical
MAIN EFFECTS						
A:Material	453786.9	1	453786.9	6.510	.0254	4.75
B:Percent	2670089.3	2	1335044.7	19.152	.0002	3.89
C:Repetitions	982310.3	2	491155.2	7.046	.0095	3.89
RESIDUAL	836487.44	12	69707.287			
TOTAL (CORRECTED)	4942674.0	17				

Analysis of Variance for Rutting @ 60C - Type III Sums of Squares

Source of variation	Sum of Squares	d.f.	Mean square	F	Sig. level	F-critical
MAIN EFFECTS						
A:Material	1474044.5	1	1474044.5	9.224	.0103	4.75
B:Percent	2171981.4	2	1085990.7	6.796	.0106	3.89
C:Repetitions	3416588.1	2	1708294.1	10.690	.0022	3.89
RESIDUAL	1917607.6	12	159800.63			
TOTAL (CORRECTED)	8980221.6	17				

Table D15: Statistical Analysis of Rutting Data for Base Coarse

Material	Percent	Repetitions	P-Defr @ 45C	P-Defr @ 60C
G2H	0	500	1618	1441
G2H	0	1000	2090	2315
G2H	0	2000	2843	3673
G2H	2	500	1700	6449
G2H	2	1000	2090	8480
G2H	2	2000	2800	10311
G2H	5.5	500	1370	1949
G2H	5.5	1000	1783	2575
G2H	5.5	2000	2185	3708
G2L	0	500	1618	1441
G2L	0	1000	2090	2315
G2L	0	2000	2843	3673
G2L	2	500	1535	4677
G2L	2	1000	1795	6177
G2L	2	2000	2110	8244
G2L	5.5	500	366	590
G2L	5.5	1000	425	685
G2L	5.5	2000	445	850

Analysis of Variance for Rutting @ 45C - Type III Sums of Squares

Source of variation	Sum of Squares	d.f.	Mean square	F	Sig. level	F-critical
MAIN EFFECTS						
A:Material	1532416.9	1	1532416.9	9.096	.0107	4.75
B:Percent	4085105.8	2	2042552.9	12.124	.0013	3.89
C:Repetitions	2121051.4	2	1060525.7	6.295	.0135	3.89
RESIDUAL	2021684.3	12	168473.69			
TOTAL (CORRECTED)	9760258.4	17				

Analysis of Variance for Rutting @ 60C - Type III Sums of Squares

Source of variation	Sum of Squares	d.f.	Mean square	F	Sig. level	F-critical
MAIN EFFECTS						
A:Material	8.3354E0006	1	8335445	11.766	.0050	4.75
B:Percent	1.1356E0008	2	5677882	8.0144	.010	3.89
C:Repetitions	1.6230E0007	2	8115097	11.455	.0016	3.89
RESIDUAL	8501553.2	12	708462.77			
TOTAL (CORRECTED)	1.4662E0008	17				

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