THE UNIVERSITY OF CALGARY

LABORATORY EVALUATION OF RUTTING BEHAVIOUR
OF VIRGIN AND RECYCLED
ASPHALT CONCRETE PAVEMENTS

BY

AJITH B. WIJERATNE

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The undersigned certify that they have read and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled "LABORATORY EVALUATION OF RUTTING BEHAVIOUR OF VIRGIN AND RECYCLED ASPHALT CONCRETE PAVEMENTS", submitted by AJITH B. WIJERATNE in partial fulfilment of the requirements for the degree of Master of Science in Engineering.

Michel Sargious

Dr. M.A. Sargious (Supervisor)
Department of Civil Engineering

J.E. Gillott

Dr. J.E. Gillott
Department of Civil Engineering

Dr. R.C. Joshi
Department of Civil Engineering

R.G. Moore

Dr. R.G. Moore
Department of Chemical and
Petroleum Engineering

August 12, 1986

ABSTRACT

At present there is an urgent demand for efficient techniques of pavement reconstruction in the highway and airport industry. This situation is created as a result of two concurrent factors. Firstly, is the increase in the demand for rehabilitation of existing pavements. Secondly, is the decrease in the available funds for construction and maintenance of pavements in highways and airports.

Since recycling of asphalt concrete is one such technique, this research was undertaken to study the rutting (permanent deformation) behaviour of recycled asphalt mixtures. To compare the behaviour of recycled asphalt concrete with that of virgin asphalt concrete, two mixtures were designed: a recycled mixture containing 48% reclaimed asphalt material, and a virgin control mixture.

Repeated load tri-axial tests were used to study the rutting in asphalt concrete by simulating the stress conditions occurring in asphalt concrete pavements subjected to vehicular traffic.

Forty two cylindrical asphalt concrete specimens of 101.5 mm diameter and 200.0 mm height were used in the test program which was conducted at a temperature of 20°C.

From the tri-axial tests, equations were developed for use in predicting rutting in asphalt concrete at any particular number of load repetitions, once the stress distribution in the pavement is known.

These rut prediction equations were also used to develop charts for determining the thickness of asphalt pavements made of recycled and virgin mixtures. A modified Chevron5L multi-layer system elastic

analysis program was used for this purpose.

These charts indicate that, based on rutting behaviour alone, a recycled asphalt concrete pavement requires a smaller thickness than a virgin asphalt concrete pavement carrying the same load for an identical number of load repetitions.

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NOTATIONS

σ_1, σ_2, σ_3 = principal stresses

σ_z = vertical stress

σ_r = radial stress

p = mean normal stress (stress invariant)

τ_{oct} = octahedral shear stress

q = stress invariant representing octahedral shear stress

v = volumetric strain

γ_{oct} = shear strain

ε_z = vertical strain

ε_r = radial strain

ε_1, ε_2, ε_3 = principal strains

D_N = total permanent deformation

ε_p = permanent strain

ε_{ip} = vertical permanent strain in the i^{th} layer

N = number of applied load repetitions

C_o, C_1 = regression coefficients

a_1, a_2, a_3 = regression coefficients

b_1, b_2 = regression coefficients

ν = Poisson's ratio

M_R = resilient modulus

CHAPTER 1

INTRODUCTION

1.1 Overview of Pavement Recycling

The increase in the demand for rehabilitation of existing highway and airport pavements accompanied by the decrease in the available funds for construction and maintenance have resulted in development of new, more efficient techniques for reconstruction. The recycling of pavement is one such technique.

Although the use of the pavement recycling technique on a wide scale started in U.S.A. in 1975, it was first used in Singapore in 1915.

Starting in U.S.A., this technique has been successfully used in countries like, Germany, Netherlands, Canada, etc.

Research in the technique of recycling started in early 70's. This resulted in modifications of Drum and Batch mixers and other equipment in order to handle recycling.

1.2 Study Objectives

To design good quality pavements, constructed from recycled materials, characterization of the material used is essential. This can be achieved by simulating field stress conditions in the laboratory. Another property of recycled material that can be investigated is the long term behaviour, which is necessary to obtain information regarding the durability of the pavement. These experimental results can also be used in economic evaluations to estimate relevant costs.

In this study an effort is made to investigate the rutting behaviour of recycled asphalt concrete as well as to compare the performance of hot-mixed recycled asphalt concrete with that of virgin asphalt concrete with respect to rutting.

1.3 Scope of the Study

In Chapter 2 a literature review on pavement recycling and rutting is presented. Chapter 3 deals with the design of virgin and recycled mixes. In Chapter 4, past development and details of the repeated load tri-axial test are discussed. The tri-axial test allows the development of a relationship between applied stress and the resulting permanent strain in asphaltic concrete. From the results of this test it is also possible to develop a relationship between permanent strain and the number of loading cycles for any particular applied stress as well as to calculate resilient modulus and Poisson's ratio of the material.

In Chapter 5, the results of the laboratory tests are analyzed and predictive relationships for rutting due to repeated loading are developed.

Charts are developed for use in determining the thickness of pavements by limiting rutting to a specified amount during the design period. The Chevron5L multi-layer elastic analysis program was used for this purpose. These charts can mainly be used to compare the performance of virgin and recycled asphalt concretes. They are presented in Chapter 6.

Discussion, conclusions and recommendations are presented in Chapter 7.

CHAPTER 2

LITERATURE REVIEW

2.1 Definition of Asphalt Pavement Recycling

The life span of an asphalt pavement is determined by several factors such as the volume and weight of traffic, weather conditions, quality of construction materials, drainage etc. Also good maintenance is critically important for extending the pavement life.

However, when the pavement ages, it starts to deteriorate. But the old pavement material need not be wasted. It can be used in the process of rebuilding a new pavement.

This re-use of old asphalt pavement material which has served its first intended purpose, usually after some processing and mixing with virgin material to build a new road, is called asphalt pavement recycling.

2.2 Recycling as an Alternative

Several methods can be used to rehabilitate a distressed asphalt pavement.

Before choosing the method of rehabilitation, the cause of distress has to be established. This is usually done based on the:

1. original pavement design and construction records
2. field tests (i.e. deflection, surface condition, skid resistance etc.) and
3. field samples and laboratory tests.

Then rehabilitation methods can be evaluated on the basis of pavement design principles, environmental influences, expected traffic conditions, highway geometrics and economics.

Recycling has the following major advantages when compared with other forms of rehabilitation techniques such as overlays, seal coats, totally new construction etc.

1. conservation of aggregates and binder
2. conservation of energy
3. preservation of environment and existing highway geometrics
4. structurally comparable with new asphalt concrete pavements.

2.3 Types of Asphalt Pavement Recycling

There are three basic types of asphalt pavement recycling. They are:

1. surface recycling (hot or cold process)
2. in-place surface and base recycling (hot or cold process)
3. central plant recycling (hot or cold process)

Surface Recycling

It involves reworking the surface of a pavement to a depth of less than 25 mm (1 inch). Hence surface recycling has limited usefulness in repairing severely rutted roads or in increasing the load carrying capacity of a road. But it can be used to treat most of the pavement distress while the pavement is still performing at a moderate level.

The different surface recycling techniques are illustrated in figure 2.1.

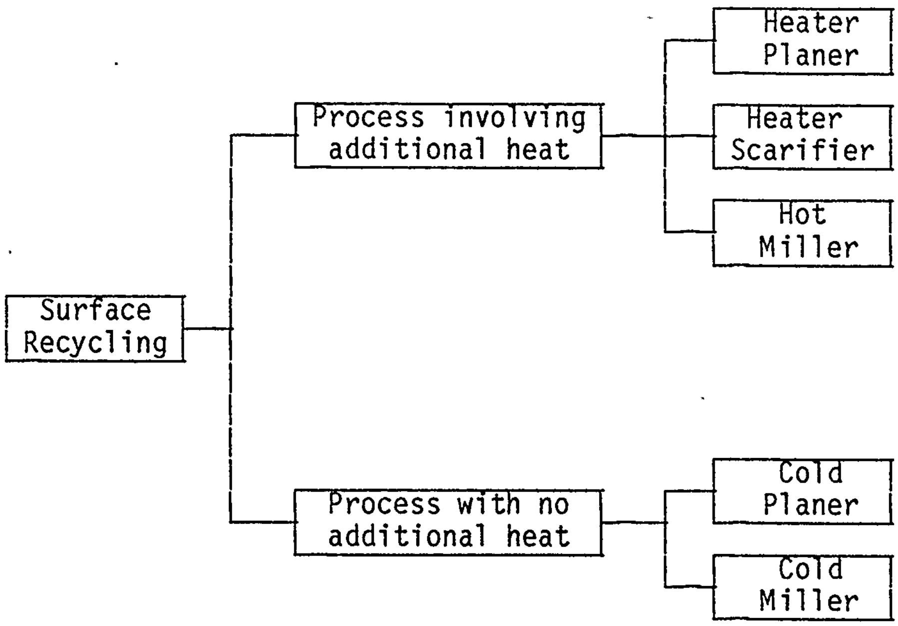


Figure 2.1 Surface Recycling Technqiues

(Source: Epps et al, 1980)

<u>In-place Surface and Base Recycling</u>

This involves pulverization of existing pavement to a depth greater than about 25 mm (1 inch), followed by reshaping and compaction. This operation may be performed with or without the addition of a stabilizer.

In-place recycling has the ability to improve the load carrying capability of a pavement without changing the highway geometry. Also it can treat almost all types of pavement distress. Alternatives of in-place recycling are illustrated in Figure 2.2.

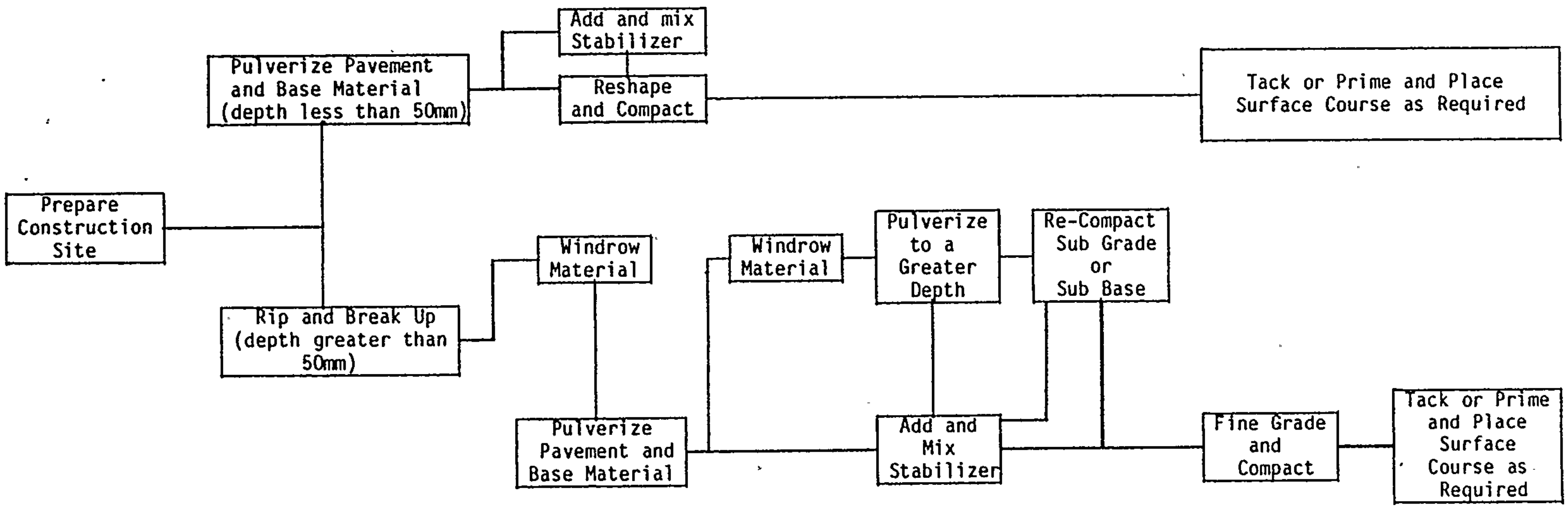


Figure 2.2 In-Place Recycling Alternatives

(Source: Epps et al, 1980)

Central Plant Recycling

This process involves, scarification of the pavement material, pulverization prior to or after the removal of the pavement from the road, processing of the material with or without the addition of a stabilizer at a central plant, laying down and compacting to a desired grade at the road site. Processing at the central plant could involve additional heat depending on the type of material recycled and stabilizer used.

The process with additional heat (hot-mix recycling) is the most commonly used technique in the field.

Central plant hot-mix recycling has all the advantages of in-place surface and base recycling. In addition, this technique allows better quality control. Also in this case it is easy to alter the highway geometrics.

Alternatives available in central plant recycling are illustrated in Figure 2.3.

2.4 Major Advantages and Disadvantages of Different Recycling Techniques

A summary of the advantages and disadvantages of the three recycling techniques are given in Table 2.1.

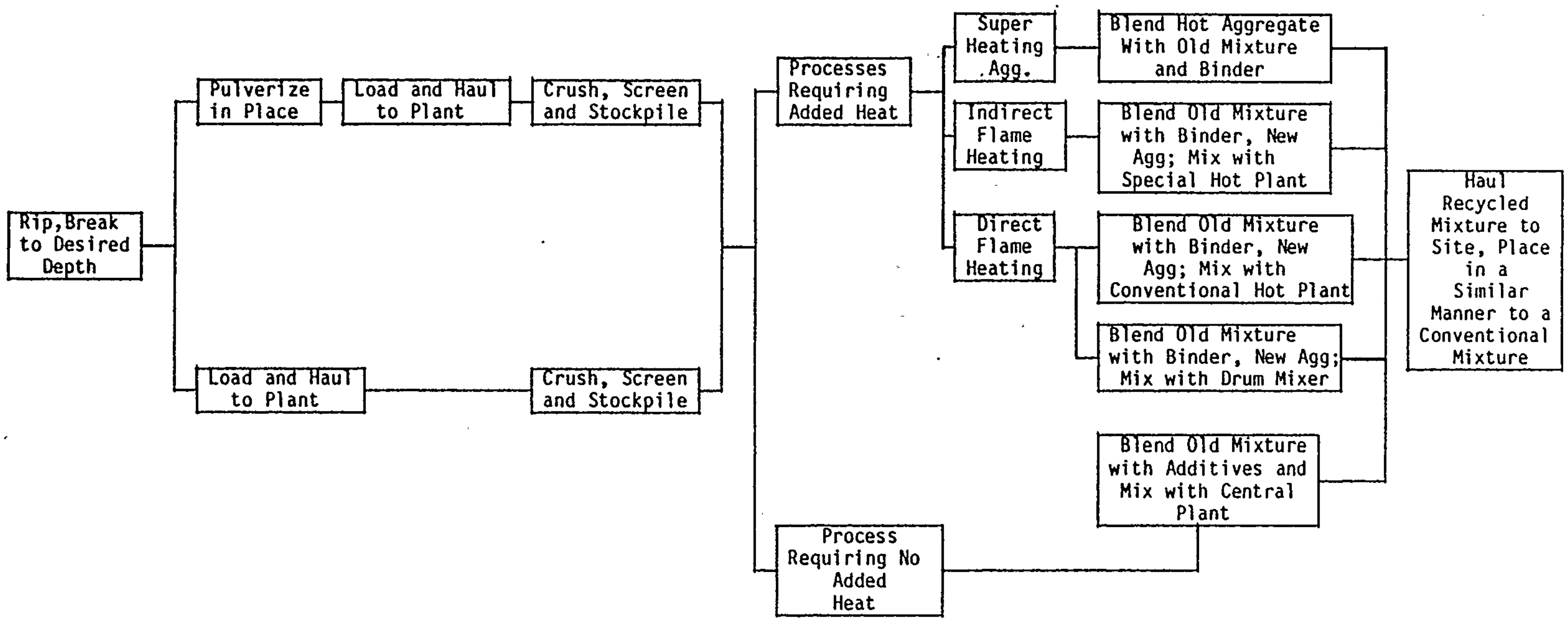


Figure 2.3 Central Plant Recycling Alternatives

(Source: Epps et al, 1980)

Table 2.1 Major Advantages and Disadvantages of Recycling Techniques

Recycling Techniques	Advantages	Disadvantages
Surface	° Reduces frequency of reflection cracking ° Promotes bond between old pavement and thin overlay ° Provides a transition between new overlay and existing gutter, bridge, pavement, etc. that is resistant to raveling (eliminates feathering) ° Reduces localized roughness due to compaction ° Treats a variety of types of pavement distress (raveling, flushing, corrugations, rutting, oxidized pavement, faulting) at a reasonable initial cost ° Improved skid resistance	° Limited structural improvement ° Heater-scarification and heater-planning has limited effectiveness on rough pavement without multiple passes of equipment ° Limited repair of severely flushed or unstable pavements ° Some air quality problems ° Vegetation close to roadway may be damaged ° Mixtures with maximum size aggregates greater than 50 mm cannot be treated with some equipment
In-Place	° Significant structural improvements ° Treats all types and degrees of pavement distress ° Reflection cracking can be eliminated ° Frost susceptibility may be improved ° Improve skid resistance	° Quality control not as good as central plant ° Traffic disruption ° Pulverization equipment in need of frequent repair
Central	° Significant structural improvements ° Improved quality control ° Treats all types and degrees of pavement distress ° Reflection cracking can be eliminated ° Improve skid resistance ° Frost susceptibility may be improved ° Geometrics can be more easily altered ° Improved quality control if additional binder and/or aggregates must be used ° Improve ride quality	° Increased disruption ° Potential air quality problems at plant site ° Traffic disruption

2.5 Economics of Asphalt Pavement Recycling

According to reports on a large number of case studies that cover a variety of asphalt recycling projects carried out in different countries, asphalt pavement recycling is an economical process of pavement reconstruction. But the long term effects and costs of most of the asphalt recycling projects are yet to be reported. Hence, it is still too early to state the real savings achieved by asphalt recycling.

The three major economic benefits that are offered by asphalt recycling are:

1. cost reductions
2. energy savings, and
3. conservation of natural resources

Many agencies have reported cost savings by the use of asphalt recycling technique for highway rehabilitation as compared to other alternative techniques. For example LaHue (1980) has reported the following savings.

1. \$146,000 saved on a 47,896 ton project.

   i.e. \$3.05 per ton savings (Wyoming)

2. \$59,385 saved on a 60,700 ton project

   i.e. \$0.98 per ton savings (Oregon)

3. \$138,418 saved on a 42,129 ton project

   i.e. \$3.29 per ton savings (Iowa)

In Netherlands, as reported by Gerardu and Hendriks (1985), recycled asphalt has been found to be 15-20% cheaper than new asphalt. The above cost savings have been achieved inspite of the fact that most of the contractors tend to recover the costs of plant modifications in one recycling project. This is because recycling is a relatively new construction process.

When reports from past projects are considered it is reasonable to assume that recycling can offer cost savings in the range of 5-20% compared to other conventional rehabilitation methods.

Recycling also conserves a substantial amount of energy compared to other rehabilitation methods. The major factors that affect the energy savings achieved by recycling are:

1. virgin aggregate haul distance
2. amount of new asphalt cement required
3. asphalt cement haul distance
4. haul distance from project to the crushing/mixing plant.

As reported by LaHue (1980) some recent projects have shown energy savings as much as:

1. 1.9 billion BTU's on a 53,000 ton project
2. 3.8 billion BTU's on a 47,900 ton project
3. 151 million BTU's on a 60,700 ton project.

Also Halstead (1980), has reported details of energy savings in 25 projects, where energy savings were in the range of 70-7730 equivalent gallons of diesel fuel per lane-mile of the road constructed.

The asphalt recycling conserves both asphalt cement and virgin aggregate. The conservation of asphalt cement is very important as it is a petroleum product which has a high energy content as well as being expensive.

Conservation of aggregate is also important due to the fact that most of the aggregate sources near major cities have already been consumed.

As reported by LaHue, accumulated savings in 27 major hot-recycling projects are:

recycled mix (Total)	1,182,000 tons
virgin agg. conserved	771,000 tons
asphalt cement conserved	42,800 tons.

Hence considering such factors as the present trend of higher highway construction costs and the shortage of funds for highway maintenance, the asphalt pavement recycling seems to offer an ideal answer for rehabilitation of highways in the future.

2.6 The Methodology of Central Plant Hot-Mix Recycling in the Field

Once the recycling decision is made based on a cost benefit analysis and some preliminary laboratory tests, the first step would be to break up the existing pavement. This material is then transported to the plant where this old asphalt concrete material can be reduced to an acceptable size (for example 25 mm) by passing it through two crushers.

In the North American practice, these two crushers will reduce the maximum particle size in the reclaimed asphalt concrete to about 25 mm.

In the Netherlands, the reclaimed asphalt concrete is first sent through a crusher to reduce its particles to about 40 mm maximum size. Then it is steam treated at atmospheric pressure in order to reduce the size further by seperating aggregate particles.

At this stage, samples are taken to determine the composition of the reclaimed asphalt concrete and the properties of the aged bitumen.

From the results of the above tests, the properties of the new bitumen and aggregate which may be necessary for mixing with reclaimed material, can be determined. Afterwards, the Marshall test can be used to design the recycled asphalt concrete mix.

After the reclaimed pavement material is reduced to an acceptable size, it is mixed with new aggregate and bitumen according to the mix design.

To carry out this mixing operation there are many types of mixing equipment available. Central plant equipment that is available for the recycling operation can be divided into three categories.

1. direct flame heating
2. indirect flame heating, and
3. equipment in which heat transfer from superheated aggregate is used to heat the reclaimed material.

Direct flame heating is usually associated with drum mixers. However, direct flame heating drum mixers cannot be directly used for asphalt recycling as the flame tends to oxidize aged asphalt giving 'blue smoke', which causes high air pollution.

Many modifications have been done to the drum mixers to solve the above problem. These are:

1. Drum mixer with heat dispersion shield. In this case the heat shield and the introduction of cooling air are used to reduce the temperature of hot gases so that the 'blue smoke' formation is reduced. This type (illustrated in figure 2.4) can successfully recycle mixes containing up to 70% reclaimed asphalt concrete.
2. Drum within a drum plant. In this process virgin aggregate is first superheated in the first drum and reclaimed asphalt concrete is mixed with superheated aggregate in the second drum. Hence heat transfer from superheated aggregate is used to heat up the reclaimed material. This type (illustrated in figure 2.5) can recycle mixes containing up to 50-60% reclaimed asphalt concrete.
3. Drum mixer with split feed. In this case new aggregate is introduced at the flame end of the drum where it is superheated to about 150°-260° C. The reclaimed material is fed at the middle of the drum. Hence, it gets heated up due to heat transfer from virgin aggregate as well as due to hot gases. This plant (illustrated in figure 2.6) can successfully recycle mixes, which contain up to 70% reclaimed bituminous material.
4. Drum mixers are sometimes fitted with heating tubes so that reclaimed material can be heated indirectly.

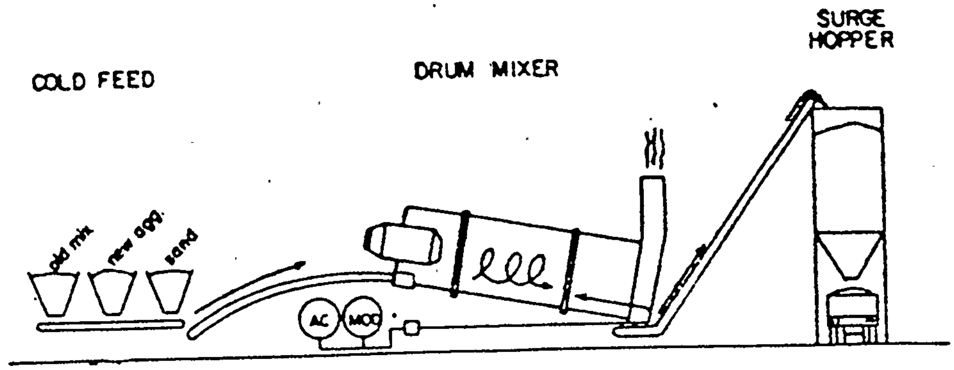


Figure 2.4 Drum Mixer with Heat Dispersion Shield

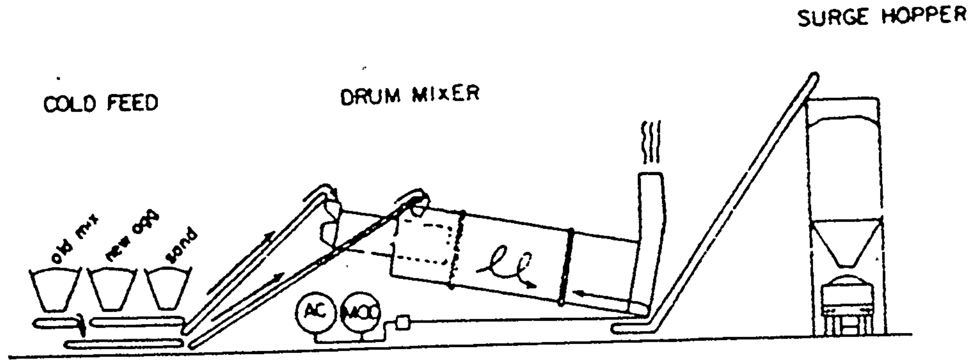


Figure 2.5 Drum Within a Drum Plant

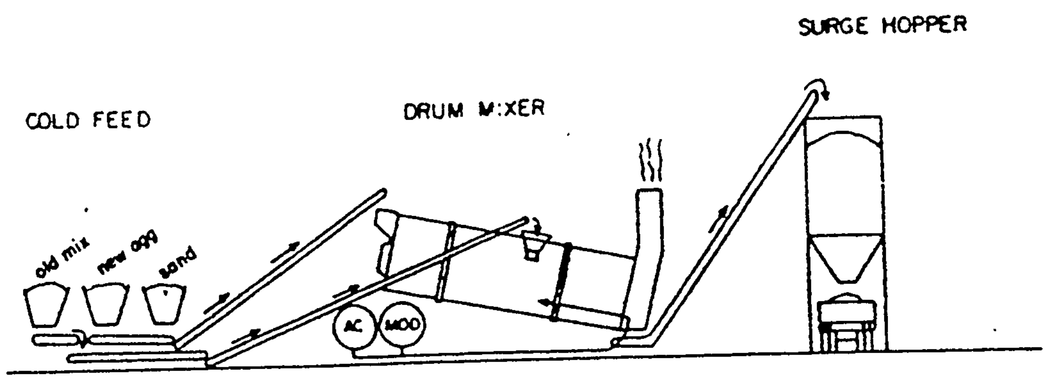


Figure 2.6 Drum Mixer with Split Feed

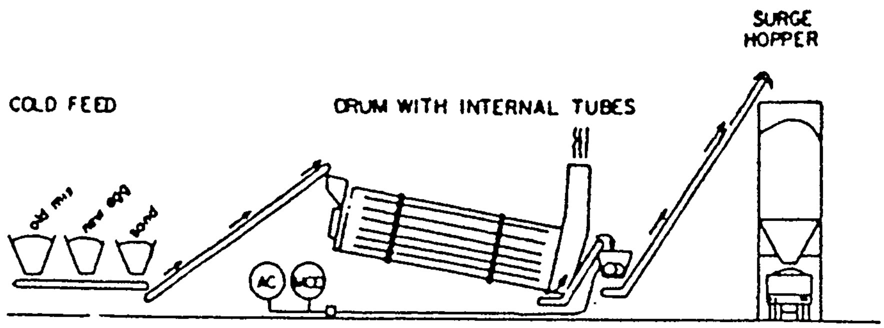


Figure 2.7 Special Drum Mixer with Heat Exchanger Tubes

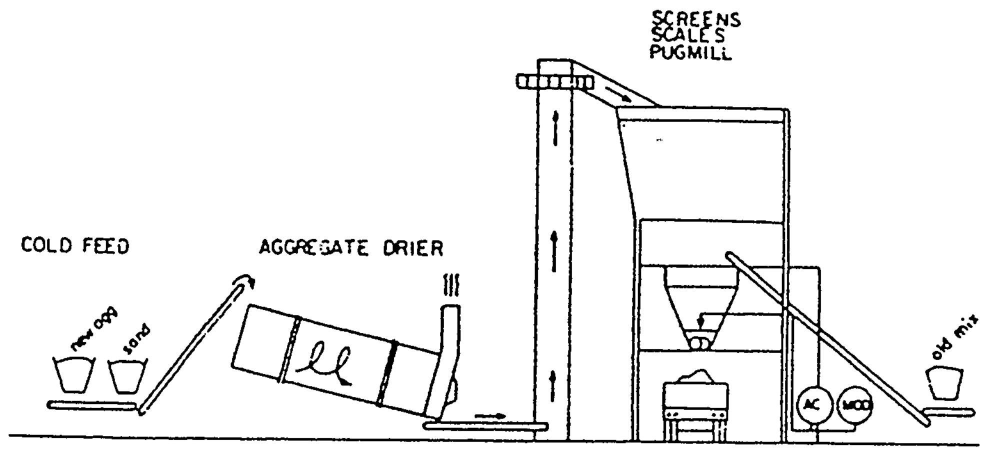


Figure 2.8 Standard Batching Plant with Old Mix Added to Superheated Aggregate at the Pug Mill

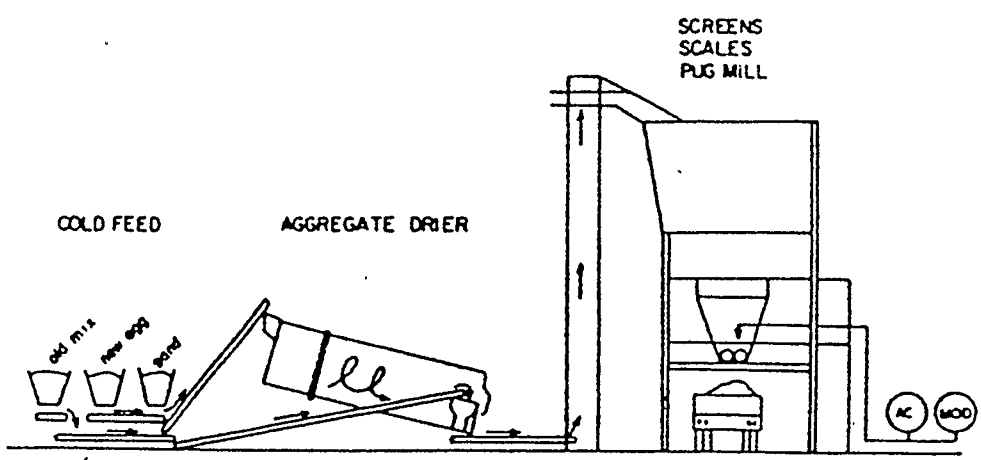


Figure 2.9 Standard Batching Plant with Old Mix Added to Superheated Aggregate at Drier Discharge

Hot gases are passed through these tubes, so that heat is transferred to recycling mixes. This type of equipment can recycle up to 100% reclaimed asphalt concrete; it is illustrated in figure 2.7.

Standard batching plants can also be used for asphalt recycling after some modification. The heat transfer from superheated aggregate is the method adopted in these plants to heat the reclaimed asphalt concrete material. Two types of these batching plants used for asphalt recycling are shown in figures 2.8 and 2.9.

In the Netherlands, the drum mixers are used for asphalt recycling. Both direct and indirect heating is partially used in these drum mixers. It is called the Rehofalt process as reported by Hendriks and Gerardu (1985). These drum mixers can recycle material which contains up to 100% reclaimed asphalt concrete.

In other countries in Europe, techniques similar to those used in U.S.A are adopted for asphalt recycling.

2.7 Design of Recycled Asphalt Concrete Mixes

Recycled asphalt mix design is a process that consists of three important steps. These are: a) the laboratory evaluation of reclaimed material, b) selection of the amount of virgin aggregate, the proper bituminous modifier and the amount, that has to be mixed with reclaimed pavement material and c) the determination of the optimum bitumen content in the final mix using the Marshall test.

2.7.1 Laboratory Evaluation of the Reclaimed Asphalt Pavement (RAP)

This consists mainly of determining the properties of the aggregate, asphalt and the binder content.

In order to determine the properties of aggregate, the asphalt has to be extracted from the RAP. This is achieved by extracting the asphalt from asphalt concrete using a suitable solvent and a centrifuge extractor (ASTM D2172-81) and subsequent recovery of asphalt from solution by the Abson method (ASTM D1856-79). The gradation of the reclaimed aggregate material (RAM) is then determined by performing a sieve analysis. Usually, due to the action of the traffic and the method of removal of old asphalt pavement, the gradation of reclaimed aggregate tends to be fine. Hence it may be necessary to add virgin aggregate in order to bring the resultant gradation of the aggregate in the recycled mix within the limits specified by the ASTM D3515. The percentage of asphalt in the RAP is calculated from the weights of recovered aggregate and asphalt. The recovered asphalt is tested for viscosity at 60°C and penetration at 25°C. Reclaimed asphalts generally show high viscosities and low penetration values indicating aging during service in the pavement. The amount of aging can be measured by comparing the viscosity of the reclaimed asphalt with the viscosity of the virgin asphalt after aging in a thin film oven. The thin film oven test represents the aging of the asphalt during the mixing process of asphalt concrete.

2.7.2 Selection of a Bituminous Modifier

As mentioned earlier the reclaimed asphalt has a high viscosity and a low penetration values. This means a consistancy that makes it unsuitable for use in an asphalt concrete mix. Therefore, it has to be modified in such a manner that it will regain a good consistancy and durability in order that it will yield a workable asphalt concrete mix resistant to aging during service. This can be achieved by selecting the proper modifier that has to be mixed with RAP. To carry out the proper selection it is necessary to examine the components of asphalt which play a critical role in defining its properties, particularly the durability and viscosity. Rostler has investigated this aspect in depth (Rostler & White, 1962; Rostler et al, 1972 and Rostler, 1981). He has identified five basic components in asphalt. These components and their functions are presented in Table 2.2 and figure 2.10. The asphalt consistancy is mainly governed by the viscosity of the maltene fraction and the percentage of asphaltenes present in the asphalt.

The aging of asphalt is mainly an oxidation process where maltenes are converted to asphaltenes. Hence during aging, the ratio $(N+A_1/P+A_2)$ of the asphalt decreases. This is the ratio of more reactive components to least reactive components. It represents the durability of asphalt. However, the previous consistancy and durability can be achieved by mixing aged asphalt with a suitable modifier. The

Table 2.2 Characteristics of Asphalt Components

FRACTION	GENERAL DISCRIPTION	SPECIFIC CHEMICAL ACTIVITY	FUNCTION	REACTIVITY
Asphaltenes (A)	Higher molecular weight condensation products	Insoluble in n-pentane	Bodying agent	low
Nitrogen bases (N)	Maltene fraction containing all nitrogen compounds.	Precipitate with 85% sulfuric acid	Peptizer for asphaltenes	high
First acidaffins (A_1)	Unsaturated resinous hydrocarbons	Precipitable with concentrated sulfuric acid	Solvent for peptized asphaltenes	high
Second acidaffins (A_2)	Slightly unsaturated hydrocarbons	Precipitable with fuming sulfuric acid (30% So_3)	Solvent for peptized asphaltenes	low
Paraffins (P)	Saturated hydrocarbons	Nonreactive with fuming sulfuric acid	Gelling agent for asphaltenes	low

(Source: Rostler, 1981)

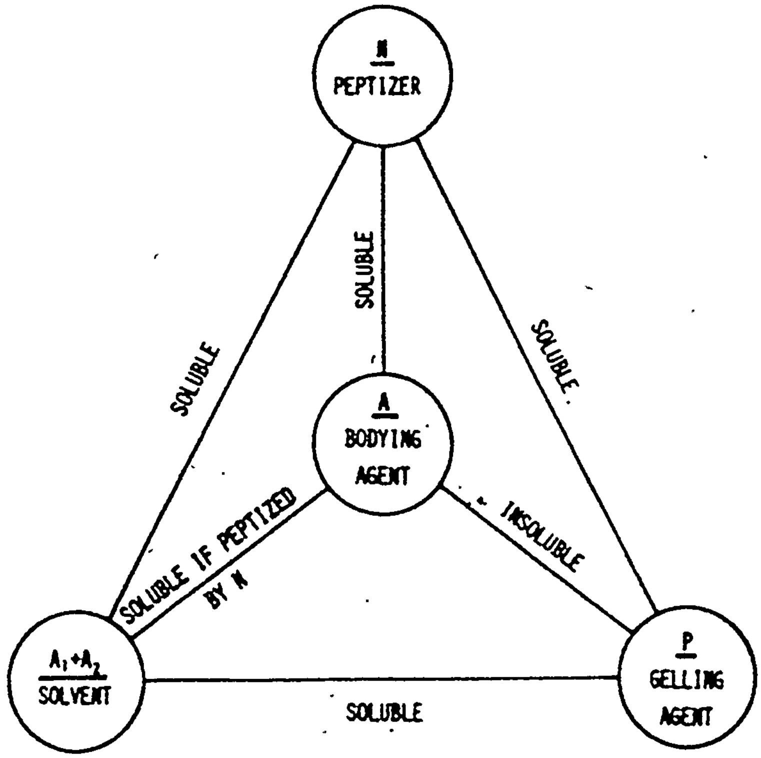


Figure 2.10 Relationships Between Various Components in Asphalt

(Source: Rostler, 1981)

effectiveness of the modifiers can be characterized by two ratios. They are the durability parameter $(N+A_1/P+A_2)$ and N/P, where N/P indicates the susceptibility to syneresis (i.e. the exudation of paraffins from the asphalt). Guidelines are available for bituminous modifiers used for recycling where ranges of values for above ratios and other properties are specified (Davidson et al, 1977; Davidson and Escobar, 1979; Kari et al, 1980 and Epps et al, 1980). Specifications for recycling agents are presented in Table 2.3. Dunning and Mendenhall, (1977) used a different technique to measure the effectiveness of recycling agents. They defined two effectiveness ratios based on viscosity and also they recommended minimum limits of 9% and 60% for polar and aromatic compounds in modifiers, respectively. The ability of recycling agents to modify aged asphalts is also evaluated by the asphaltene setting rate test. This test measures the degree of dispersion of asphaltenes by maltenes and hence, the compatibility of asphalt components (Plancher et al, 1979 and Gannon et al, 1980).

2.7.3 Calculation of the Percentage of Asphalt required in Recycled Asphalt Concrete Mixes

The Asphalt Institute Manual No-20, (1981) gives the following equation to calculate the asphalt demand of an aggregate mix.

Table 2.3 Proposed Specifications for Hot Mix Recycling Agents[1]

Test	ASTM TEST METHOD	RA 5		RA 25		RA 75		RA 250		RA 500	
		min.	max.	min.	max.	min.	max.	min.	max.	min.	max.
Viscosity @140°F cp	D2170 or 2171	200	800	1000	4000	5000	10000	15000	35000	40000	60000
Flash Point COC, °F	D92	400	-	425	-	450	-	450	-	450	-
Saturates, wt. %	D2007	-	30	-	30	-	30	-	30	-	30
Residue from RTF-C Oven Test @325°F	D2872[2]										
Viscosity Ratio[3]	-	-	3	-	3	-	3	-	3	-	3
RTF-Oven Weight Change %	D2872[2]	-	4	-	4	-	4	-	4	-	4
Specific Gravity	D 70 or D1298	Report		Report		Report		Report		Report	

1. The final acceptance of recycling agents meeting this specification is subject to the compliance of the reconstituted asphalt blends with current asphalt specifications.

2. The use of ASTM D1754 has not been studied in the context of this specification, however, it may be applicable. In cases of dispute the reference method shall be ASTM D2872.

3. $$\text{Viscosity Ratio} = \frac{\text{RTF-Viscosity at 140°F, cp}}{\text{Original Viscosity at 140°F, cp}}$$

°C = 5/9 (°F = -32)
1 cp = 0.001 Pa.s

(Source: Epps et al, 1980)

$$P_c = 0.035a + 0.045b + \begin{cases} 0.15c \text{ for 11-15 percent passing No. 200} \\ 0.18c \text{ for 6-10 percent passing No. 200} \\ 0.20c \text{ for 5 percent or less passing No. 200} \end{cases} + F \qquad (2.1)$$

where, Pc = percent of asphalt by weight of total mix

a = percent of mineral aggregate retained on No. 8 sieve.

b = percent of mineral aggregate passing No. 8 sieve and retained on No. 200 sieve

c = percent of mineral aggregate passing No. 200 sieve

F = 0.0 - 2.0 percent. Based on absorption of light or heavy aggregate.

Once the reclaimed aggregate is modified by addition of virgin material, such that the resultant gradation falls within the limits specified by ASTM D3515, the asphalt demand of the resultant mix can be calculated by the above equation. The next step is to calculate the amount of new asphalt that has to be added to the recycled mixture. It can be found from the following formula.

$$Pr = Pc - (Pa \times Pp) \qquad (2.2)$$

where, Pr = percent new asphalt in the recycled mix

Pc = percent of asphalt by weight of total mix

Pa = percent of asphalt in the reclaimed pavement

Pp = decimal percent reclaimed material in the recycled mix.

The quantity of new binder as a percentage of total binder = (Pr/Pc) x 100.

The viscosity of virgin soft asphalt that has to be added to the recycled mix can be found by using the asphalt blending chart shown in figure 2.11. It is very important that the soft asphalt or the recycling agent chosen using the blending chart has other relevant properties specified in Table 2.3.

After selecting a bituminous modifier suitable for recycling the aged asphalt pavement, the optimum asphalt content in the recycled asphalt concrete mix is found by performing Marshall tests.

2.8 Factors Affecting Performance of Pavements

In order to identify the factors that affect the performance of pavements, the failure modes of pavements have to be examined. There are three basic types of pavement failures:

1. cracking due to fatigue
2. vertical permanent deformation (rutting)
3. cracking due to thermal effects

The cracking due to fatigue could be caused by the following factors:

I. repeated loading; which causes continuous changes in stresses and strains in the pavement

II. temperature variations; which cause reversal of stresses and strains in the pavement.

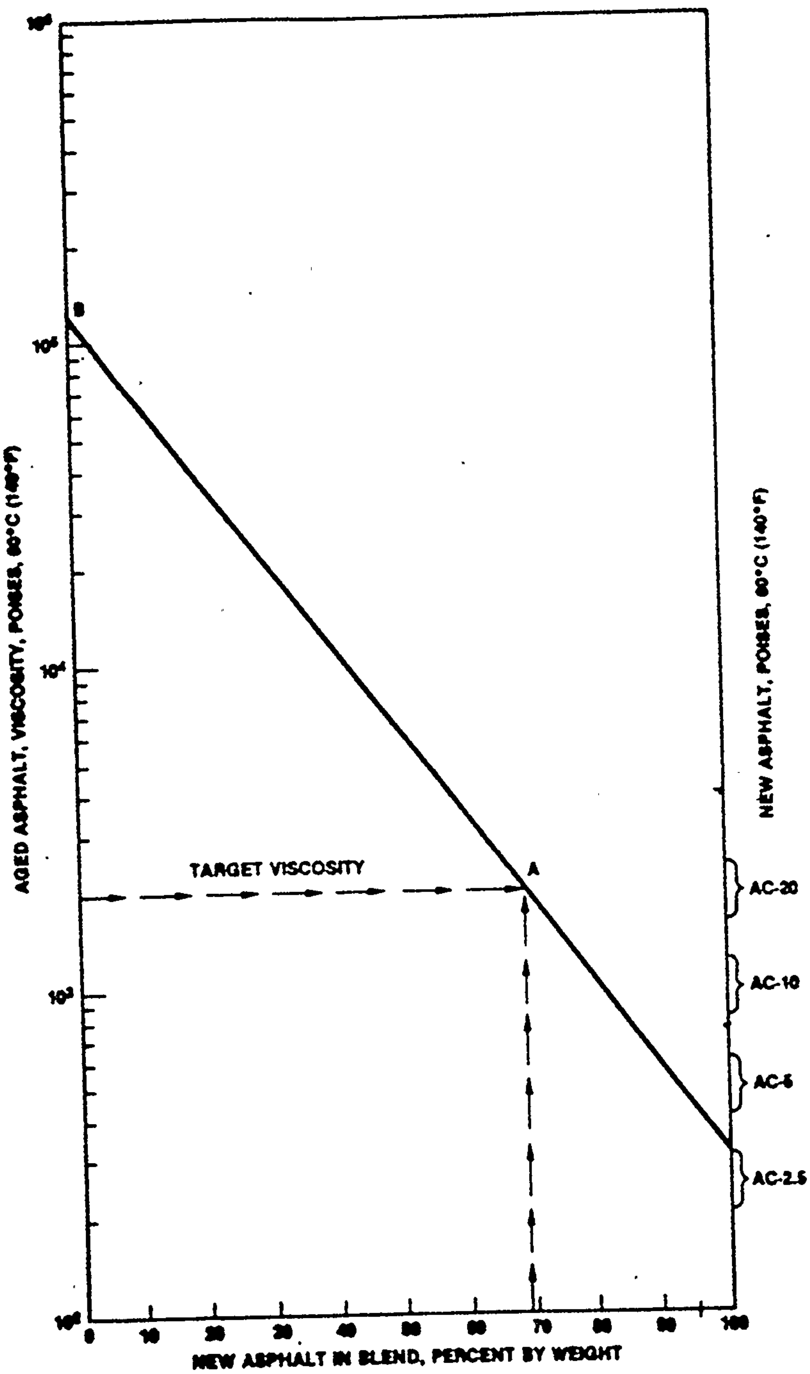


Figure 2.11 Blending Chart for Reclaimed Asphalt and New Asphalt Cements

(Source: Asphalt Institute)

The rutting can be divided into two categories depending on the factors that cause it.

a) rutting due to applied loads

b) rutting due to environmental effects

The rutting due to applied loads is caused by three types of loads.

I. repeated loads, which cause permanent strains to accumulate in the asphalt bound layer.

II. long term loads, which cause creep in the asphalt bound layer

III. excessive loads, which cause shear failure

The environmental effects can be due to many reasons. Some of these are:

I. expansive subgrade soil which causes heaving

II. compressible material underlying the pavement which causes settlements.

III. frost susceptible material which can cause heaving.

IV. weakened subgrade due to spring thawing which could lead to settlements.

The thermal cracking is mainly due to contraction which causes tensile strains in the pavement, when pavement is subjected to low temperatures.

2.9 Past Work in the Field of Pavement Recycling

In the past, research on pavement design has mainly concentrated on investigations of the behaviour of virgin material. The behaviour of virgin asphalt has been investigated with respect to:

1. cracking due to fatigue
2. rutting due to repeated loading and long term loading
3. thermal cracking

The properties of recycled asphalt also have been investigated (to a lesser extent when compared to virgin asphalt) with respect to its potential for cracking due to fatigue by a few people (Shethata, 1984 and Little, Epps and Holmgreen, 1982).

The behaviour of recycled asphalt concrete with respect to rutting has been very briefly investigated by Little (Little et al, 1982). They have tested recycled asphalt concrete samples where the pavement material used in recycling was taken from highways in California and Oregon.

The rutting potential of recycled asphalt mixes in which aged asphalt material is reclaimed from pavements in Canada has not yet been investigated.

Canadian pavements are subjected to much more severe climatic conditions than are the pavements in the southern states of U.S.A. such as Texas and California. Hence results obtained for these pavement materials cannot be directly applied to Canadian conditions.

In view of the present trend of emphasis on rehabilitation of pavements rather than constructing new roads and because of the failure of many pavements by rutting, it is quite appropriate to investigate the rutting potential of recycled asphalt material. This investigation forms the main topic of the present thesis.

2.10 Permanent Deformation Due to Repeated Loading

Permanent deformation (rutting) due to repeated loading can be estimated using two approaches.

In the first method the vertical compressive strain at the subgrade surface is limited to some tolerable amount at a specific number of load repetitions. This strain level can be maintained below the specified value by using materials of adequate stiffness and sufficient thickness in conjunction with proper construction procedures.

In the second method the actual rutting is estimated by using appropriate information on material characterization and by a stress analysis of the pavement structure, treating it as a layered elastic system.

The latter method is superior due to the fact that it can be used to predict the rutting in a pavement after any specified number of load cycles, whereas the first one is an empirical technique which depends on data collected from several test roads and can only be used to limit rutting at a specific number of load repetitions.

2.10.1 Estimation of Rutting in a Pavement Using Material Characterization Information

There are several procedures which can be categorized under this approach, which use material characterization information in order to estimate rutting.

The first procedure is the use of an elastic layered system to represent the pavement structure, with the materials characterized by repeated load triaxial compression tests. The second method is the use of a viscoelastic layered system to represent the pavement structure with materials characterized by means of creep tests.

The first procedure has been found to give realistic results in predicting deformation in several test roads. It also has the advantage over the second method due to the fact that it is more simple and easy to use in practice.

2.10.2 Consideration of the Pavement as an Elastic Layered System in Order to Estimate Rutting

By representing the pavement as an elastic layered system, the stresses in the pavement due to surface loading can be computed. This was shown by several researchers (Burmister, 1945; Barksdale, 1972 and Romain, 1972). These stresses can then be used with relationships obtained from material characterization to estimate rutting at some specified number of load repetitions. This type of analysis requires two relationships that have to be obtained from material characterization experiments. The first one is a relationship between permanent strain and applied stress for each of the pavement components, i.e.;

$$\varepsilon_p = f\ (\ \sigma_{ij}\ ) \qquad (2.3)$$

where, ε_p = permanent strain

σ_{ij} = stress state

The second is a relationship between permanent strain and number of applied load cycles.

With these relationships, it is possible to estimate the permanent deformation occuring in the asphalt concrete layer by sub dividing it to several sub-layers. The permanent strain at the mid depth of each sub-layer is computed so that it is possible to approximately define the strain variation with depth. The permanent deformation in each sub-layer is the product of the average permanent strain and the thickness of the sub-layer. The total rut depth can be estimated by summing the contributions from each sub-layer. From the knowledge of the permanent strain at various numbers of load repetitions, the development of rutting with traffic can be estimated. Several investigators have used this approach to predict permanent deformation in either the asphalt bound layer or of the full pavement structure (Barksdale, 1972; Morris et al, 1974; Monismith, 1976; McLean and Monismith, 1975; Brown and Snaith, 1974; Brown and Bell, 1977; Brown and Bell, 1979 and Allen and Deen, 1981).

2.10.3 Empirical Relationships Obtained from Material Characterization

Several researchers have used the repeated load tri-axial test to obtain the above mentioned relationships for virgin asphalt concrete (Morris et al, 1974; McLean and Monismith, 1975; Francken, 1977; Brown and Bell, 1979 and Allen and Deen, 1981).

Morris and others have fitted laboratory test data over a range of temperatures and stresses to a regression equation of the form:

$$\varepsilon_p = f(\sigma_1, \sigma_3, T, N) \pm E \qquad (2.4)$$

where E = is the estimate of Error

McLean and Monismith have fitted a polynomial of the following form to their test data.

$$\log(\varepsilon_p) = C_0 + C_1 (\log N) + C_2 (\log N)^2 + C_3 (\log N)^3 \qquad (2.5)$$

The influences of stress, time, loading and temperature are included in the coefficients C_0, C_1, C_2 and C_3.

Francken has developed a relationship from laboratory tests conducted at the centre for road research in Belgium. The form of the equation is:

$$\varepsilon_p(t) = A\, t^B \qquad (2.6)$$

A and B are coefficients that are determined from regression, while t is the time period after the start of the application of load repetitions.

Brown and Bell fitted their laboratory test data to a similar relationship of the form:

$$\log(\varepsilon_p) = A + B(\log t - 1) \qquad (2.7)$$

Allen and Deen obtained an expression identical to that of McLean and Monismith from their tests on asphalt concrete.

The above mentioned technique have been used to predict rutting occuring in several test roads. The overall results are encouraging as shown by the reasonably close prediction of measured values of rutting.

CHAPTER 3

DESIGN OF VIRGIN AND RECYCLED ASPHALT MIXES

3.1 Objectives in Mix Design

As the purpose of this research is to investigate the economic advantage of recycled asphalt concrete in comparison to the virgin asphalt concrete, the objective in designing the recycled asphalt concrete mix is to obtain an optimum mix with basic properties comparable to that of an optimum virgin asphalt concrete mix. In order to use either drum mixers or batching plants to produce recycled mixes, the percentage of reclaimed material in the recycled mix should not exceed 50%. This limitation is due to the pollution created by the blue smoke emitted in heating reclaimed asphalt concrete. Hence a reclaimed material percentage of 48% was chosen in this study for the recycled mix in order to maximize the amount of reclaimed material within the above constraint.

The virgin mix is designed so that it is similar to the recycled mix with respect to aggregate gradation, asphalt viscosity and Marshall test properties.

3.2 Laboratory Analysis of Reclaimed Material

In this research old asphalt concrete was obtained from the pavement of a major arterial in Calgary, Alberta, Canada, for use in the recycled mix. This is a 125 mm thick full depth asphalt pavement that has been in service for 10 years.

The old asphalt material was heated, broken down into small sized particles and mixed together in order to make up a homogeneous material. In order to perform a sieve analysis on the aggregate in the reclaimed material, the asphalt had to be extracted. This was achieved by first extracting the asphalt into a solution by using tri-chloroethane and a centrifugal extractor. Then asphalt was separated from the solution by distillation.

When wet sieve analysis was performed on recovered aggregate from old asphalt concrete, it was found that the aggregate gradation of reclaimed aggregate was within the limits specified in D3515 ASTM standard for bituminous paving mixtures having 25 mm maximum size aggregate. The reclaimed aggregate gradation is shown in Table 3.1 and Figure 3.1.

Table 3.1 Wet Sieve Analysis of Reclaimed Aggregate

Sieve Size	Percentage Passing
25 mm	100
19 mm	93.15
13 mm	71.36
9.5 mm	59.63
4.75 mm	42.48
2.36 mm	33.83
1.18 mm	28.99
600 um	25.24
300 um	17.79
150 um	9.54
75 um	5.93

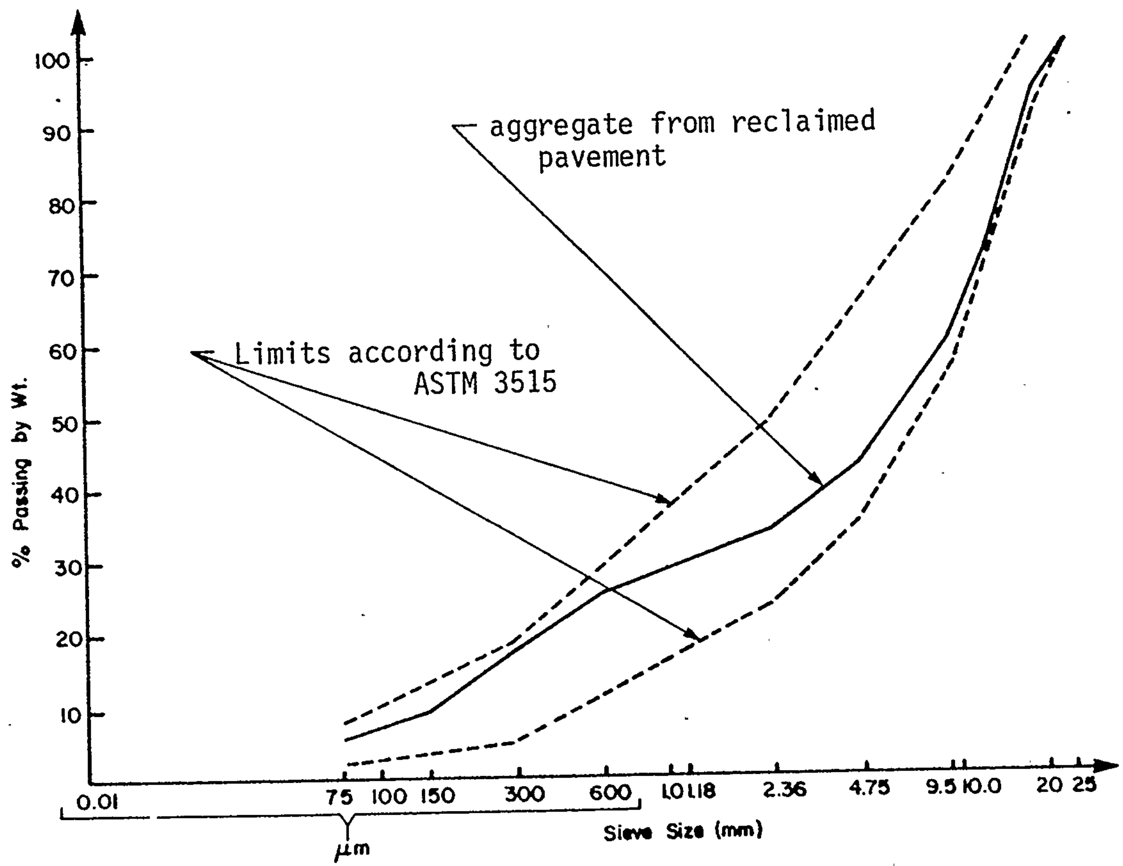


Figure 3.1 Aggregate Gradation of Reclaimed Aggregate

From the extraction process it was found that the old asphalt concrete has an asphalt content of 3.75% by weight of total mix. The reclaimed asphalt was tested for penetration at 25°C and viscosity at 60°C. These results are presented in Table 3.2.

Table 3.2 Properties of the Reclaimed Asphalt

penetration at 25°C	32 (in 0.10 mm units)
viscosity at 60°C	1731 Pa.s

3.3 Design of the Recycled Asphalt Concrete Mix

For the purpose of mixing with reclaimed aggregate, a virgin aggregate mix (virgin aggregate mix R) was designed, with a gradation similar to what is given in Table 3.3. Virgin aggregates from three sources were mixed for this purpose. The mixture consisted of 40% of 19 mm maximum size aggregate, 30% of 9.5 mm maximum size aggregate and, 30% of 2.5 mm maximum size aggregate by weight.

The reclaimed aggregate and virgin aggregate were then mixed in the ratio of 48:52 by weight.

This blend resulted in an aggregate grading which lies well within the ASTM D3515 gradation limits, and is presented in Table 3.4 and figure 3.2

Table 3.3 Aggregate Gradation of the Virgin Aggregate Mix R

Sieve Size	Percentage Passing
25 mm	100
19 mm	100
9.5 mm	62.52
4.75 mm	50.47
2.36 mm	41.49
1.18 mm	35.18
600 μm	28.7
300 μm	15.62
75 μm	5.71

Table 3.4 Aggregate Gradation of the Recycled Mix

Sieve Size	Percentage Passing
25 mm	100
19 mm	96.71
9.5 mm	61.13
4.75 mm	46.64
2.36 mm	37.81
1.18 mm	32.21
600 μm	27.04
300 μm	16.66
75 μm	5.81

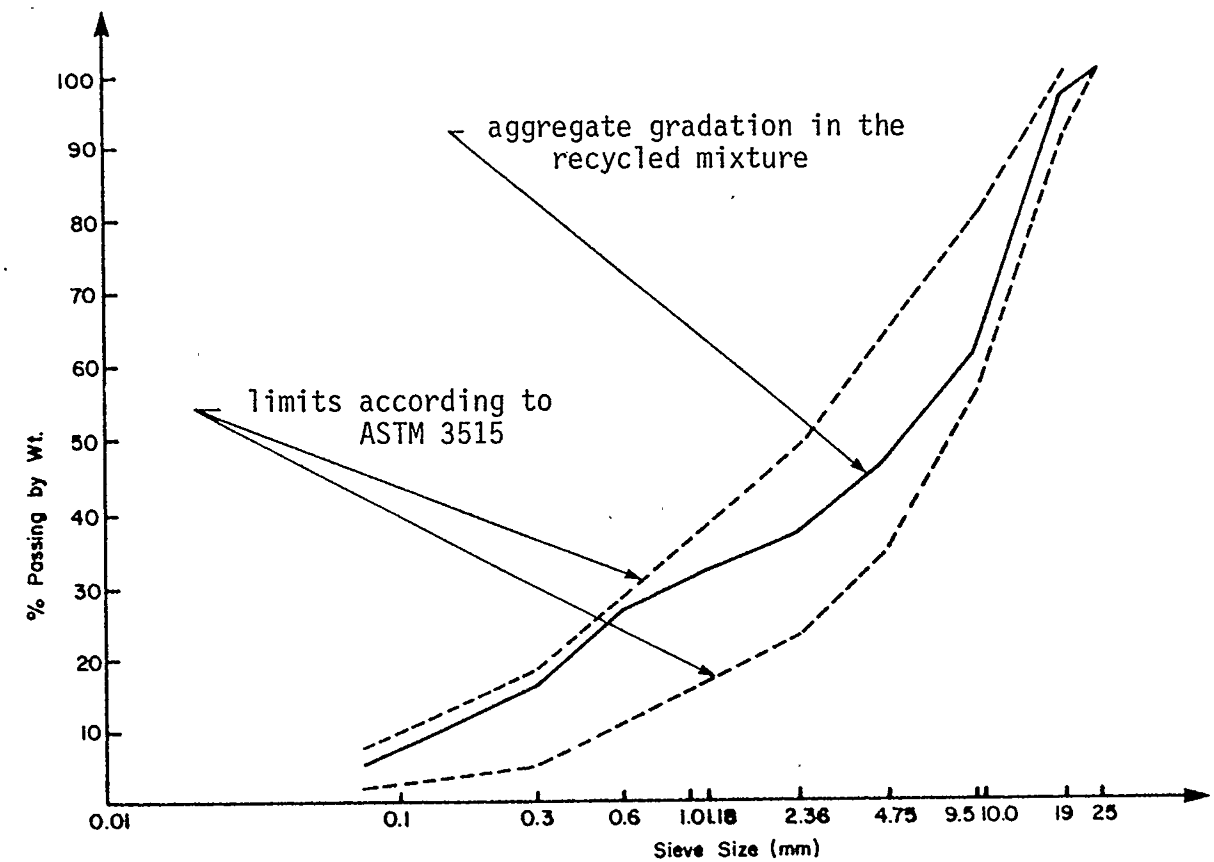


Figure 3.2 Aggregate gradation in the Recycled Mixture

The asphalt demand for the recycled asphalt mix was then calculated using eq. (2.1) as follows:

Asphalt demand (by weight of total mix) = P_c

$P_c = 0.035 \times 62.19 + 0.045 \times 32.0 + 0.18 \times 5.81 + 1.0$

$= 5.66\%$

The next step was to calculate the percentage of new asphalt in the mix by using eq. (2.2):

Percentage of new asphalt in the recycled mix = P_r

$P_r = 5.66 - (3.75 \times 0.48) = 3.86\%$

Percentage of new asphalt based on weight of binder = $\frac{3.86}{5.66} = 68.2\%$

This means that in the mix, new asphalt will be mixed with aged asphalt in the ratio of approximately 68:32. Then, the asphalt blending chart was used to determine the viscosity of soft asphalt that has to be blended with aged asphalt in the above ratio in order to obtain an asphalt mix having a viscosity of 95.0 Pa.S at 60°C. This is the viscosity of the 100-150 Pen asphalt that was used in the virgin asphalt concrete mix (This asphalt was selected for the virgin mix as it is widely used by the City of Calgary in making asphalt concrete mixtures for full depth asphalt concrete pavements). The blending chart with the relevant graphical constructions for the above purpose is shown in figure 3.3. From the blending chart it can be seen that a soft asphalt having a viscosity of 25.0 Pa.S at 60°C is required for blending.

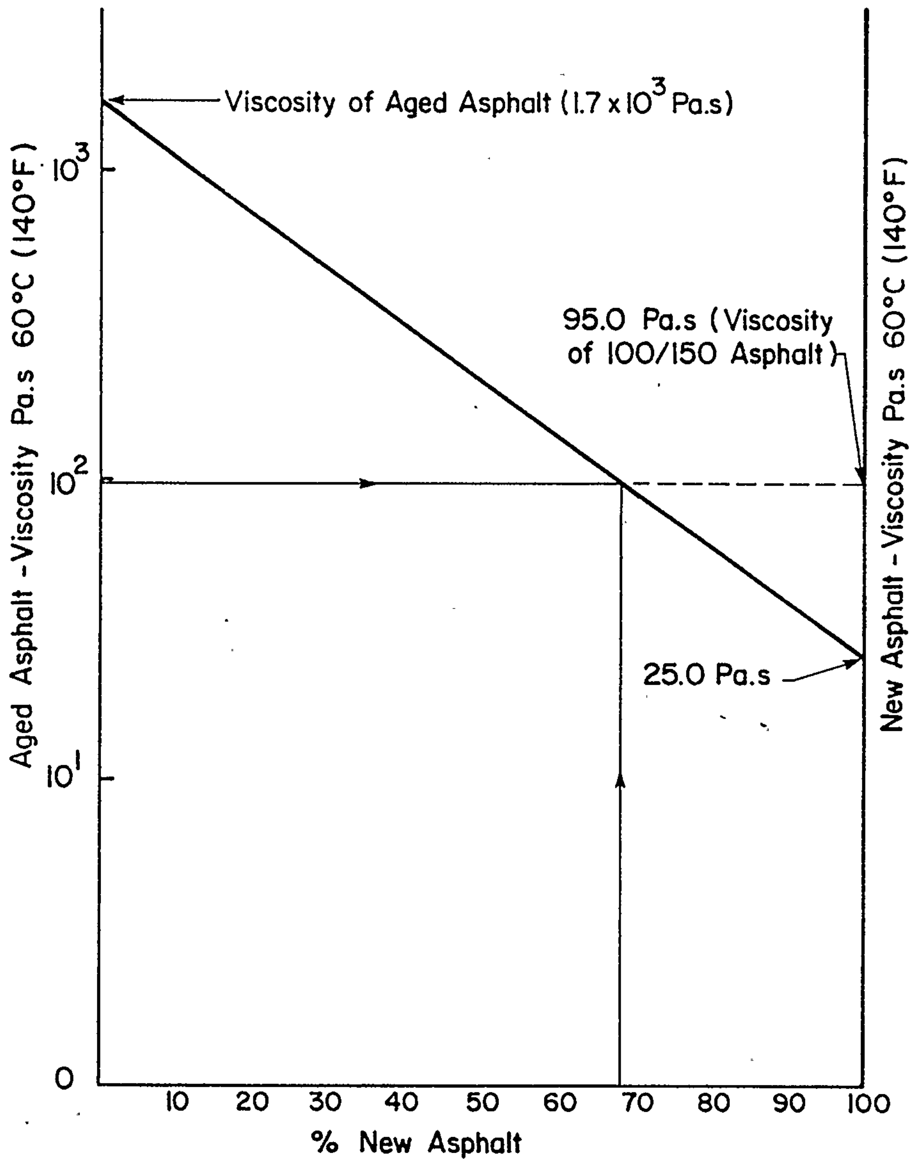


Figure 3.3 Asphalt Blending Chart

Table 3.5 The properties of Pen 300-400 Asphalt

Property	Value
Pen @ 25°C, 100g/5 sec	371
Flash point, °C	248
Viscosity @ 60°C, Pa.S	27.6
Kinematic viscosity @ 135°C, mm^2/S	160
Thin film oven test % loss	0.70
Viscosity ratio	2.83

However from the asphalts available in Alberta, the product that has a viscosity nearest to the above value is the 300-400 Pen asphalt produced by Husky Oil Ltd. It has a viscosity of 27.6 Pa.S at 60°C. The properties of this soft asphalt are summerized in Table 3.5. Hence, three trial mixes of asphalt were made by blending virgin soft asphalt and extracted aged asphalt in the ratios of 64:36, 66:34, 68:32. The absolute viscosities at 60°C of these trial asphalt blends are plotted against the percentage of new asphalt in these blends as shown in Figure 3.4. From this figure, it can be seen that when the percentage of virgin asphalt in the asphalt blend is around 67%, the absolute viscosity of the blend is about 95 Pa.S, which is the viscoity of the virgin asphalt that is used in the virgin mix.

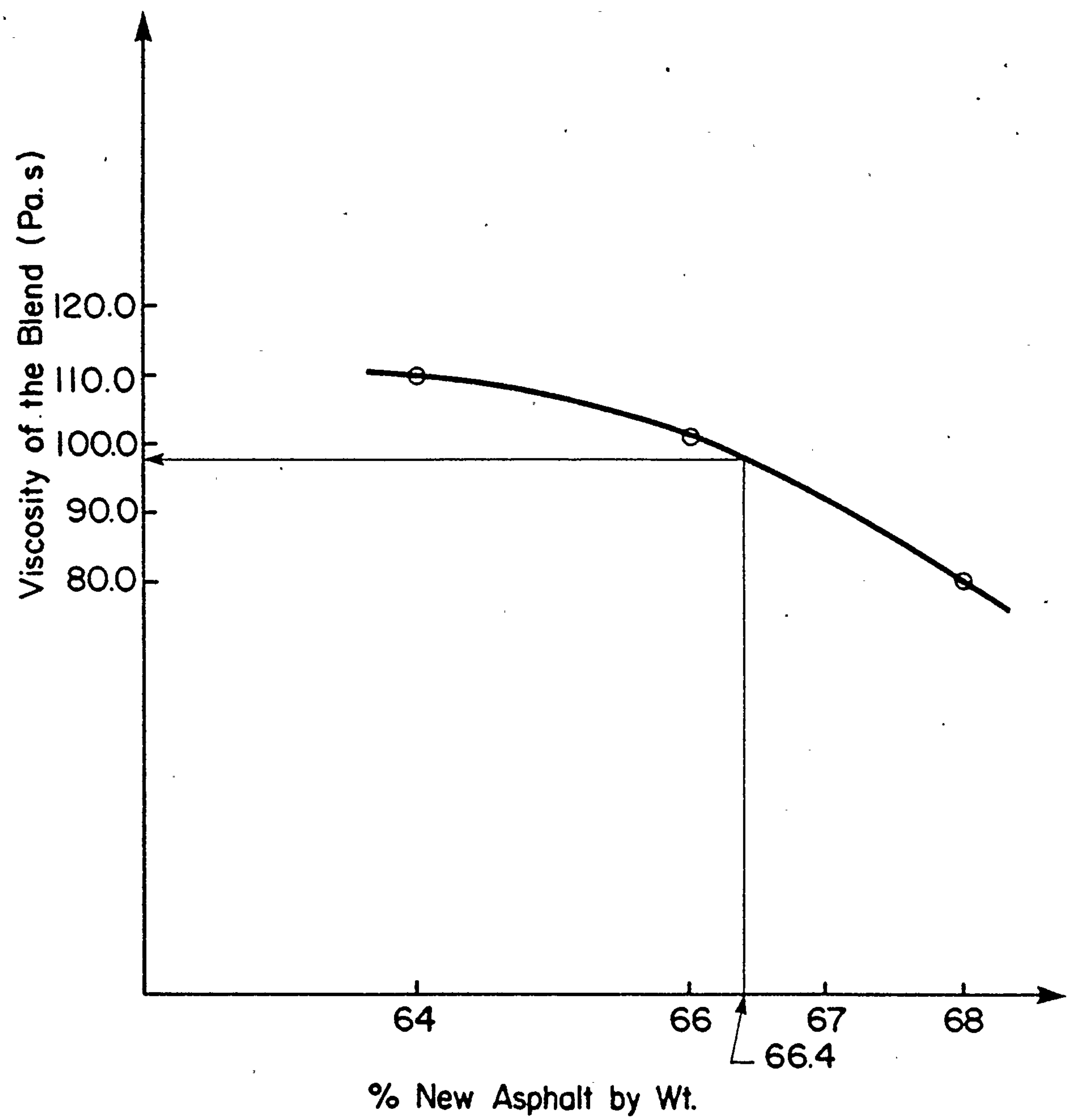


Figure 3.4 Viscosity of Trial Asphalt Blends

The Marshall test is then applied on recycled mixes made with asphalt contents of 5.16%, 5.41% and 5.66% respectively. These mixes contained 48% RAP and 52% virgin material (virgin material includes soft virgin asphalt and virgin aggregate mix R).

Marshall test results are summarized in Table 3.6 and Figure 3.5.

Table 3.6 Marshall Test Results on the Recycled Mix

Mix	A/C (%)	Stability (kN)	Flow (0.25 mm)	Unit Wt (kg/m3)	Bulk Specific Gravity	Air Voids (%)	V.M.A. (%)
R-1	5.66	7.63	18.33	2405	2.405	2.126	14.28
R-2	5.41	10.28	16.30	2385	2.385	3.085	14.58
R-3	5.16	6.51	13.33	2394	2.394	3.58	14.36

From these results the optimum asphalt content in the recycled asphalt concrete mix can be calculated as follows:

Asphalt content at maximum stability = 5.43%

Asphalt content at maximum unit weight = 5.66%

Asphalt content at 5.5% voids = 4.7 %

Optimum asphalt content = 5.3 %

At this asphalt content the amount of new binder (the soft virgin asphalt) in the recycled mix is 66%. From Figure 3.4 the resultant recycled asphalt mix viscosity would be 101 Pa.S, which is greater than the viscosity of asphalt in the virgin mix (95 Pa.S). Hence,

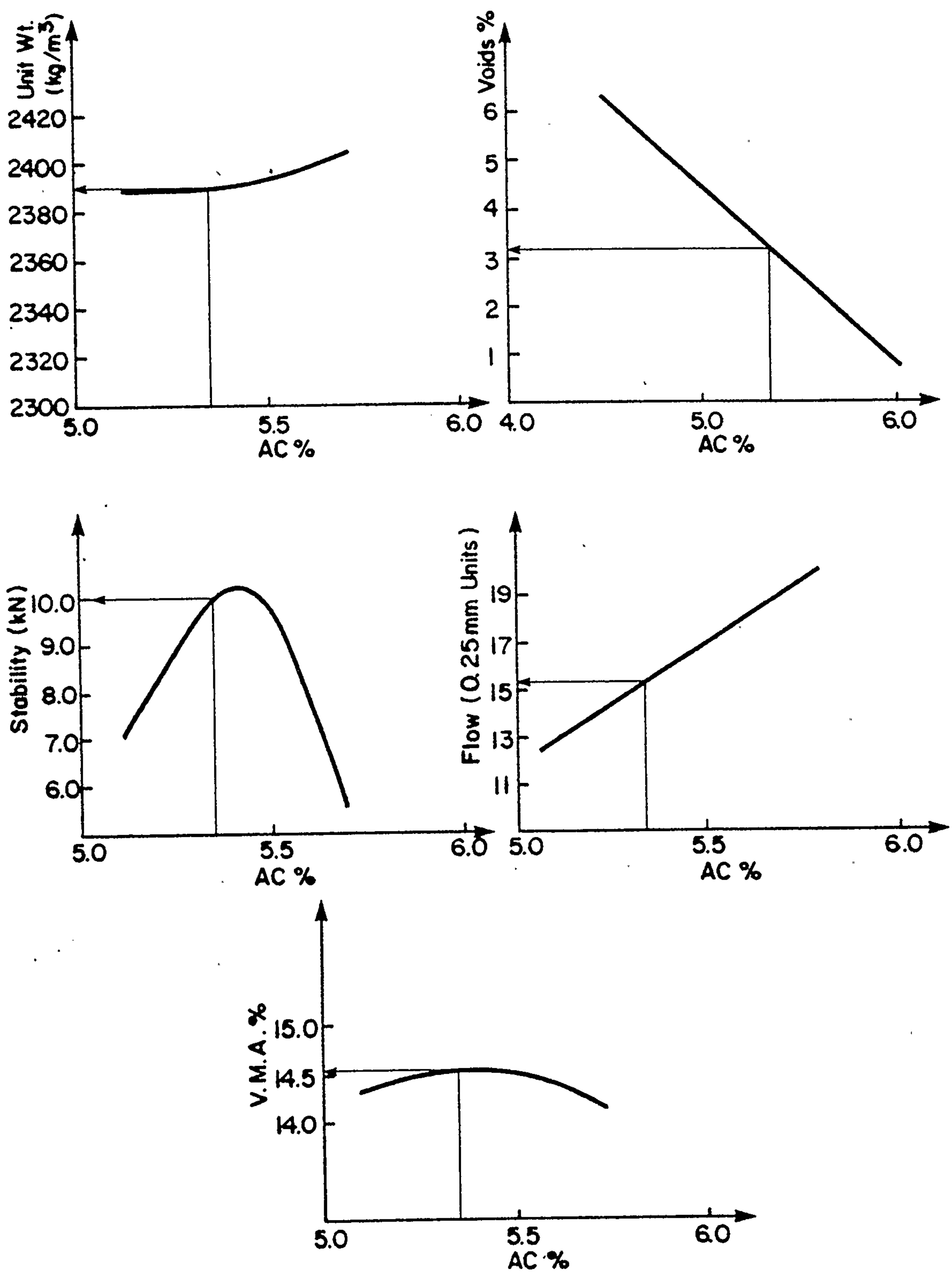


Figure 3.5 Marshall Test Results for Recycled Asphalt Concrete

in order to make viscosities of asphalts in the recycled mix and virgin mix more comparable, it was decided to use an asphalt content of 5.35% in the recycled mix. This results in a new binder content of 66.4%. Again, from Figure 3.4 it can be seen that the viscosity of the recycled asphalt blend is 98.0 Pa.S at 66.4% new binder content. This is quite close to 95.0 Pa.S, which is the viscosity of asphalt in the virgin asphalt concrete mix. At 5.35% asphalt content the recycled asphalt concrete mix has a stability of 9.9 KN, a flow of 15.4, voids percentage of 3.2% and a V.M.A. (Voids in Mineral Aggregate) of 14.55%.

3.4 Design of the Virgin Asphalt Concrete Mix

Aggregate mix (virgin aggregate mix V) for the virgin asphalt concrete was designed by using aggregates from four sources. It consisted of a mixture of 5% of 25 mm maximum size aggregate, 37.5% of 19 mm maximum size aggregate, 27.5% of 9.5 mm maximum size aggregate and 30% of 2.5 mm maximum size aggregate by weight. The aggregate grading of the virgin aggregate mix V is presented in Table 3.7 and Figure 3.6.

Table 3.7 Aggregate Gradation of the Virgin Aggregate Mix V

Sieve Size	Percentage Passing
2.5 mm	100
19 mm	98.48
9.5 mm	60.96
4.75 mm	49.01
2.36 mm	41.08
1.18 mm	34.28
600 um	28.11
300 um	14.99
150 um	7.85
70 um	5.34

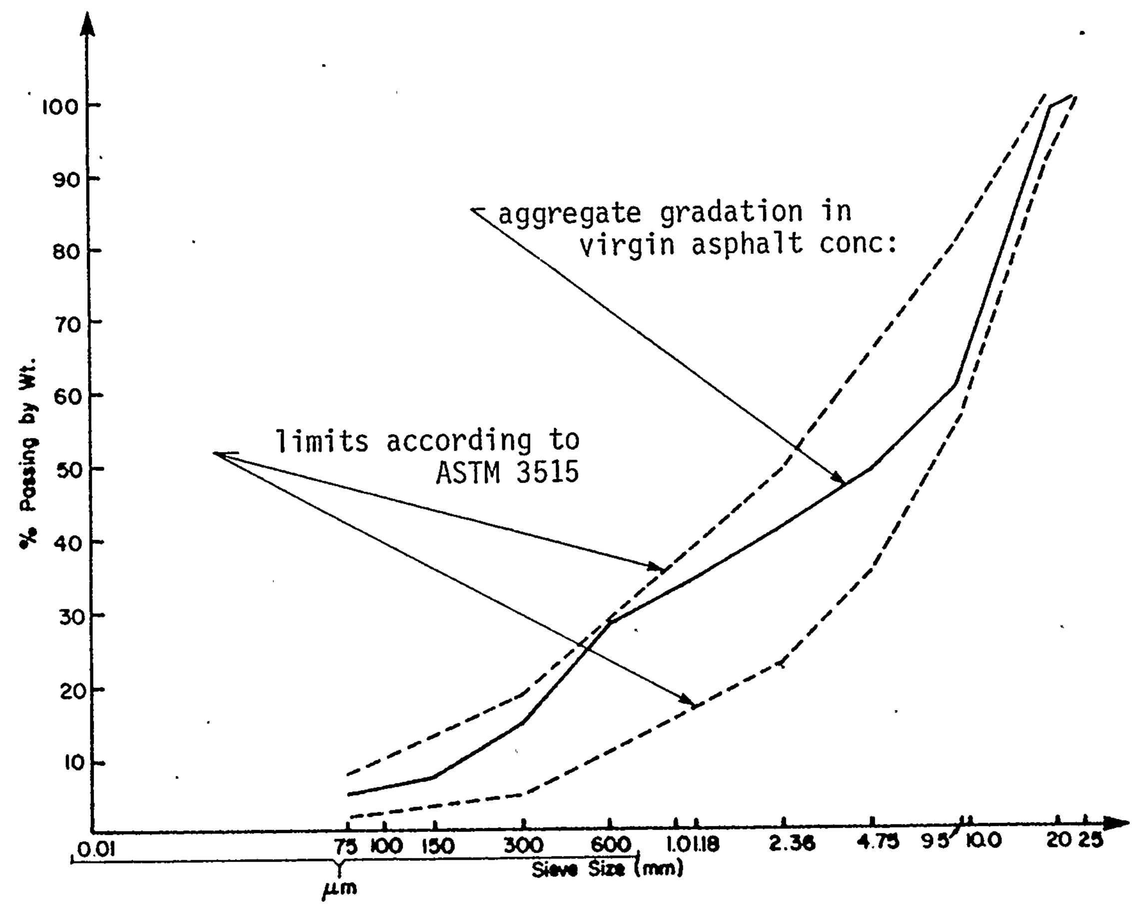


Figure 3.6 Aggregate Gradation in the Virgin Asphalt Concrete

The virgin 100-150 Pen asphalt was tested for penetration (at 25°C), absolute viscosity (at 60°C) and kinematic viscosity (at 135°C).

The results are presented in Table 3.8.

Table 3.8 Properties of 100-150 Pen Virgin Asphalt

Property	Value
Penetration (at 25°C)	141 (0.10 mm units)
Kinematic Viscosity (at 135°C)	277.33 mm^2/S
Absolute Viscosity (at 60°C)	95 Pa.S

The asphalt demand was then calculated for the virgin asphalt concrete based on the aggregate gradation of the virgin aggregate mix V from equation (2.1).

Percentage of asphalt in the mix Pc, (by weight of total mix)

$$Pc = 0.035 \times 58.92 + 0.045 \times 35.74 + 0.18 \times 5.34 + 1.0 = 5.63\%$$

The Marshall test was then performed on virgin asphalt concrete mixes having asphalt contents of 6.13%, 5.63%, 5.13% and 4.63%. The results of the above tests are summarized in Table 3.9 and Figure 3.7.

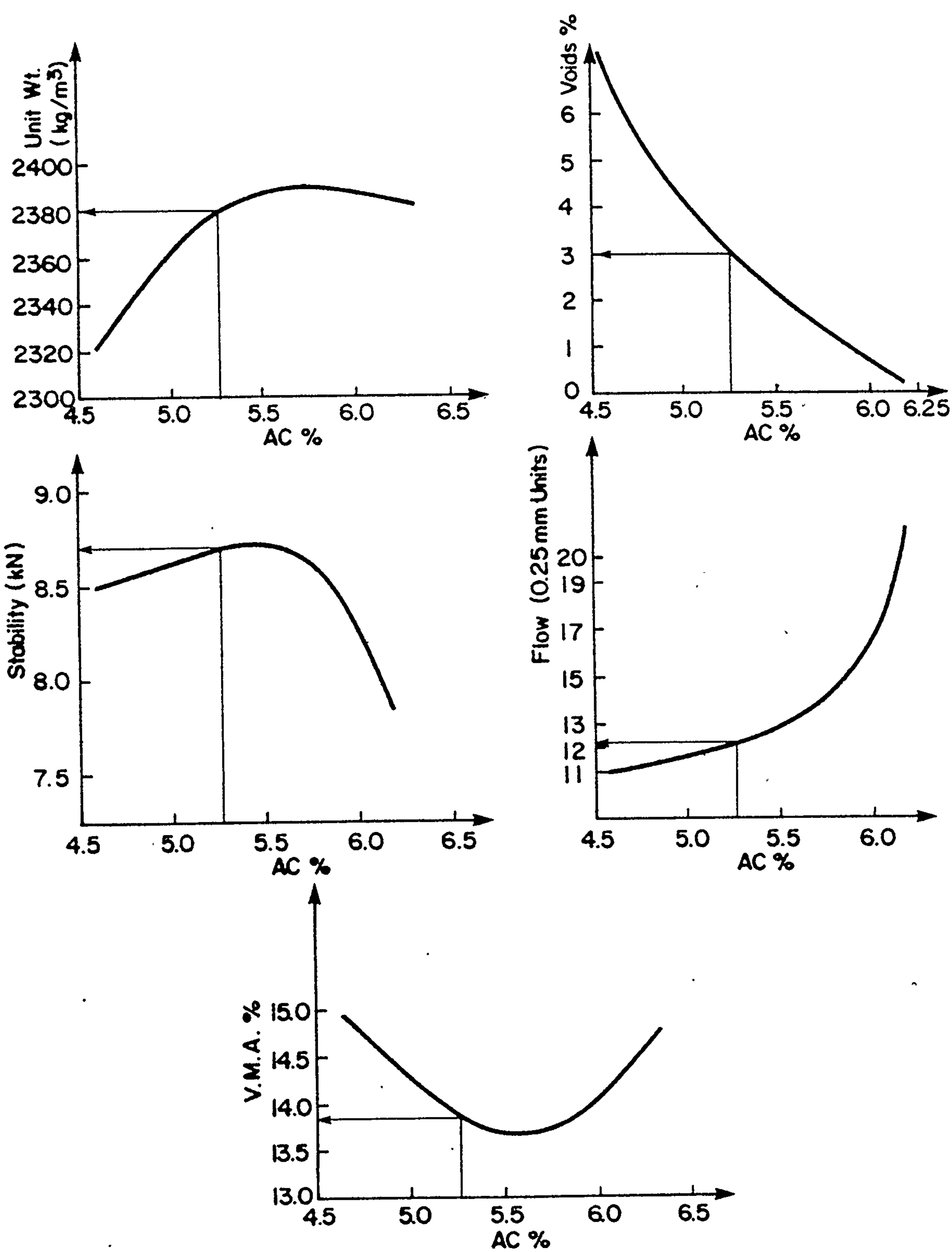


Figure 3.7 Marshall Test Results for Virgin Asphalt Concrete

Table 3.9 Marshall Test Results on the Virgin Mix

Mix	A/C (%)	Stability (kN)	Flow (0.25 mm	Unit Wt (kg/m3)	Bulk Specific Gravity	Air Voids (%)	V.M.A. (%)
V-1	6.13	7.98	19.75	2386	2.386	0.433	14.25
V-2	5.63	8.69	13.16	2389	2.389	1.824	13.68
V-3	5.13	8.54	12.00	2359	2.359	3.459	14.32
V-4	4.63	8.60	11.00	2333	2.333	6.53	14.82

From these results, the optimum asphalt content for the virgin asphalt concrete can be calculated as follows.

Asphalt content at maximum stability = 5.4%

Asphalt content at maximum unit weight = 5.65%

Asphalt content at 5.5% air voids = 4.75%

Optimum asphalt content = 5.30%

At this asphalt content the asphalt concrete mix has a stability of 8.7 kN, a flow of 12.2, air voids percentage of 3.0% and V.M.A. of 13.85%.

Finally, the material properties of recycled and virgin mixtures are summarized in Table 3.10.

Table 3.10 Summary of the Material Properties of Recycled and Virgin Mixtures

		Recycled Mixture	Virgin Mixture
Properties of the Asphalt Cement			
Penetration (in 0.10 mm units) at 25°C		153	141
Absolute Viscosity (Pa.S) at 60°C		98.0	95.0
Sieve Analysis of the Aggregate			
Sieve Size	ASTM Specification Limits		
25 mm	100	100	100
19 mm	90 - 100	96.71	98.48
9.5 mm	56 - 80	61.13	60.96
4.75 mm	35 - 65	46.64	49.01
2.36 mm	23 - 49	37.81	41.08
1.18 mm	-	32.21	34.28
600 µm	-	27.04	28.11
300 µm	5 - 19	16.66	14.99
75 µm	2 - 8	5.81	5.34
Marshall Test Results			
Optimum Asphalt Content (%)		5.35	5.3
Air Voids at Optimum A/C (%)		3.2	3.0
Stability at Optimum A/C (%)		9.9	8.7
Flow at Optimum A/C (in 0.25 mm units)		15.4	12.2
V.M.A. at Optimum A/C (%)		14.55	13.85

CHAPTER 4

REPEATED LOAD TRIAXIAL TEST AND THE TESTING PROGRAM

4.1 Repeated Load Tri-axial Test

The repeated load tri-axial test is the best technique available to obtain the relationship between permanent deformation due to repeated loading, number of load repetitions and the applied stresses for asphalt concrete, soils and other pavement materials. It allows the determination of elastic strains, permanent strains, the resilient modulus and Poisson's ratio of an asphalt concrete sample subjected to repeated loading.

4.1.1 Stress Conditions Applicable in a Tri-axial Test

In the tri-axial test, only principal stresses can be applied on the sample. Also, because of the axial symmetry of the loading arrangement, two of the above principal stresses are equal. A further restriction is that tension can be applied on the sample only in one direction (i.e. only in the vertical direction).

Due to the existence of shear stresses and principal stresses which are different from each other, the stress conditions at locations away from the loading axis in a pavement are quite different from the stress conditions that can be produced in a tri-axial test. But if an element on the axis of load is considered, the stress condition is very similar to that in the tri-axial cell. This is due to two reasons. Firstly, the principal stresses act in vertical and horizontal directions at any point on the axis of the load. Secondly, lateral principal stresses are equal. Hence, the tri-axial test can be used to exactly reproduce the stress conditions at points directly under the loading point in a pavement. This is illustrated in Figure 4.1.

However, when stress conditions of points near the bottom of the pavement are considered, a discrepancy occurs. This is due to the fact that both lateral principal stresses are tensile stresses. Since in the tri-axial test tension can be applied only in one direction, an approach which makes use of the stress invariant concept has to be applied to simulate these stress conditions in a tri-axial test (Brown, 1975).

Stress invariants are functions of the principal stresses but are independent of their directions. In a three-dimensional stress system, mean normal stress and octahedral shear stress are found to be stress invariants. Where the principal stresses are σ_1, σ_2 and σ_3, they are defined as follows.

Mean normal stress = $p = 1/3(\sigma_1 + \sigma_2 + \sigma_3)$ (4.1)

Octahedral shear stress = τ_{oct}

$$\tau_{oct} = 1/3\left((\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2\right)^{1/2} \quad (4.2)$$

When the in-situ stress condition beneath the centre of a single wheel load is considered, then:

$\sigma_1 = \sigma_z$ = vertical stress

and

$\sigma_2 = \sigma_3 = \sigma_r$ = horizontal stress

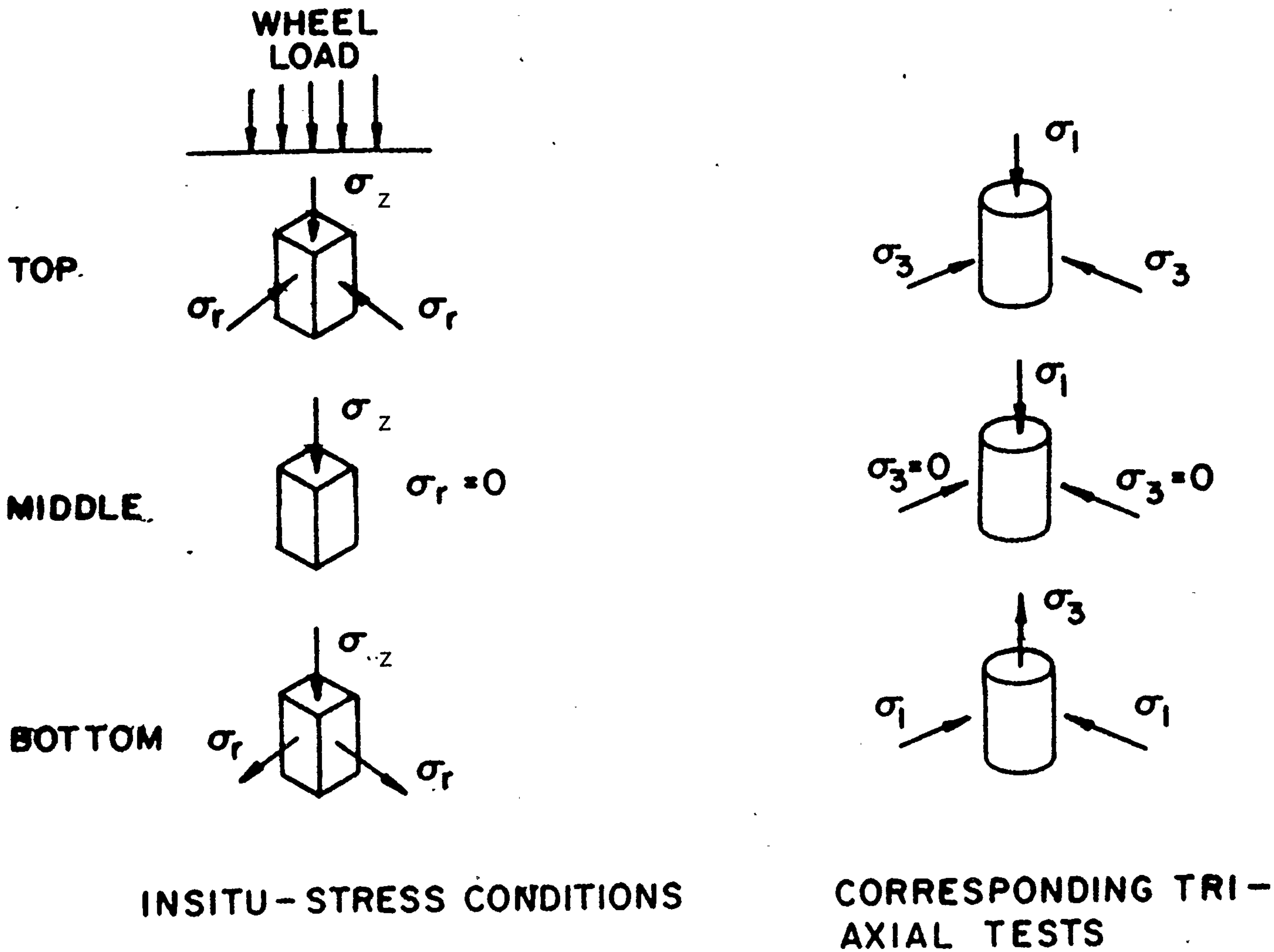


Figure 4.1 Use of Tri-axial Tests to Simulate the In-situ Stress Conditions

Hence the normal and shear stress invariants are written as

$$p = 1/3\ (\sigma_z + 2\ \sigma_r) \tag{4.3}$$

$$q = (3/\sqrt{2})\tau_{oct} = (\sigma_z - \sigma_r) \tag{4.4}$$

In the tri-axial test, the vertical compressive stress is σ_1 and the confining stress is σ_3. However, in order to simulate in-situ stress conditions in the tension zone, tension has to be applied in the vertical direction in the tri-axial test. Hence, when compressive stresses are taken as positive the lateral stress becomes σ_1 and vertical stress becomes σ_3. Therefore, when the tri-axial test is used to simulate stress conditions in the tension zone of a pavement, p and q can be expressed as

$$p = 1/3\ (2\ \sigma_1 + \sigma_3) \tag{4.5}$$

$$q = (\sigma_1 - \sigma_3) \tag{4.6}$$

Hence, the loading conditions in the tri-axial test should be maintained such that p and q have the same values for the test and for the in-situ condition of the point under consideration. By planning the stress conditions of the tri-axial test program in this manner, in-situ stress conditions can be closely simulated.

The strains measured in the tri-axial test need to be transformed in the same manner, by using strain invariants in order to obtain the in-situ strains.

In the tri-axial test, volumetric and shear strains are defined as,

$$v = 2\varepsilon_1 + \varepsilon_3 \tag{4.7}$$

$$\gamma_{oct} = \{2\sqrt{2}/3\}(\varepsilon_1 - \varepsilon_3) \tag{4.8}$$

where ε_1 and ε_3 are the principal strains. For the insitu-conditions these strains are defined as

$$v = \varepsilon_z + 2\varepsilon_r \tag{4.9}$$

$$\gamma_{oct} = \{2\sqrt{2}/3\}(\varepsilon_z - \varepsilon_r) \tag{4.10}$$

where ε_z and ε_r are the vertical and lateral strains. Both ε_1 and ε_3 can be determined from the tri-axial test. Then, as strain invariants should be the same for the test and the in-situ condition, the vertical strain ε_z can be calculated from the equations (4.7) to (4.10). ε_z is given by the following expression.

$$\varepsilon_z = 4/3\,\varepsilon_1 - 1/3\,\varepsilon_3 \tag{4.11}$$

4.1.2 Stress Application and Measurement

The original tri-axial test which was developed for soil mechanics experiments is modified for use in the repeated load tests in pavement design. Figure 4.2 shows the modifications carried out on the original test equipment in order to apply the vertical and lateral stresses on the sample independently.

Vertical load in the test has to be applied in such a manner that it represents the stress condition in the pavement. Hence, repeated vertical load with a load pulse of sinosoidal or triangular shape is usually applied in the test since these forms are found to be the best for simulation of actual stress conditions (Barksdale, 1971). The lateral stresses are also

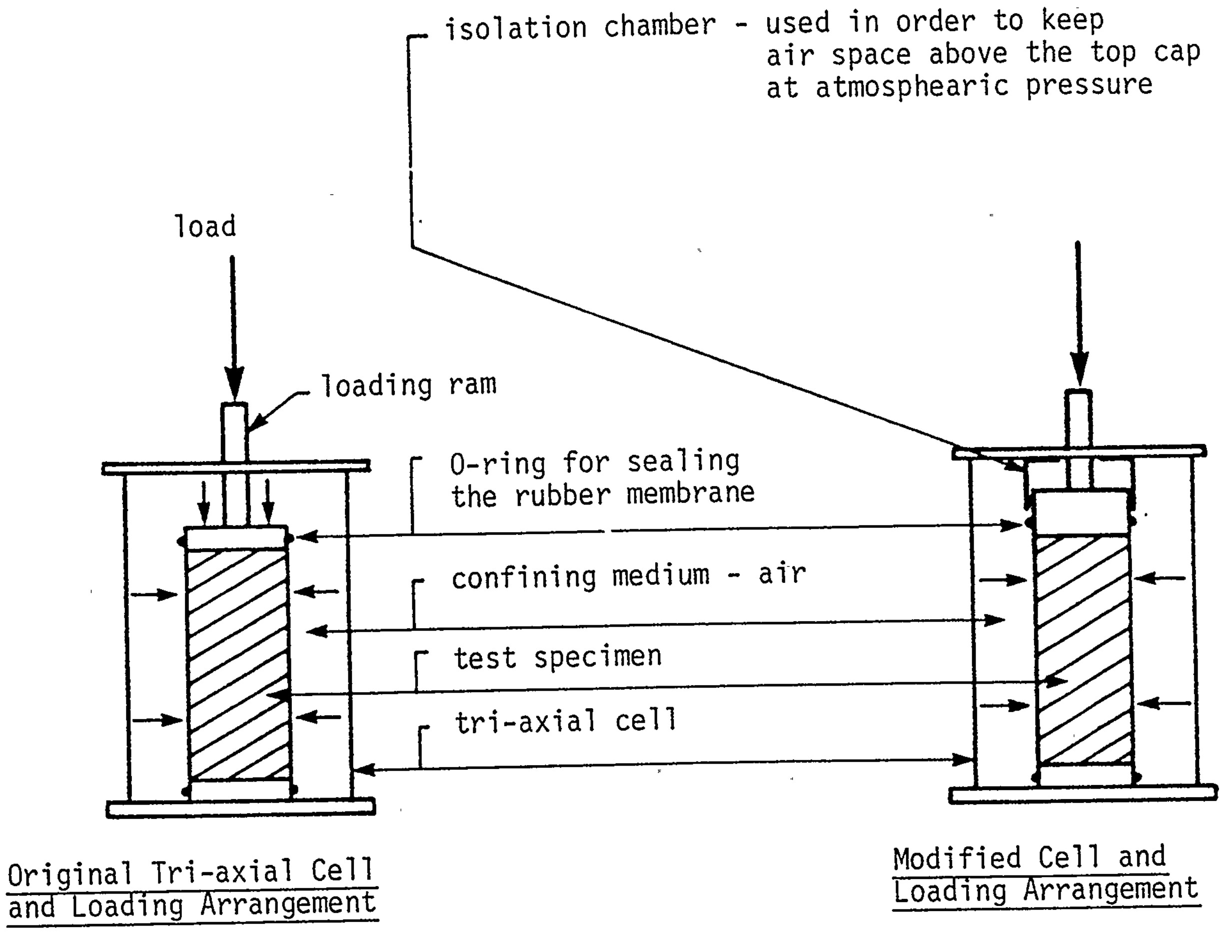


Figure 4.2 Modifications carried out on the Basic Equipment for Tri-axial Tests

applied using the same pulse shape, and in phase with the vertical stress. However, it has been found that the permanent deformation is not affected significantly by maintaining the lateral stress at an average value instead of cycling (Brown and Snaith, 1974). Hence, the tri-axial testing equipment is often simplified by maintaining a constant cell pressure.

Another factor that has to be considered in tri-axial testing is the use of rest periods between load pulses. In the in-situ situation, usually there are rest periods between loadings. But it has been found that rest periods are not significant for permanent strain tests (Brown, 1976). Hence, it is advantageous not to have rest periods between load cycles since this reduces the duration of a test. However, some researchers in North America have used rest periods in the order of two seconds (Morris et al, 1974; McLean and Monismith, 1975).

The stress pulse time that should be used can be determined from the data presented by Barksdale (Barksdale, 1971). The pulse time depends on the depth of the pavement and vehicle speed.

Three types of loading systems have been used in the past to apply stresses in the tri-axial test. Firstly, there are mechanical systems where repeated loading is applied by rotating cams. These systems can be used for either stress or strain-controlled testing. Secondly, there are pneumatic systems where air pressure is used for load application. In this arrangement air pressure is applied to an air ram through a solenoid valve which can be electronically controlled to give the correct load pulse. Pneumatic systems are best used for controlled load testing. Finally, there are servo-controlled electro-hydraulic systems which can provide either load, stress or deformation control. Several wave forms with various combinations of load pulse and rest periods are available from these systems. The major advantage in this system is its accurate load control.

Usually the measurement of the applied vertical load on the sample is done by using a load cell, which gives an electrical output proportional to the load.

When the load is measured outside the cell, an error always occurs due to the friction between the loading ram and the sleeve in the cell top. Hence, for accurate load measurement it is essential to use an internal load cell which is usually mounted on the top cap or in the pedestal on which the sample rests.

4.1.3 Strain Measurement in Tri-axial Tests

An error is often caused in strain measurement due to the friction between sample and end caps, as this prevents the samples from deforming as right cylinders. In the case of compressive tri-axial tests, this friction can be reduced by using a thin high vacuum grease layer between the sample and end caps. However, the friction cannot be eliminated in the case of tension tests as the sample has to be bonded to the platens.

The most accurate method of measuring the strains in the sample is the direct on-sample measuring technique. LVDT's or recti-linear potentiometers (R.L.P.'s) can be directly mounted on the sample using strain collars (TRB Special Report No-162, 1975). Although the TRB special report no - 162 recommends the use of two calipers for mounting R.L.P.'s on the sample, three calipers were used in this study. This is because the two caliper arrangement resulted in considerable inaccuracies in the measurement of lateral strains. The three caliper arrangement used in this investigation is illustrated in figures 4.3, 4.4 and photo 4.1. The calipers were mounted on the sample by gluing them on to the asphalt using epoxy resin.

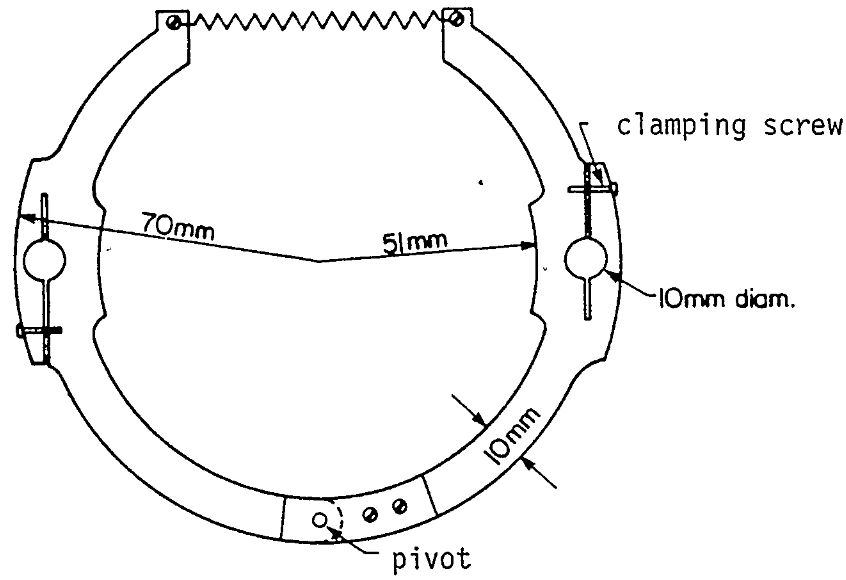
The temperature of the sample was controlled by maintaining a slow moving stream of air through the cell. The air temperature was controlled using a hot pack and a heat exchanger. As the hot-pack had a temperature range of -20°C to 80°C the sample and the tri-axial cell could be maintained at any temperature in the range from 0°C to 40°C. The layout of the temperature control system is illustrated in Figure 4.5.

The cell pressure in the tri-axial cell is also provided by the air supply.

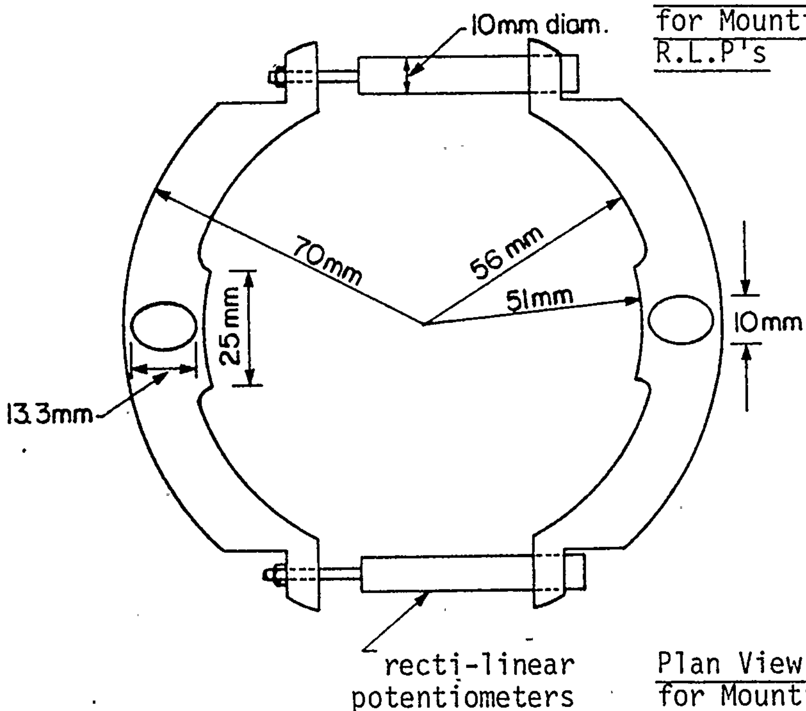
4.2 Selection of Stress Conditions for the Tri-axial Tests

4.2.1 Pavement Analysis

For the purpose of analysis, a pavement is considered as a multi-layer elastic system. Since permanent deformation



Plan View of the Caliper for Mounting Vertical R.L.P's



Plan View of the Caliper for Mounting Lateral R.L.P's

Figure 4.3 Calipers Used for Mounting R.L.P's on the Test Sample

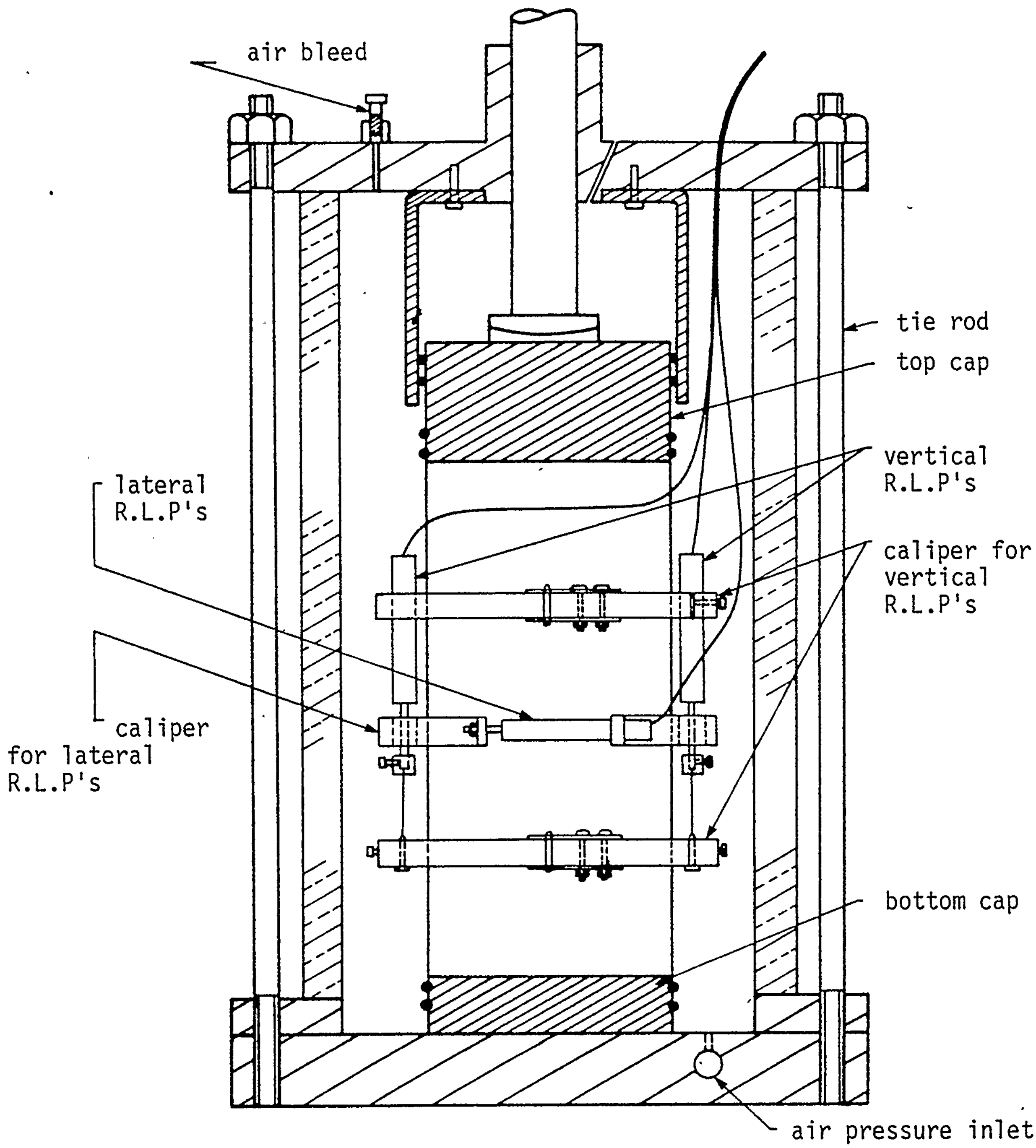


Figure 4.4 Apparatus for Measuring Permanent Deformation in Asphalt Concrete Samples

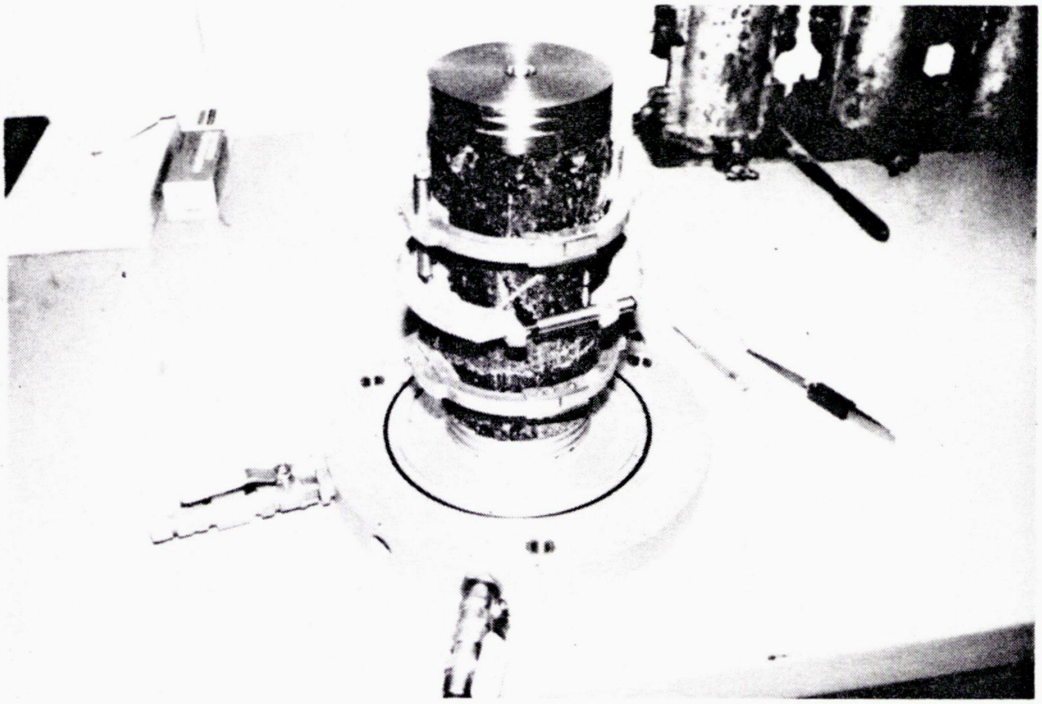


Photo 4.1 R.L.P.'s Mounted on the Test Sample Using Calipers

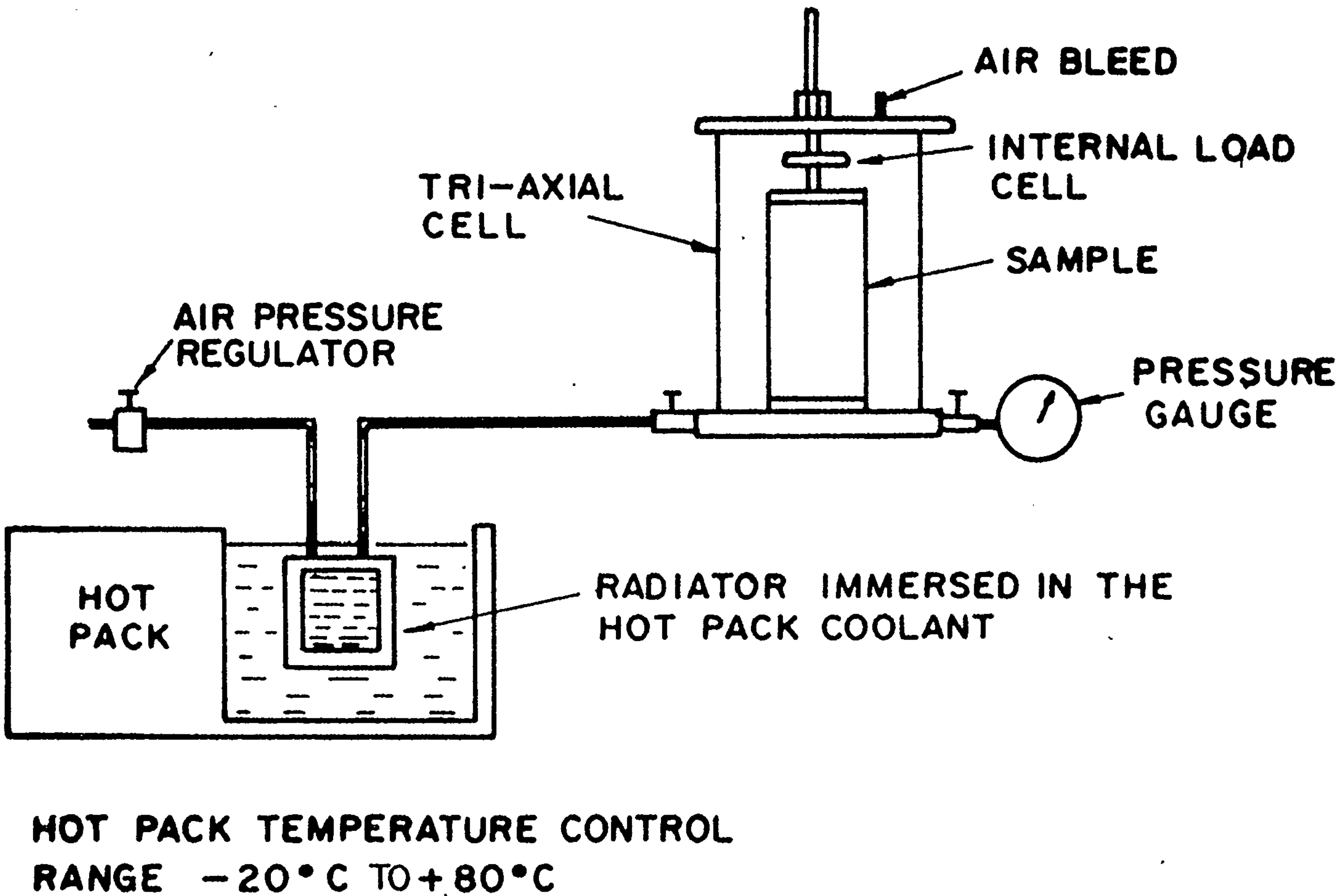


Figure 4.5 Temperature Control System for Tri-axial Test Specimens

prediction of full-depth asphalt concrete pavements is the purpose of this study, only a two layer pavement system has to be considered. The first layer, which has a finite thickness, represents the asphalt concrete layer while the second layer with infinite depth represents the subgrade. Full continuity is assumed between the two layers. It is also assumed that the asphalt layer extends to infinity in the horizontal directions. Each layer is characterized by a modulus of elasticity and a Poisson's ratio. Traffic is expressed as the number of repetitions of the 80 KN (18000 lb) single axle load on two sets of dual tires. The dual tires are spaced at 345 mm (13.57 inches) from centre to centre and each of them is approximated by a circular plate with radius 115 mm (4.52 inches). The tire pressure is equal to 483 kPa (70 psi).

This pavement system was analyzed using the Chevron5L multi-layer system elastic analysis program. Three systems with different depths of pavement were analyzed. Values used for the system parameters are presented in Table 4.1.

Table 4.1 Parameters of Pavement Systems used in the Analysis

Depth of Asphalt Concrete Layer	Asphalt		Subgrade	
	Resilient Modulus (MPa)	Poisson's Ratio	Resilient Modulus (MPa)	Poisson's Ratio
150 mm (6 inches) 200 mm (8 inches) 250 mm (10 inches)	2320	0.45	232	0.4

In each pavement, stresses were determined at ten locations uniformly spaced through out the depth of the asphalt concrete layer. All the locations were on the load axis.

These stresses were then plotted in a p,q diagram as shown in Figure 4.6.

4.2.2 Selection of Stress Levels

The purpose of conducting tri-axial tests is to determine the relationship between permanent strain due to repeated loading, the number of load repetitions and the applied stress. Hence, in selecting the stress levels for application in tri-axial tests,

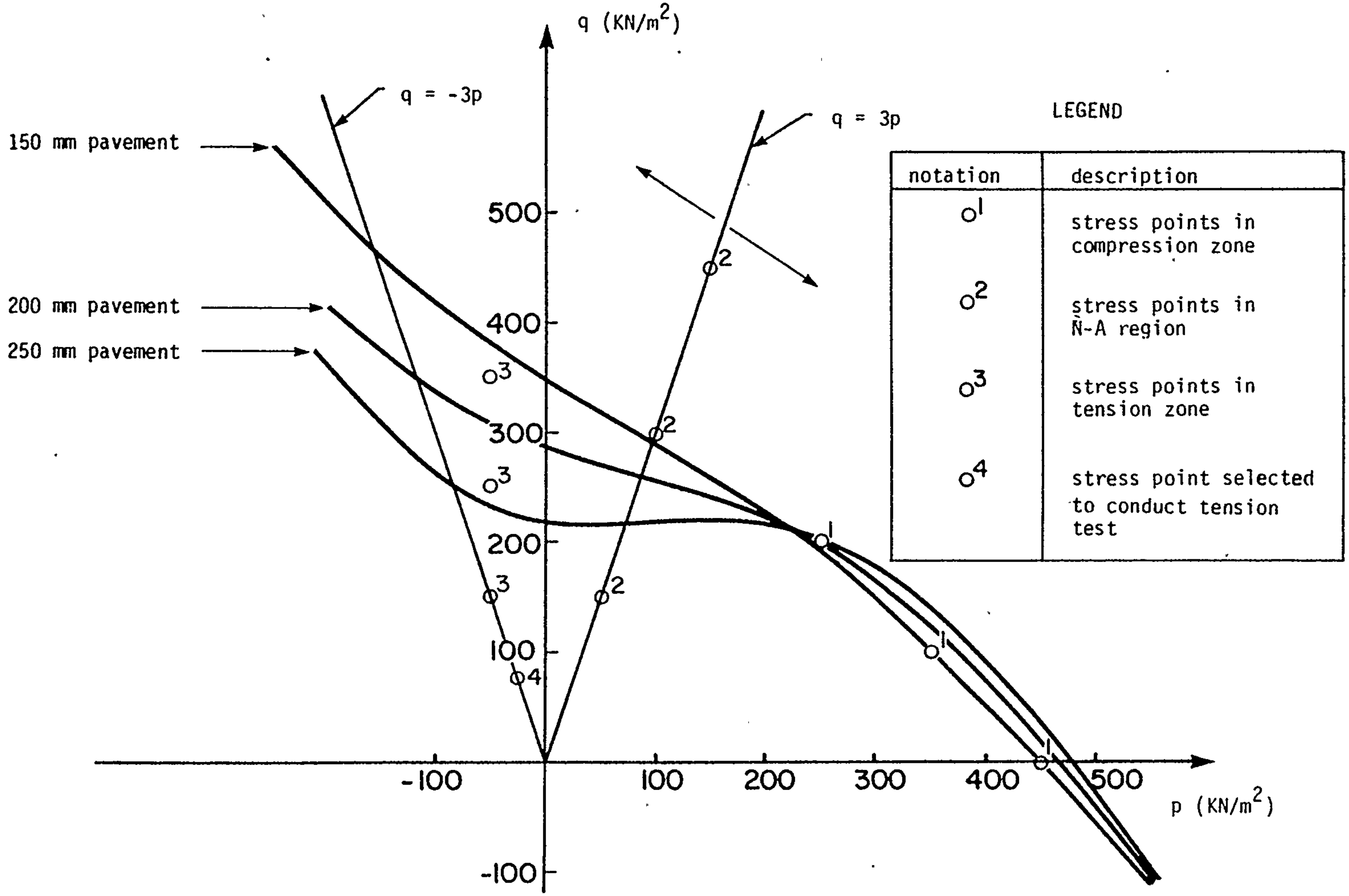


Figure 4.6 The p - q Stress Diagram

significantly different stress values should be chosen in order to obtain reasonably different magnitudes of permanent deformation. Also an effort should be made to choose stress values that represent stress conditions existing in actual pavements.

Considering the above factors, nine stress points were chosen for the tests. Three stress points were selected to represent the compression zones. The region of the neutral axis was also represented by another three stress points. Three stress points were also selected to represent the conditions in the tension zone. However, later it was found that performing tension tests at the selected stress levels were impractical due to the very short duration of tests.

Hence, one low stress level was selected for the purpose of conducting tension tests.

Therefore, only seven stress levels were used in the testing program. These stress levels are summarized in Table 4.2.

Table 4.2 Stress Levels Used in the Testing Program

Test Type	Test Identification Number	Vertical Stress (KN/m^2)	Cell Pressure (KN/m^2)	p (KN/m^2)	q (KN/m^2)
Unconfined Tri-axial Test	UC 1	450	00.0	150	450
	UC 2	300	00.0	100	300
	UC 3	150	00.0	50	150
Confined Tri-axial Test	CC 1	450	224.0	450	0
	CC 2	416.5	158.6	350	100
	CC 3	383.35	93.0	250	200
Tension Test	T 1	75	00.0	-25.0	75.0

4.3 Types of Tri-axial Tests

In order to create the selected stress conditions (given in Table 4.2) in the tri-axial test sample, three types of tests were used.

4.3.1 Confined Compression Test

In this test both vertical and lateral compressive stresses are applied on the sample in order to simulate the stress condition in the compression zone of the pavement. Vertical load was cycled in a sinosoidal manner, where as cell pressure was maintained constant at half of the chosen peak value of the lateral stress. The sample was enclosed in a rubber membrane. Also a modification was used on the tri-axial cell in order to apply vertical and lateral loads independently. An allowance was made for friction between the loading ram and the sleeve in the cell cover in determining the vertical load that was to be applied to the sample. The vertical load was applied using the MTS testing machine while lateral load was applied using air pressure. The equipment arrangement for this test is shown in photo 4.2.

4.3.2 Unconfined Compression Test

In this test only vertical load (cyclic) was applied. No cell pressure (hence no lateral stress in the sample) was applied, as this test is used to simulate stress conditions in the neutral axis region of the pavement. An internal load

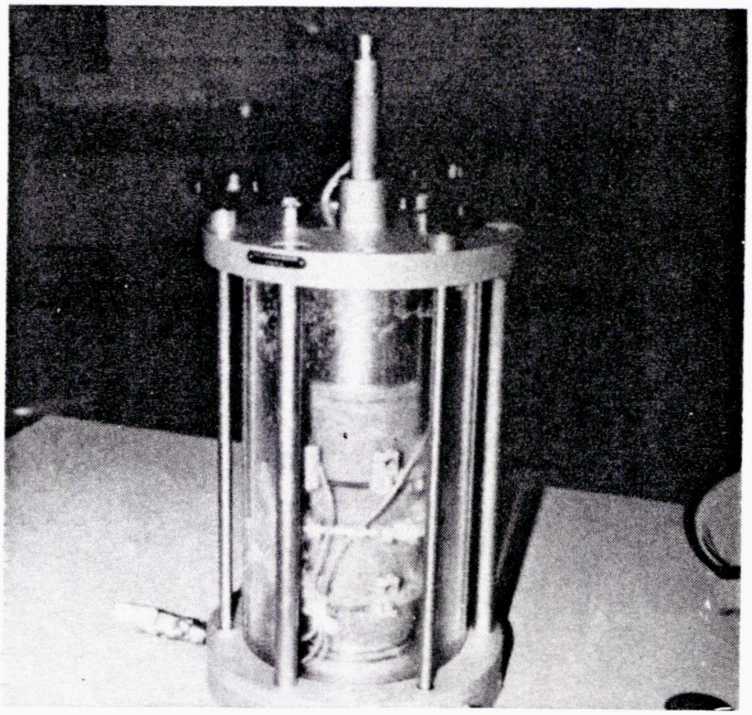


Photo 4.2 Tri-axial Cell Prepared for Conducting a Confined Compression Test

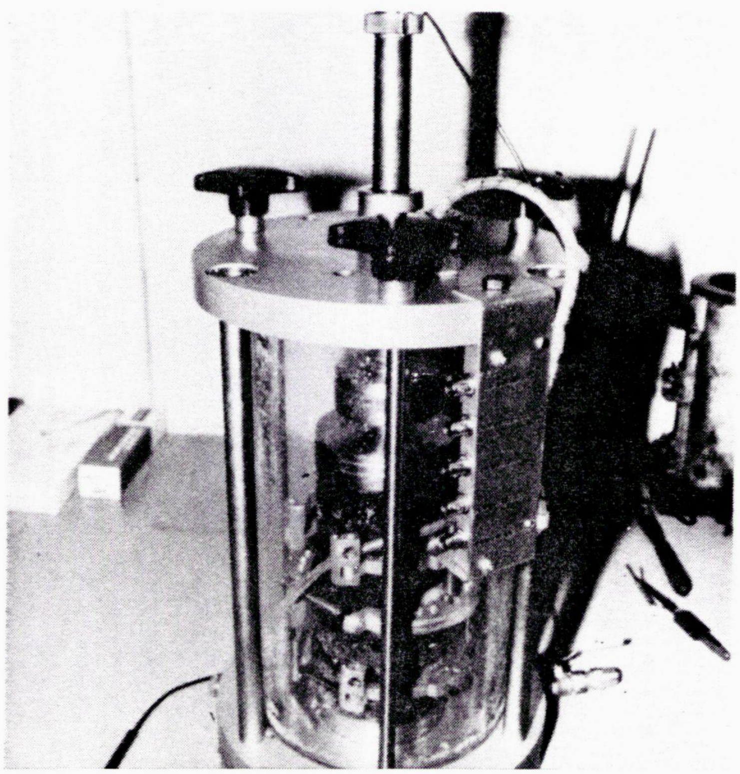


Photo 4.3 Tri-axial Cell Prepared for Conducting an Unconfined Compression Test

cell was used to accurately measure the vertical load. The test set up is illustrated in photo 4.3.

4.3.3 Tension Test

In this test, tension load (cyclic) was applied in the vertical direction. At the stress condition chosen for the test, q was equal to -3p as shown in Figure 4.6. Since the lateral stress σ_1 for the tension test is given by the expression

$$\sigma_1 = (3p + q)/3 \tag{4.12}$$

Then, $\sigma_1 = 0.00$

Therefore, no cell pressure was applied. The equipment arrangement for the tension test is shown in photo 4.4.

For each stress level the test was repeated three times in order to obtain reliable results.

All the tests were conducted at 20°C. Both recycled and virgin asphalt concrete samples were tested at all seven stress conditions.

Altogether 42 successful tri-axial tests were conducted.

4.4 Vertical Loading

In all tests the vertical load was continuously cycled between the chosen stress and 6.0 KN/m^2 in a sinosoidal manner at a frequency of 22 cycles per second. The 6.0 KN/m^2 is the preload used to prevent the ram from leaving the sample. The duration of the load pulse was 0.045 seconds. This is the vertical load pulse duration created at a depth of 75 mm (3 inch) in a pavement of 150 mm (6 inches) total thickness by an axle moving at 50 Km/hr (Barksdale, 1971). A maximum of 10^5 load cycles were applied in

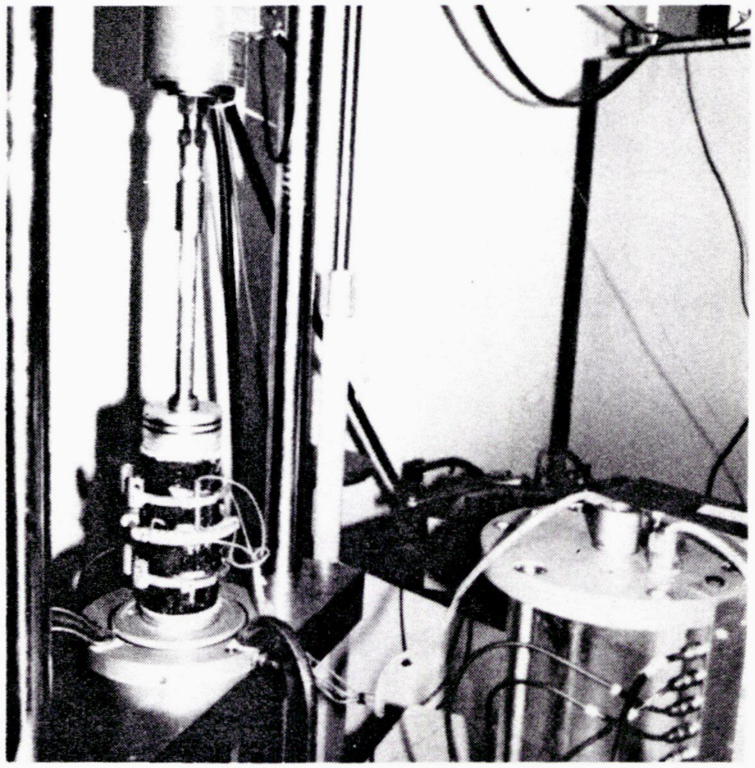


Photo 4.4 Apparatus, Prepared to
Conduct a Tension Test

each test unless the strain in the sample reached 2.5% before a 10^5 number of load repetitions was applied. In the latter case, the test was stopped when the strain in the sample reached 2.5%. During each test the permanent strains were continually recorded throughout the duration of the test.

4.5 Measurement of Permanent Deformation

Deformation of the test sample was measured by recti-linear potentiometers directly mounted on the sample. Outputs from these R.L.P.'s were fed into two twin channel stripchart recorders, an oscilloscope and a spectrum analyzer.

From the stripchart recorders, it was possible to obtain an analog plot of permanent deformation (vertical or lateral) versus the number of applied load cycles. The oscilloscope was used to observe the output waveform from the R.L.P.'s. The spectrum analyzer was used to measure the amplitude of elastic deformation recorded by each of the R.L.P.'s. The general arrangement of the equipment is shown in photos 4.5 and 4.6.

4.6 Tri-axial Test Specimens

Cylindrical specimens having a diameter of 101.5 mm and a height of 200 mm were used for the tri-axial tests.

For the purpose of casting the above specimens, asphalt concrete mixes were made according to ASTM D1561 using the asphalt contents obtained from the Marshall mix design procedure described in Chapter 3. The compaction of the specimens was

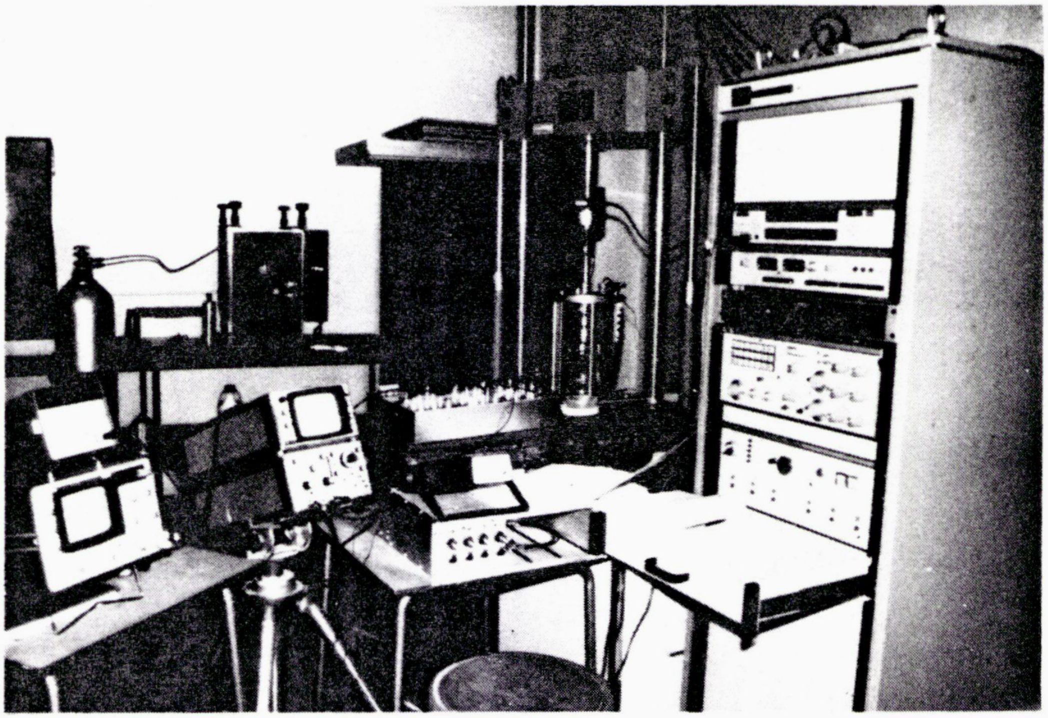


Photo 4.5 General Layout of the Load Control Equipment

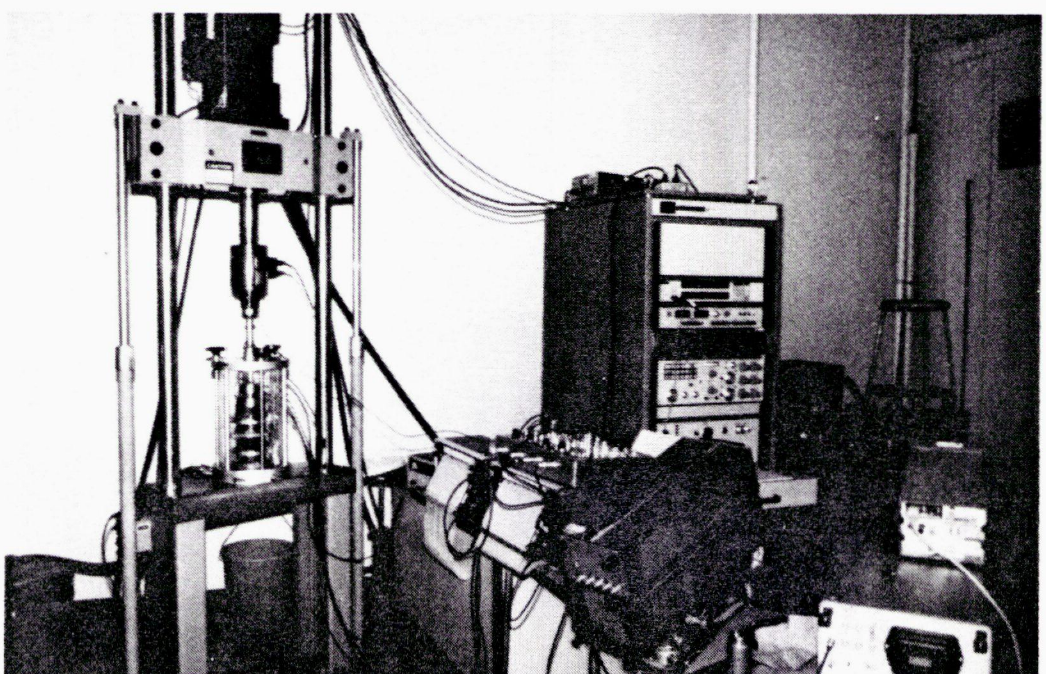


Photo 4.6 General Layout of the Strain Measuring and Recording Equipment

carried out according to the procedure described in TRB special report no-162. It consisted of two steps. Firstly, is the application of dynamic compaction using a Kneading Compactor. Secondly is the static compression using a Uni-axial compression machine. Specifications for this compaction process are summarized in Table 4.3.

Table 4.3 Compaction Procedure for Casting Tri-axial Specimens

Type of Compaction	Procedure
Dynamic	Preliminary compaction using kneading compactor by applying 60 blows at 1725 K.Pa (250 psi) foot pressure at 110°C.
Static	Uni-axial compression at 60°C using a strain rate of 0.021 mm/s (0.05 inches per minute) until a final load of 56 kN is reached.

CHAPTER 5

EXPERIMENTAL RESULTS AND ANALYSIS

5.1 Relationship Between Vertical Permanent Strain and Number of Load Repetitions

The permanent deformations in the R.L.P.'s mounted on the tri-axial test specimen can be obtained from the outputs of the stripchart recorders. As the output of the stripchart recorder is an analog plot between permanent deformation in R.L.P.'s and the number of load repetitions, the value of the permanent deformation can be obtained at any specific number of load repetitions. Then, the permanent strain recorded by each of the R.L.P.'s can be calculated by using the following equation.

permanent strain = (permanent deflection of R.L.P.
x voltage ratio of stripchart recorder
x calibration constant of R.L.P.)/gauge length

In, the case of vertical deformations, gauge length is the distance between the calipers which are used to mount the vertical R.L.P.'s. In the case of lateral deformations, gauge length is the diameter of the tri-axial test sample. The vertical permanent strain in the sample is obtained by averaging the strains recorded by two R.L.P.'s. Similarly the lateral permanent strain is obtained by averaging the values recorded by lateral R.L.P.'s.

The vertical permanent strain is then regressed with the number of load cycles applied, in order to obtain a relationship between these two quantities. This is done for each test, which means that each relationship obtained from the above regression is specific to a particular test, and hence to a particular stress condition.

A relationship of the form:

$$\varepsilon_p = f(N) \tag{5.1}$$

or $$\log \varepsilon_p = C_0 + C_1(\log N) \tag{5.2}$$

was obtained for all tests.

where, ε_p = vertical permanent strain in the asphalt concrete due to repeated loadings

C_0, C_1 = regression coefficients

N = number of applied load cycles.

As the above relationship is specific to each test, the coefficients C_0 and C_1 depend on the applied stress in the sample. In other words for any particular stress condition, the relationship between the vertical permanent strain and the number of applied load cycles can be expressed in the form of a log-linear equation.

5.1.1 Results for Virgin Asphalt Concrete

For the seven stress conditions used in this study to test the virgin asphalt concrete, the values of C_0 and C_1 obtained from the relationship between vertical permanent strain and number of applied load cycles are presented in Table 5.1. Each of these values of C_0 and C_1 represent the results obtained from at least three test samples. The values of the "Coefficient of determination (R^2)" for the above relationship ranged between 72.8% and 99.3%. The full results of regression (Standard Error of Regression Coefficients, Significance Levels of T-Tests, etc.) are given in Appendix A1.

Table 5.1 C_0 and C_1 Values of the Relationship Between Vertical Permanent Strain and Number of Load Cycles for Virgin Asphalt Concrete

Test Condition Identification Number	p (KN/m^2)	q (KN/m^2)	C_0	C_1
VUC 1	150	450	-3.32243	0.28076
VUC 2	100	300	-3.47985	0.28788
VUC 3	50	150	-3.73858	0.26140
VCC 1	450	00.0	-3.46788	0.0000
VCC 2	350	100	-3.28007	0.04808
VCC 3	250	200	-3.08395	0.08435
VT 1	-25	75	-4.61024	0.52791

Note: The letter "V" is used in front of the test identification number to indicate this is a virgin asphalt concrete test.

The relationship between vertical permanent strain and the number of applied load cycles is presented in a graphical form in Figure 5.1 for the test condition VUC 1.

Similar relationships for the other six test conditions are included in Appendix A1.

5.1.2 Results for Recycled Asphalt Concrete

The relationship between the permanent strain and the number of applied load cycles for recycled asphalt concrete was developed in a similar manner to that of virgin asphalt concrete. Also, in the case of recycled asphalt concrete this relationship was of the log-linear form.

The values of C_0 and C_1 in the relationship between vertical permanent strain and the number of applied load cycles for recycled asphalt concrete is presented in Table 5.2. R^2 values obtained for the above relationship ranged between 73.2% and 92.4%. This relationship is also shown in Figure 5.2 in a graphical form for the test RUC 1. The relationship for the other six test conditions are included in Appendix A1, together with the regression results.

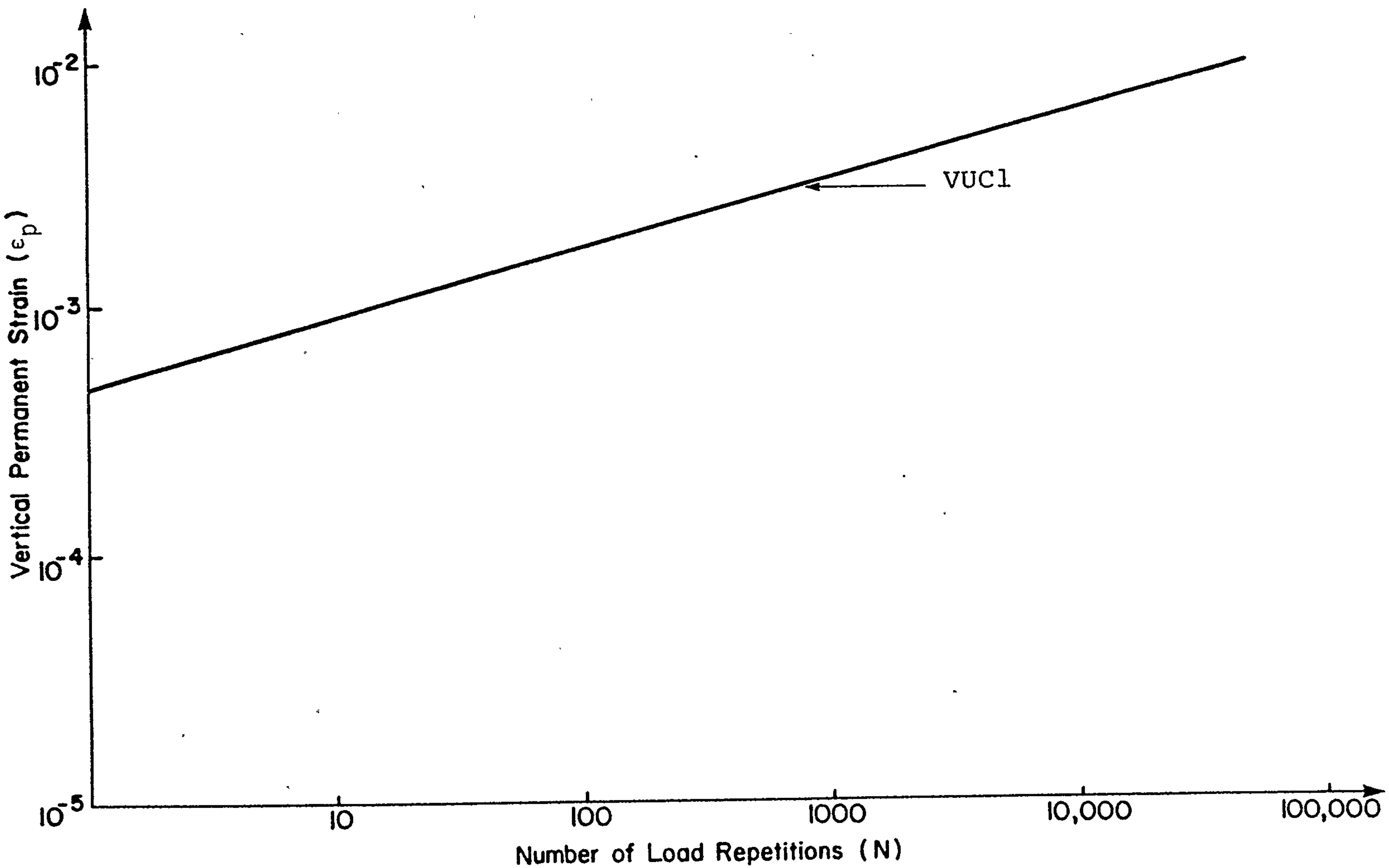


Figure 5.1 Relationship Between Vertical Permanent Strain and the Number of Load Repetitions in Virgin Asphalt Concrete

Table 5.2 C_0 and C_1 Values of the Relationship Between Vertical Permanent Strain and Number of Load Cycles for Recycled Asphalt Concrete

Test Condition Identification Number	p (KN/m^2)	q (KN/m^2)	C_0	C_1
RUC 1	150	450	-3.35729	0.28292
RUC 2	100	300	-3.28400	0.23003
RUC 3	50	150	-3.61705	0.23503
RCC 1	450	00.0	-3.55944	0.05022
RCC 2	350	100	-3.32163	0.11175
RCC 3	250	200	-3.10738	0.14681
RT 1	-25	75	-4.66694	0.55528

5.2 Relationship between C_0, C_1 and the Applied Stress Condition.

As the C_0 and C_1 coefficients were observed to be dependent on the applied stress condition in the test sample which can be characterized by the p and q values, an attempt was made to obtain a relationship of the form:

$$C_0, C_1 = f\ (p, q) \tag{5.3}$$

First, C_0 was regressed with the p and q values for each test condition. This resulted in a relationship of the form:

$$C_0 = a_1 + a_2\ (p) + a_3\ (q) \tag{5.4}$$

for both virgin and recycled asphalts. Secondly, C_1 was regressed

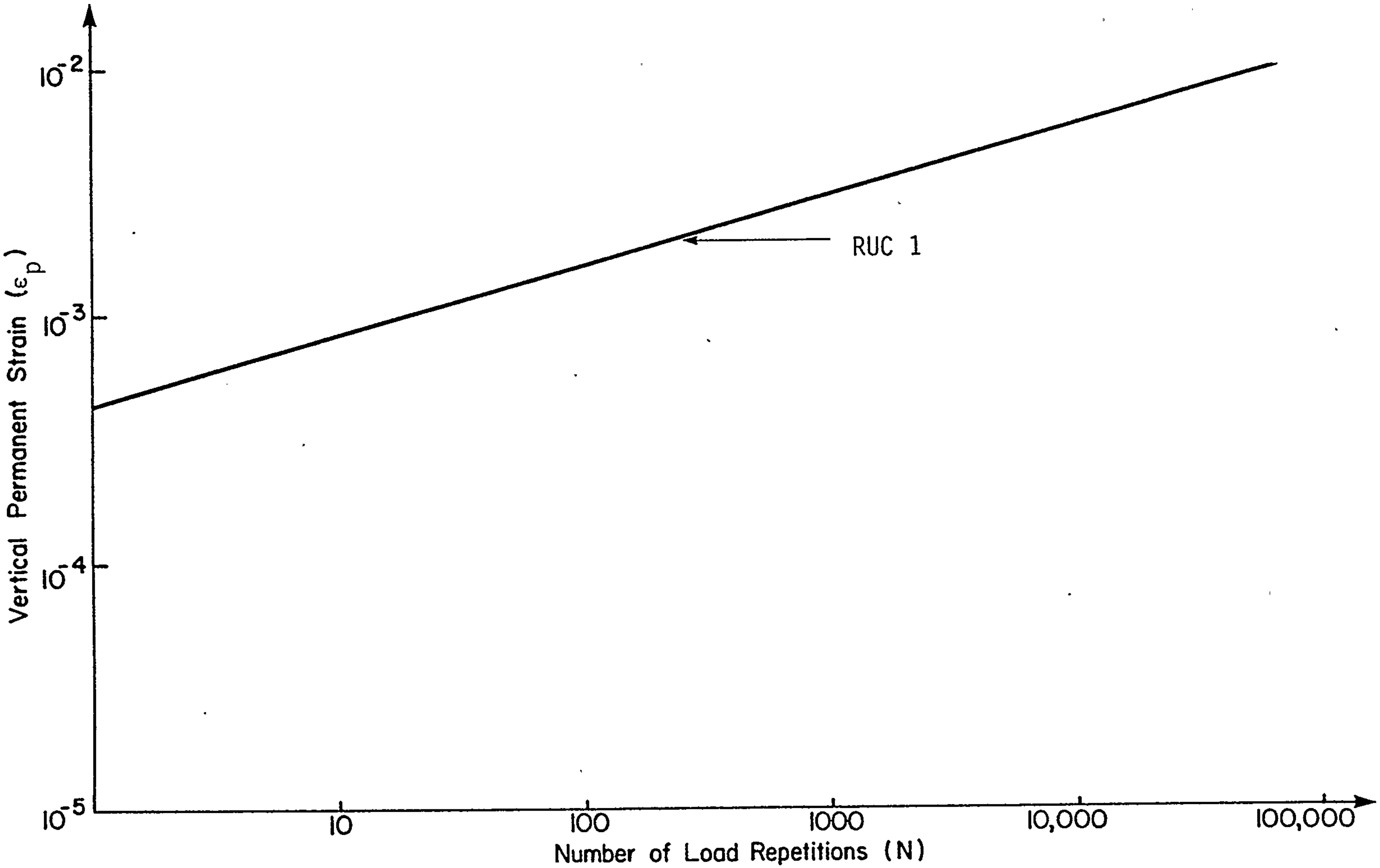


Figure 5.2 Relationship Between Vertical Permanent Strain and the Number of Load Repetitions in Recycled Asphalt Concrete

with the p and q values for each test condition.

This resulted in a relationship of the form:

$$C_1 = b_1 + b_2 (p) \qquad (5.5)$$

for both virgin and recycled asphalts.

5.2.1 C_0 and C_1 Values for Virgin Asphalt Concrete

The following relationship was obtained between C_0 and the applied stress condition for virgin asphalt concrete.

$$C_0 = -4.45061 + 0.00259 (p) + 0.00215 (q) \qquad (5.6)$$

The R^2 value was found to be 79.54% for this relationship and the complete regression result is included in Appendix A2.

Similarly, the relationship between C_1 and the applied stress condition was obtained by regressing C_1 with the p and q values for each test condition. This resulted in obtaining the following relationship:

$$C_1 = 0.40340 - 0.00101 (p) \qquad (5.7)$$

for all p<400.0 KN/m2.

C_1 = 00.0 for all p>400.0 KN/m^2

The R^2 value for equation (5.7) was found to be 86.9%. The complete result of this regression is included in Appendix A2.

5.2.2 C_0 and C_1 Values For Recycled Asphalt Concrete

The relationship between C_0 and the p and q values in the case of recycled asphalt concrete, can be expressed as follows:

$$C_0 = -4.42063 + 0.00233 (p) + 0.00231 (q) \qquad (5.8)$$

The R^2 value obtained for this regression equation was 68.5% and the complete result of the regression is included in Appendix A2. The regression of C_1 with the p and q values for each test condition for recycled asphalt resulted in the following relationship:

$$C_1 = 0.38782 - 8.3223 \times 10^{-4} (p) \qquad (5.9)$$

for all $p < 466.0$ KN/m2

$C_1 = 00.0$ for all $p > 466.0$ KN/m2.

The R^2 value for this regression equation was found to be 73.74% and the complete result of the above regression is included in Appendix A2.

5.3 Resilient Modulus and Poisson's Ratio of Asphalt Concrete

Repeated load tensile tests (Shehata, 1984) were performed on both virgin and recycled asphalt concrete samples in order to obtain the resilient modulus and the instantaneous Poisson's ratio.

Samples used for the repeated load tensile tests performed in this study were cut from tri-axial test specimens. Each sample height was approximately 63.5 mm (2 1/2 inches). In the repeated load indirect tensile test, repeated compressive loads were applied on a cylindrical sample through steel loading strips which are curved at the interface with the specimen. The load was applied vertically along the diameter of the sample. This loading configuration develops a relatively uniform tensile stress perpendicular to the plane of the applied load and along the vertical diametrical plane through the centre of the loading strip.

A vertical load of 2.0 KN (which corresponds to a tensile stress of 190 KN/m^2) was used in the indirect tensile tests.

From these tests, the Poisson's ratio can be calculated using the following equation:

$$\nu = \frac{0.066\ (v_d/h_d\ ) - 0.774}{- 0.25\ (v_d/h_d\ )} \qquad (5.10)$$

where,

h_d, v_d = instantaneous horizontal and vertical deformations in mm.

The resilient modulus can be determined by using the following equation.

$$M_R = \frac{P}{h_d x h} (0.27 + \nu\ ) \qquad (5.11)$$

where,

M_R = resilient modulus of elasticity (MPa)

h = height of the specimen (mm)

P = repeated vertical load (N)

h_d = instantaneous horizontal deformation (mm)

Using equations (5.10) and (5.11), the values of the resilient modulus and Poisson's ratio of the asphalt concrete mix were determined.

For virgin asphalt concrete specimens,

M_R = 770.0 MPa, and

ν = 0.35

For recycled asphalt concrete specimens,

M_R = 737.0 MPa, and

ν = 0.29

5.4 Prediction of Rutting in Asphalt Concrete

Rutting (permanent deformation) in an asphalt concrete layer can be predicted using the relationships between the vertical permanent strain, the number of applied load repetitions and the stress condition in asphalt concrete. These relationships are expressed by equations (5.2), (5.6) and (5.7) for virgin asphalt concrete. For recycled asphalt concrete, the above relationships are expressed by equations (5.2), (5.8) and (5.9). The equations obtained above can be expressed by the following formula:

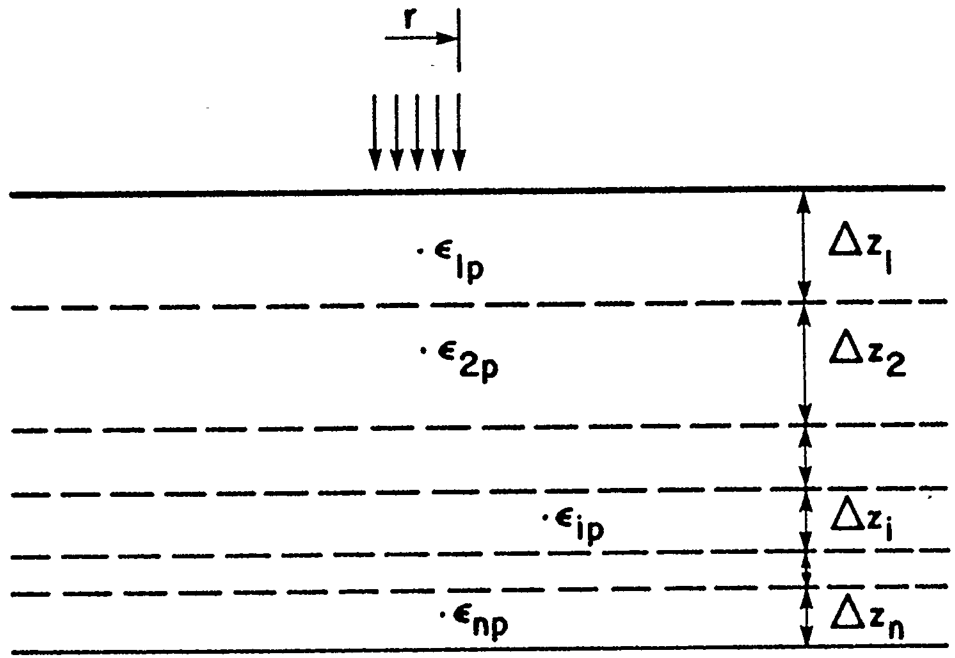
$$\varepsilon_{ip} = f (\sigma_i) \tag{5.12}$$

for any particular number of applied load repetitions.

where, ε_{ip} = vertical permanent strain in asphalt concrete due to repetitive loadings at a location i.

σ_i = stress condition, which can be defined by the p and q values at a particular location i.

For the purpose of rut prediction, the asphalt concrete layer should be divided into several sub-layers as shown in Figure 5.3. The number of sub-layers should be chosen such that the strain variation throughout the asphalt concrete layer can be accurately represented. The vertical permanent strain that will occur at the mid point of each layer due to a particular number (N) of standard axle passes can be calculated using equation (5.12). The permanent deformation occurring in each sub-layer can be calculated through multiplying the vertical permanent strain at the mid point of the sub-layer by the depth of the sub-layer. The total permanent deformation in the asphalt concrete layer is



$$D_N = \epsilon_{1p}\cdot\Delta z_1 + \epsilon_{2p}\cdot\Delta z_2 + \epsilon_{3p}\cdot\Delta z_3 + \cdots$$
$$+ \epsilon_{ip}\cdot\Delta z_i \cdots + \epsilon_{np}\cdot\Delta z_n$$

D_N — is the vertical deformation at the surface after a particular number (N) of standard axle passes

ϵ_{ip} — vertical permanent strain in the i^{th} layer

Figure 5.3 Calculation of Permanent Deformation in the Asphalt Concrete Layer the sum of the contributions from sub-layers. Hence, the deformation at the surface due to permanent deformation in the asphalt concrete layer can be expressed as:

$$D_N = \sum_{i=1}^{n} (\varepsilon_{ip} . \Delta z_i) \tag{5.13}$$

where, D_N = the permanent deformation of the asphalt concrete layer after N number of standard axle passes.

ε_{ip} = vertical permanent strain in the i^{th} layer due to N number of standard axle passes.

Δz_i = thickness of the i^{th} sub layer

In summary, from the knowledge of the variation of permanent strain with the number of load repetitions (expressed by equation (5.2)) and the change of the permanent strain with stress (expressed by equations (5.6) and (5.7) for virgin asphalt concrete, or by equations (5.8) and (5.9) for recycled asphalt concrete), the permanent deformation occuring in an asphalt concrete layer due to any particular traffic volume (corresponding to a specific number of standard axle passes) can be predicted.

5.5 Variation of Vertical Permanent Strain with Depth within the Asphalt Concrete Layer.

In figure 5.4, the variation of vertical permanent strain with depth is illustrated for a 250.0 mm (10 inch) thick virgin asphalt concrete layer resting on a subgrade having a resilient modulus of 50 MPa and a Poisson's ratio of 0.4, after being subjected to 10^6 number of load repetitions. The vertical permanent strain increases with depth. The permanent strain in the compression zone of the pavement is comparatively small compared to the permanent strain in the tension zone. In this particular case the maximum

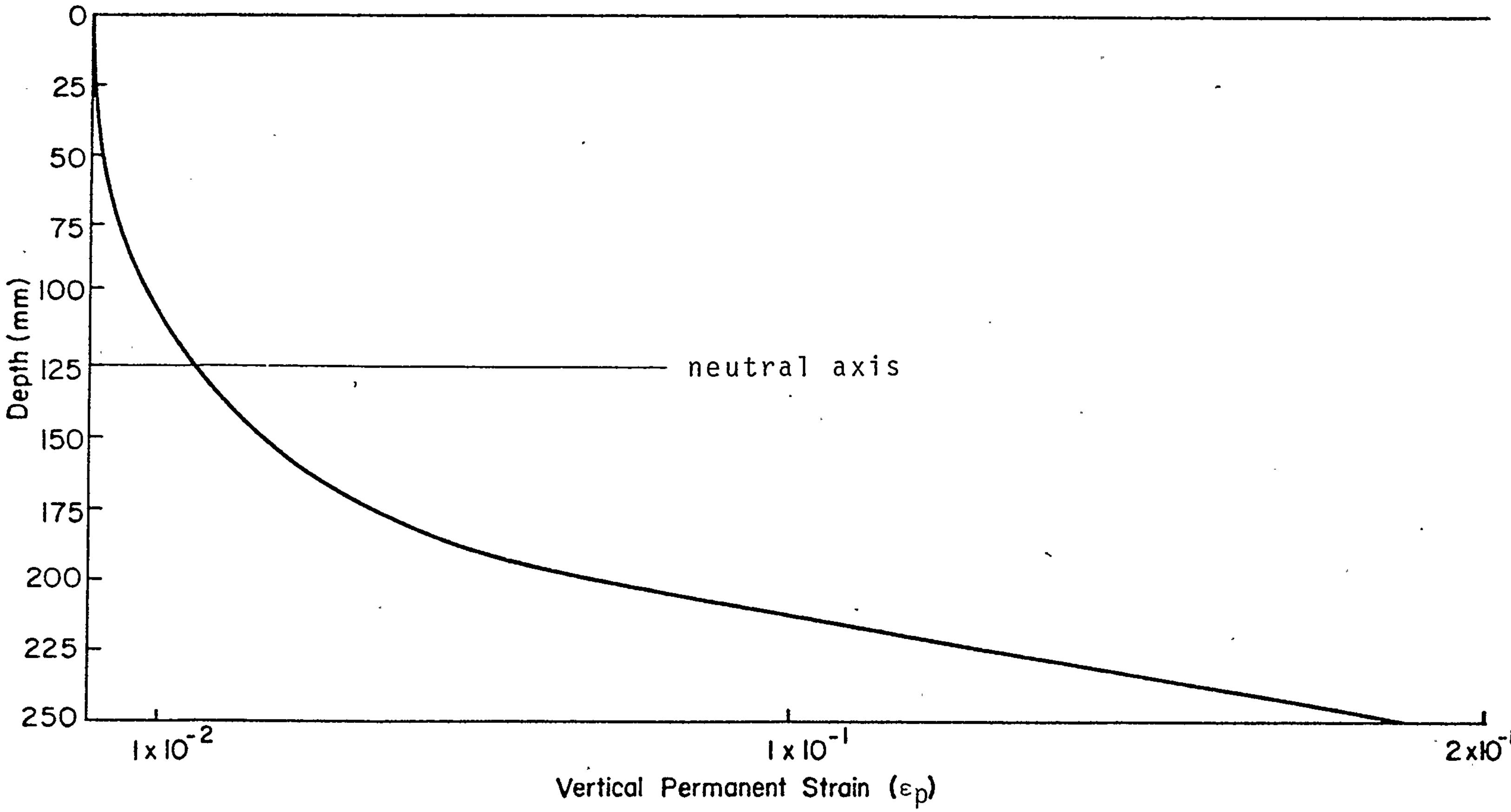


Figure 5.4 Variation of Permanent Strain with Depth

permanent strain in the compressive zone is 8% of the maximum permanent strain in the tension zone.

5.6 Accumulation of Permanent Deformation with Depth in the Asphalt Concrete Layer.

In Figure 5.5 the accumulation of the permanent deformation with increase in depth is presented for the case analyzed in paragraph 5.5.

The tension zone provides the major portion of the permanent deformation that occurs in the asphalt concrete layer. In this particular case the contribution from the tension zone is 94% of the total permanent deformation in the asphalt concrete layer. A similar variation of permanent deformation with depth has been reported by Morris et al (Morris et al, 1974).

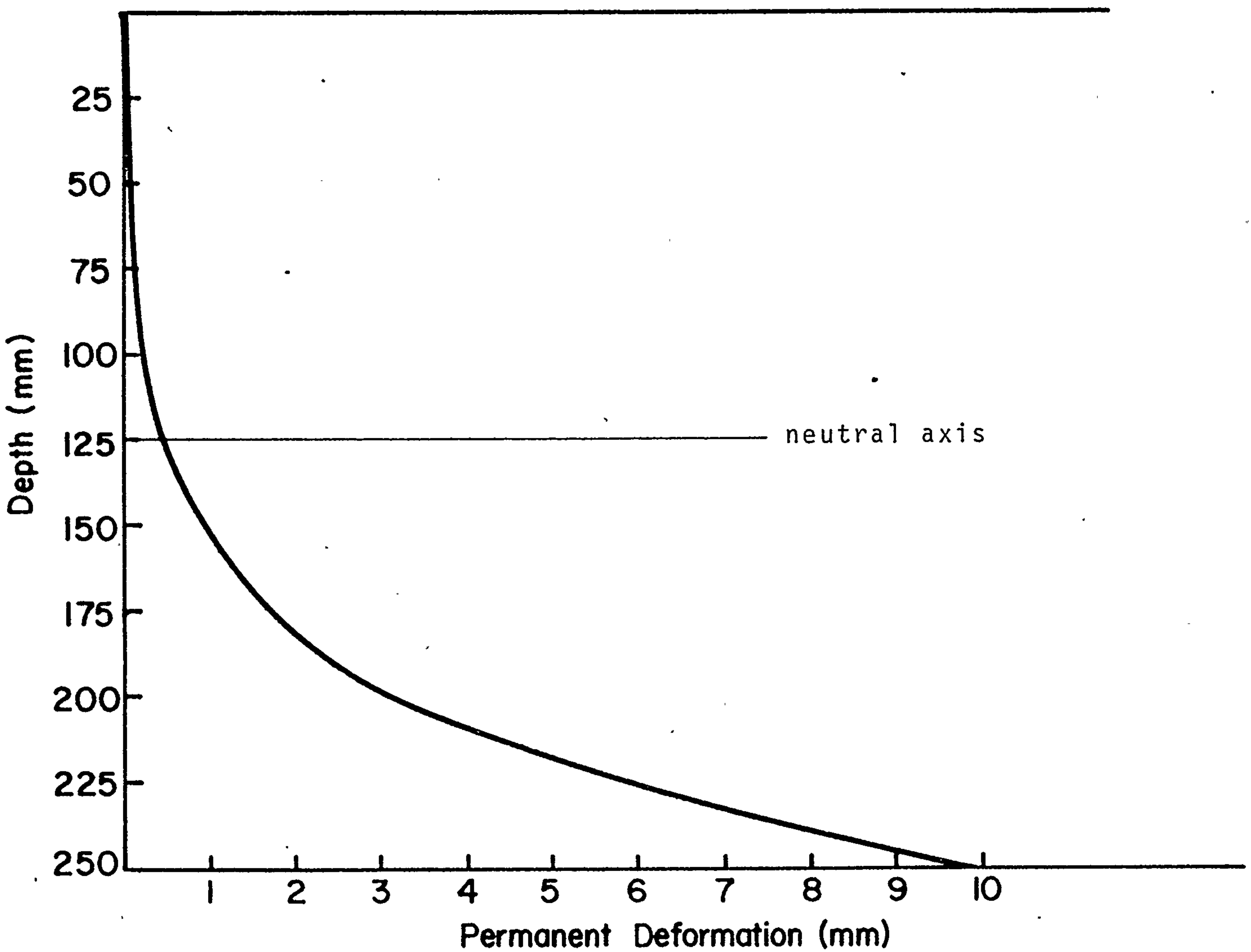


Figure 5.5 Variation of Permanent Deformation with Depth

CHAPTER 6

NUMERICAL EXAMPLE AND CHARTS FOR DETERMINING PAVEMENT THICKNESS

6.1 Numerical Example - The Use of Predictive Equations to Calculate the Permanent Deformation in an Asphalt Concrete Layer.

Depth of virgin asphalt concrete layer = 250 mm

Resilient modulus of virgin asphalt concrete = 770 MPa

Poisson's ratio of virgin asphalt = 0.35

Resilient modulus of subgrade = 50 MPa

Poisson's ratio of subgrade = 0.4

Number of standard axle passes = 10^6

In order to calculate the permanent deformation that will occur in the asphalt concrete under the wheel path, the asphalt concrete layer is divided into five sub-layers.

In this example it is assumed that all the axles will travel along the same wheel path (i.e. there is no lateral distribution of wheel paths). First, the stresses (i.e. stress invariants p and q) should be computed at mid depth of each sub-layer in the asphalt concrete. For this purpose the pavement can be represented by a two layer system, and the elastic layer theory can be used to determine the stresses at the required depths in the asphalt concrete layer. This task can be made easier by using the Chevron5L multi-layer elastic analysis program to compute the stresses. The stresses that will occur in the asphalt concrete sub-layers in this example are summarized in Table 6.1.

Table 6.1 Summary of the Permanent Deformation Computation for the 250 mm Virgin Asphalt Concrete Layer

Depth (mm)	p (KPa)	q (KPa)	C_0	C_1	ε_p	Deflection (mm)
25	463.59	20.85	-3.20509	0.0000	0.624×10^{-3}	0.0312
75	237.50	215.92	-3.37124	0.16352	0.407×10^{-2}	0.2035
125	81.94	226.90	-3.75057	0.32065	0.149×10^{-1}	0.745
175	-36.09	231.37	-4.04663	0.43985	0.391×10^{-1}	1.955
225	-147.77	296.49	-4.19587	0.55264	0.132×10^{0}	6.6

At the mid depth of the top most sub-layer, the stress condition can be defined by

$$p = 463.59 \text{ KN/m}^2$$

$$q = 20.85 \text{ KN/m}^2$$

Using equations (5.6) and (5.7) we can calculate C_0 and C_1 corresponding to the above stress condition that exists at mid depth of the top most sub-layer.

$$C_0 = -4.45061 + 0.00259\,(463.59) + 0.00215\,(20.85) = -3.20509$$

As p is greater than 400.0 KN/m^2, then:

$$C_1 = 00.00$$

Now, the plastic strain at mid depth of the top most sub-layer after 10^6 load repetitions can be computed by equation (5.2):

$\log \varepsilon_p = -3.20509 + 00.00 \times \log 10^6$

or $\varepsilon_p = 6.24 \times 10^{-4}$

This plastic strain is taken as the average value of the 50.0 mm thick top sub-layer. Hence, the contribution of permanent deformation from the top sub-layer is,

$50.0 \times 6.24 \times 10^{-4} = 0.0312$ mm.

The permanent strain and permanent deformation values for the other four sub-layers are presented in Table 6.1. After performing the above calculations, the total permanent deformation in the asphalt concrete layer can be obtained by taking the summation of the permanent deformations that occur in each sub-layer.

Hence the total permanent deformation of the asphalt concrete layer under consideration is,

$D_N = 0.0312 + 0.2035 + 0.745 + 1.955 + 6.6 = 9.53$ mm.

6.2 Charts for Determining Pavement Thickness Based on Rutting

6.2.1 Basis for the Analysis and Design Criteria

In this research, only full depth asphalt concrete pavements are considered. They are represented by a two-layer elastic system. Each layer (i.e. the asphalt concrete layer and the subgrade), is characterized by its resilient modulus and Poisson's ratio. For design purposes, the loading on the pavement is represented by the 80 KN standard axle with dual tires at each side as mentioned in paragraph 4.2.1.

The Chevron5L multi-layer elastic analysis program provides the most convenient method to analyze the full depth asphalt pavement loaded by the 80 KN standard axle load and to determine the stresses in the pavement. The Chevron5L program was modified by incorporating the permanent strain predictive equations (5.2), (5.6) and (5.7) in the case of virgin asphalt concrete and equations (5.2), (5.8) and (5.9) in the case of recycled asphalt concrete pavements. This modification allows determination of the permanent deformation in an asphalt concrete layer at a specific number of load repetitions. Also, the modified Chevron5L program has the ability to calculate the permanent deformations that occur in the asphalt concrete layer of ten pavements having different thicknesses. These thicknesses can increase in steps of 25 mm (1 inch) from any specified initial value, and may be used with a specific subgrade and a number of load repetitions. Using this modified program, the permanent deformation of the asphalt concrete layer of a series of pavements resting on different subgrades was calculated. Five types of subgrades and three different numbers of load repetitions were considered to develop charts for determining the thickness of asphalt concrete pavements such that the permanent deformation in the asphalt concrete layer is limited to a specific value.

6.2.2 Input Data for the Modified Chevron5L Program

Resilient modulus and Poisson's ratio values of 770.0 MPa and 0.35 were used to represent the characteristics of virgin asphalt concrete. In the case of recycled asphalt concrete pavements, values of 737 MPa and 0.29 were used as the resilient modulus and Poisson's ratio of the asphalt mixture.

Three different values of Poisson's ratios of soil were used with each of the five subgrades. They are summarized in Table 6.2. Also permanent deformations of the asphalt concrete layers were determined at 10^5, 5×10^5 and 10^6 number of load repetitions.

Table 6.2 Subgrade Properties used in the Development of Charts for Determining Pavement Thickness.

Poisson's ratio	Resilient Modulus of Subgrade (MPa)
0.4	10 20 50 70 100
0.3	10 20 50 70 100
0.2	10 20 50 70 100

An initial value of the asphalt concrete layer thickness was specified as 125 mm (5 inch). Hence, each run of the program would calculate permanent deformations that occur in the asphalt concrete layer of ten pavements starting with an initial pavement thickness of 125 mm (5 inches) and increasing in steps of 25 mm (1 inch) for a specific subgrade and a number of load repetitions. It is also necessary to specify the load on a single tire of the 80 KN standard axle and the tire pressure, to define the load configuration on the pavement. Values of 20 KN and 483 KPa, respectively, were used for these two parameters.

6.2.3 Pavement Thickness Charts and Their Limitations

From the output of several runs of the computer program in which the inputs specified in paragraph 6.2.2 were used, it was possible to develop thickness charts for virgin and recycled asphalt concrete pavements. The criterion used to develop the design charts, was to limit the permanent deformation in the asphalt concrete layer of the pavement to 12.5 mm. This particular value was chosen because it is generally accepted that rutting becomes critical only after rut depth exceeds 12.5 mm (RTAC publication, 1977).

Thickness charts for virgin asphalt concrete are presented in Figure 6.1, while similar ones for recycled asphalt concrete pavements are shown in Figure 6.2. From these charts, it can be seen that the thickness required for recycled asphalt concrete pavements is less than that for virgin asphalt concrete ones.

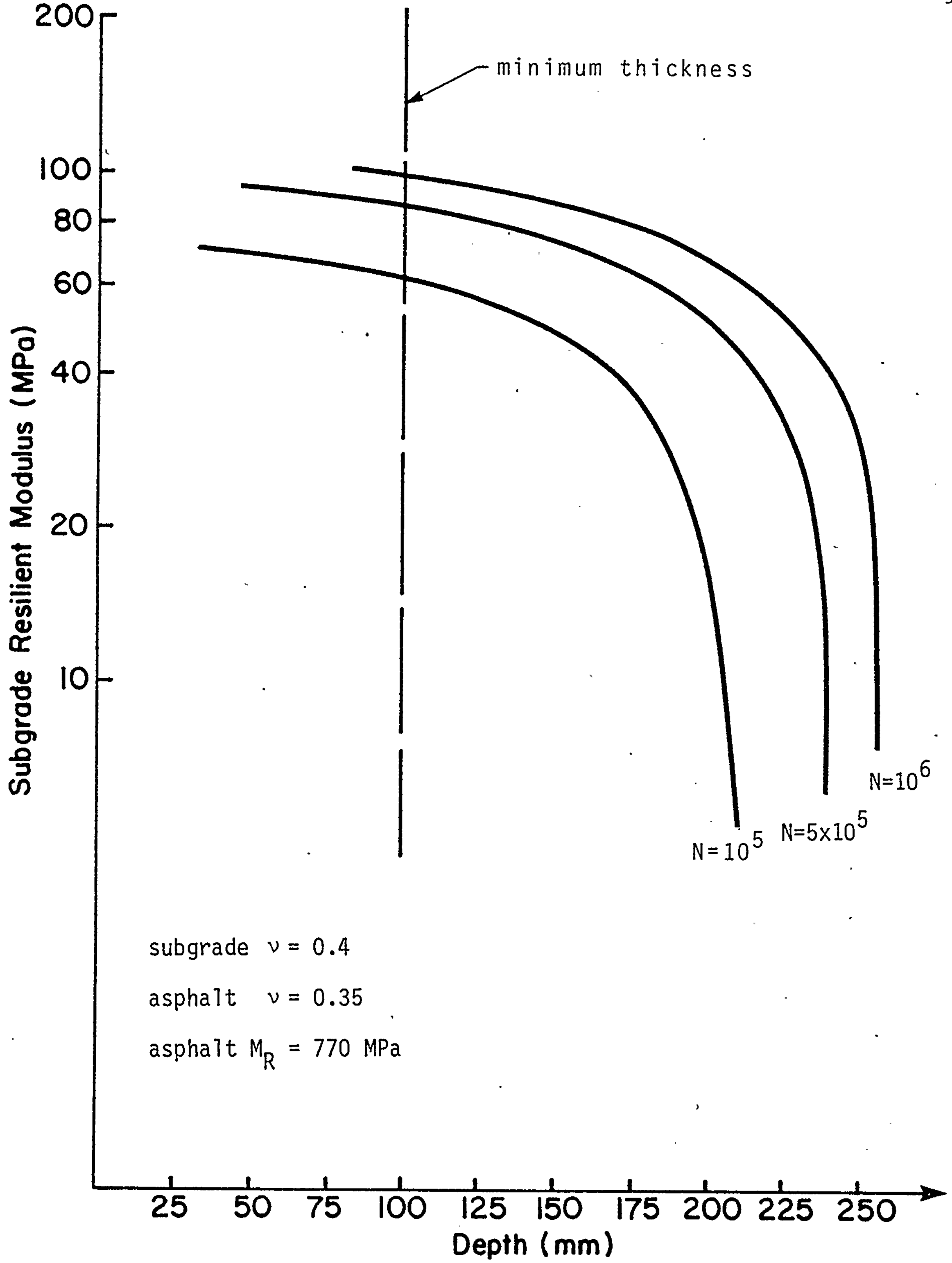


Figure 6.1 Thickness Chart for Virgin Asphalt Concrete Pavements Based on Rutting

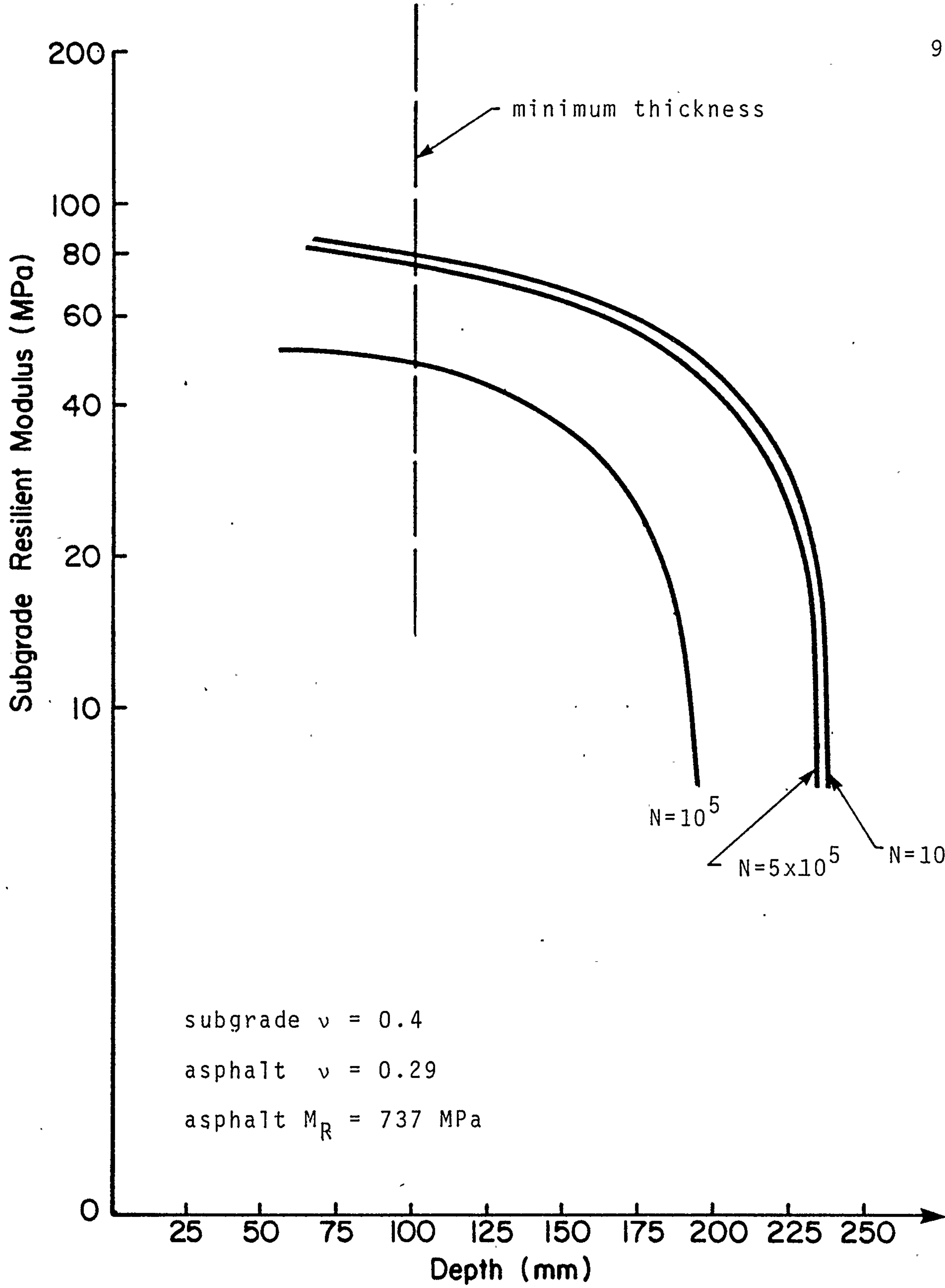


Figure 6.2 Thickness Chart for Recycled Asphalt Concrete Pavements Based on Rutting

Also the difference in the required pavement thickness for virgin and recycled asphalt concrete pavements increases as the number of load repetitions increases. Hence the economy of recycled pavements becomes significantly large in the case of pavements designed for large traffic volumes. However, these charts have several limitations.

1. They are developed from tests carried out on one mixture of each asphalt concrete. For general design purposes it is necessary to test several different mixtures of virgin and recycled asphalt concretes before being able to develop the appropriate charts.

2. They are applicable only to asphalt concrete at a temperature of 20°C. In the field, temperature varies with seasons and with depth in the pavement. Hence in order to obtain a reliable thickness, design charts can be developed only after investigating the permanent deformation properties of asphalt concrete at several different temperatures.

3. These charts are developed on the basis of laboratory test results. However, the conditions in the field are different from those in the laboratory, mainly due to two reasons. The first one is the effect of rest periods and the second is the effect of lateral distribution of wheel paths. Hence, these two factors must be taken into account in order to obtain a practical design. From past research it has been found that rest periods do not have a significant effect on the accumulation of permanent deformation (Brown, 1974).

Studies on the lateral distribution of wheel paths of heavy vehicles, indicate that about 40% of the vehicles travel on the same wheel path in a design lane (PCA Publication, 1966). The remaining vehicles travel in other wheel paths that are normally distributed around the mean wheel path. Hence, in order to relate the charts developed in this research to field conditions, the number of load repetitions in the chart should be taken as being equal to 40% of the equivalent number of axle passes in the field.

4. The design charts are based on limiting the permanent deformation in the asphalt concrete layer. However, total permanent deformation (rutting) in the pavement consists of the permanent deformation in the asphalt concrete layer plus the permanent deformation in the subgrade. Hence the use of these charts will result in pavements with total rutting (permanent deformation) greater than 12.5 mm. In order to investigate the magnitude of the contribution of the subgrade to rutting, the equation developed by Brown (Brown and Bell, 1979) for predicting the permanent deformation in weak subgrade soil (a silty clay known as Keuper Marl) was used to calculate the permanent deformation of the subgrade. Several cases were analyzed in which the Keuper Marl soil was considered as the subgrade underneath virgin asphalt concrete pavements of different thicknesses. The predictive equation for a Keuper Marl soil is,

$$100\ \varepsilon_{kp} = (\ q/70\ )^2 \log N \qquad (6.1)$$

where q is the stress invariant representing the octahedral shear stress, N is the number of load repetitions and ε_{kp} is the vertical permanent strain in Keuper Marl. Keuper Marl has a resilient modulus of 20 MPa and a Poisson's ratio of 0.4. The permanent deformation in the subgrade was calculated at 10^6 number of load repetitions in a manner similar to that of the asphalt concrete by dividing the subgrade into several sub-layers. A total subgrade depth of 150 mm was analyzed under each asphalt concrete pavement. Asphalt concrete pavement thicknesses varied from 125 mm (5 inches) to 350 mm (14 inches). Results of this analysis are summarized in Table 6.3. From these results it can be seen that the contribution from the subgrade to rutting does not exceed 3.0% of the permanent deformation in the asphalt concrete. Also other researchers have reported that the contribution from the subgrade to rutting in the pavement structure is very small (Morris et al, 1974).

Hence the charts developed in this research may be used in preliminary designs to determine the pavement thickness required to limit the pavement rutting to 12.5 mm (1 inch).

Table 6.3 Permanent Deformation in Subgrade as a Percentage of Rutting in an Asphalt Concrete Layer

Depth of Asphalt Concrete Layer (mm)	Rutting in Subgrade (%)
125	0.16
150	0.44
175	0.94
200	1.73
225	2.03
250	2.27
275	2.17
300	1.88
325	1.28
350	1.05

6.2.4 Effect of Subgrade Poisson's Ratio on Rutting in the Asphalt Concrete

A change in the Poisson's ratio of the subgrade can affect the permanent deformation in the asphalt concrete only through the resulting change in the stress level in the asphalt concrete layer. The effect of the subgrade Poisson's ratio on rutting in the asphalt concrete is clearly illustrated in the charts presented in Figures 6.3 and 6.4.

The chart for virgin asphalt concrete at 10^6 load repetitions is shown in Figure 6.3, and the corresponding chart for recycled asphalt is shown in Figure 6.4. In each chart, there are three curves for three different

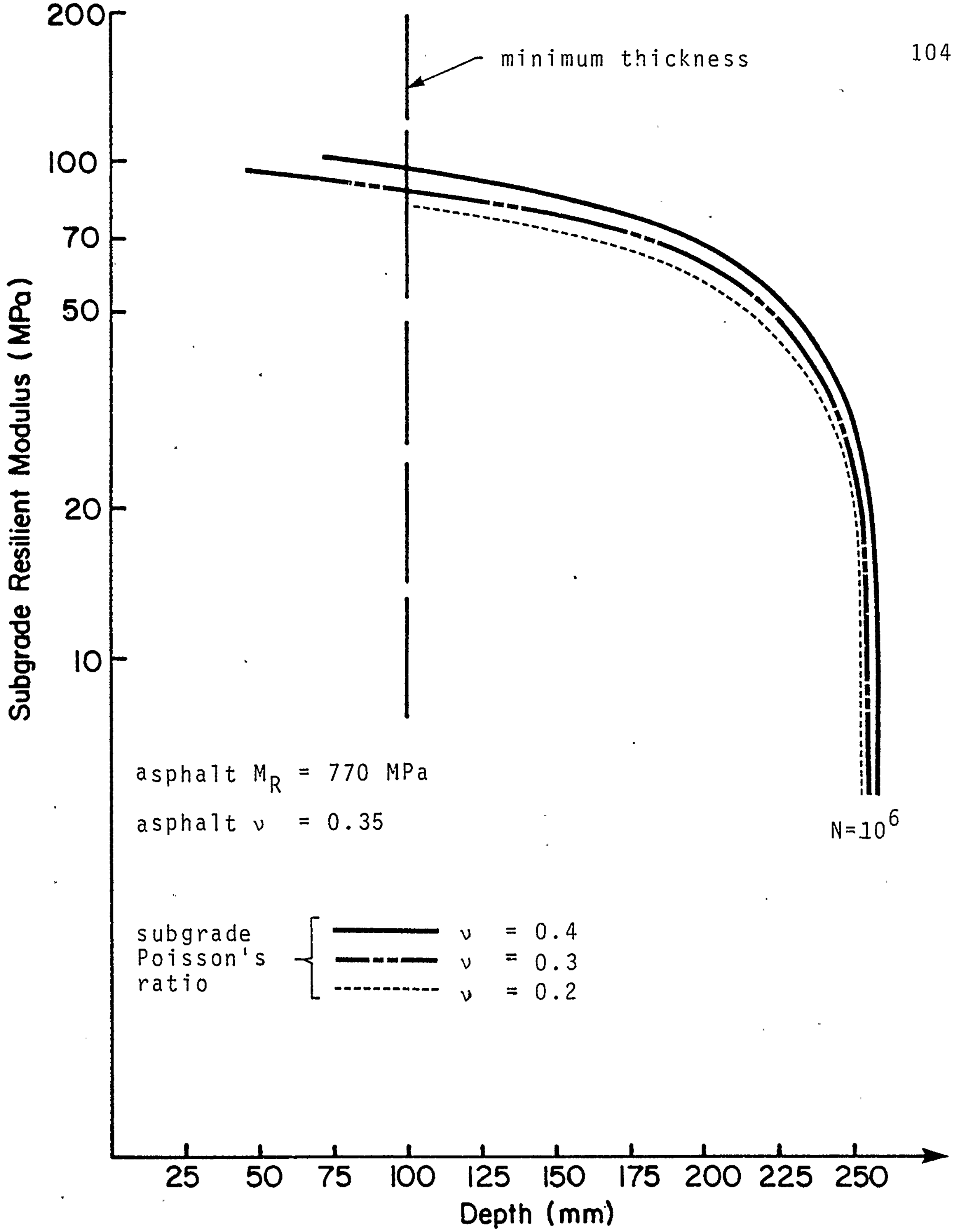


Figure 6.3 Effect of Subgrade Poission's Ratio on Permanent Deformation in Virgin Asphalt Concrete

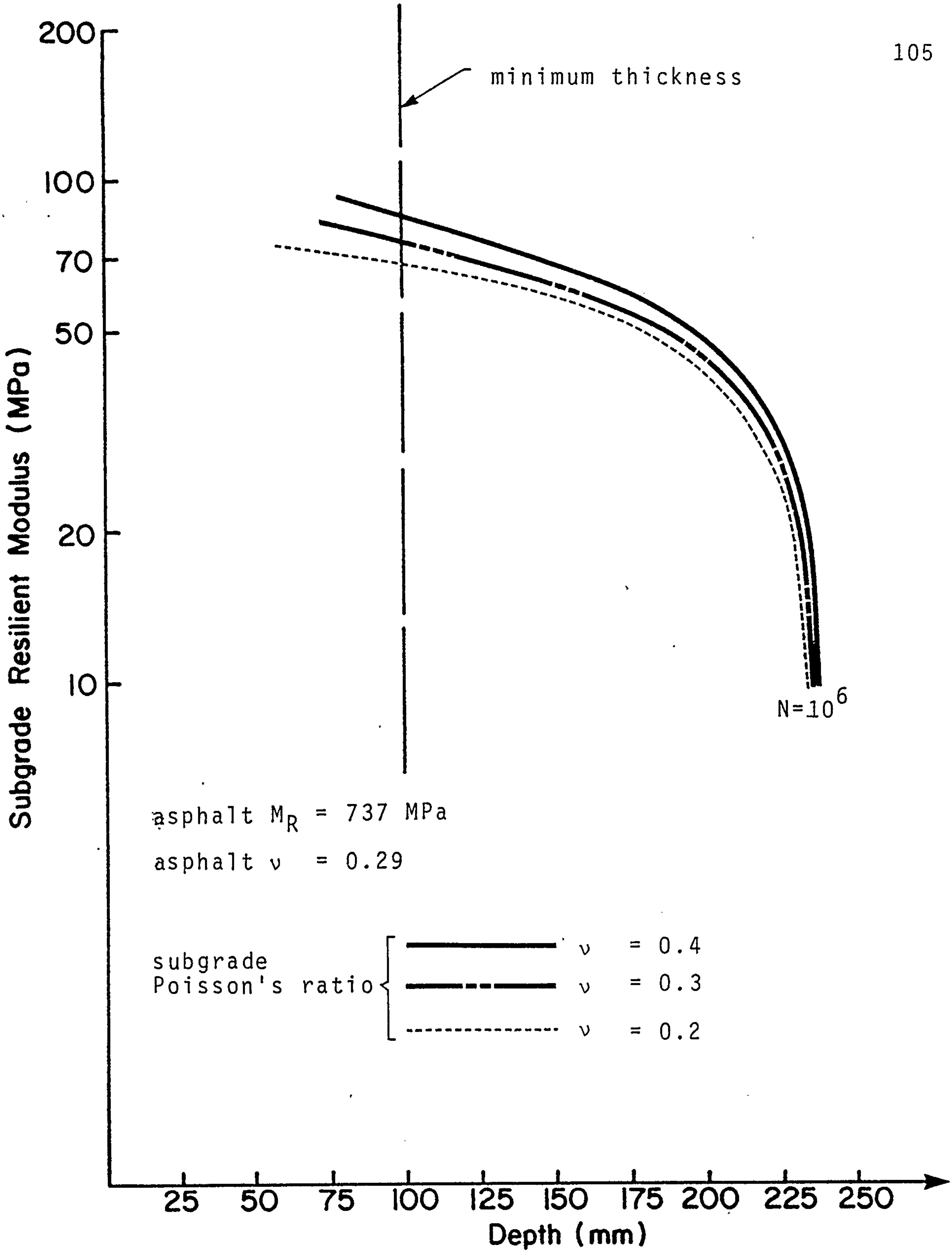


Figure 6.4 Effect of Subgrade Poisson's Ratio on Permanent Deformation in Recycled Asphalt Concrete

Poisson's ratios of subgrade. From these charts, it can be seen that when the Poisson's ratio of the subgrade increases, the pavement thickness required to limit the rutting increases. This implies that the pavement deformation in the asphalt concrete layer increases with the increase in the Poisson's ratio of the subgrade for both virgin and recycled asphalt concrete pavements.

CHAPTER 7

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary and Conclusions

This study was undertaken to investigate the performance of both virgin and recycled asphalt concrete with respect to permanent deformation under repeated loading, when they are used in full depth asphalt concrete pavements. In order to achieve this objective, the following steps were taken.

1. Aged pavement material was obtained from a main city arterial and was analyzed to determine the properties of aged binder and properties of aggregate.

2. The recycled asphalt concrete mix was designed by mixing aged pavement material with some virgin material (i.e new binder and aggregate). In designing this mix, the percentage of reclaimed pavement material was kept at a value slightly less than 50% by weight in order to obtain a mix that can be used either in a drum mix plant or a batching plant in the field.

3. A virgin mixture with comparable characteristics to the recycled mixture was designed to compare the performances of virgin and recycled asphalt concrete.

4. Repeated load tri-axial tests were used to simulate the stress conditions that occur in the field in full depth asphalt concrete pavements.

Empirical relationships were developed using the tri-axial test results to predict the permanent strain that occurs in asphalt concrete in terms of number of applied load repetitions and the resulting stresses in the pavement.

Altogether, three basic relationships were developed for each asphalt concrete. Their format is given in equations (5.2), (5.4) and (5.5). Equation (5.2) relates the permanent deformation to the number of applied load repetitions. Several other researchers have obtained similar relationships for the virgin asphalt concrete (Mclean and Monismith, 1975; Brown and Bell, 1979; Allen and Deen, 1981 and Francken, 1977). They are given in equations (2.5), (2.6) and (2.7). Monismith's equation has two additional terms with higher powers of log N but the coefficents of these terms are very small. Brown had defined N in terms of the duration of the test (t). The basic format in Brown's work is very similar to that obtained in this investigation.

When the expressions for C_0 and C_1 are considered, more variety exists in reported results in the literature. Monismith has found that C_0 is a function of the vertical elastic strain and the applied deviator stress (q), while C_1 is independent of the applied stress. Brown had found C_0 to be a function of q and C_1 to be a function of p Francken had found C_0 to be a function of q and the resilient modulus of the asphalt concrete, and C_1 to be a function of the (maximum vertical stress - maximum vertical stress required to create plastic failure). According to basic equations (5.4) and (5.5) which were obtained from the results of the present study, C_0 is a function of p and q while C_1 is only a function of p (mean normal stress). Hence the results obtained in this study generally agree with the results reported by other research workers for virgin asphalts. From equations (5.2), (5.4) and (5.5) which were obtained from the results of the tests carried out in this study, the following conclusions can be drawn regarding the factors affecting permanent deformation in both virgin and recycled asphalt concretes.

i. For any particular stress level, the permanent strain is related to the number of load repetitions by a log-linear relationship.

ii. The permanent strain after one load repetition (i.e. C_0) increases with the increase in the mean normal stress (p). Also it increases with the increase in the octahedral shear stress q, where q is also equal to the deviator stress under tri-axial test stress conditions.

iii. The accummulation of the permanent strain with the number of load repetitions (represented by C_1) decreases with the increase in the mean normal stress (p).

5. The predictive equations (5.2), (5.4) and (5.5) were used to calculate the permanent deformation in the asphalt concrete layer of a full depth asphalt concrete pavement, by dividing the layer into several sub-layers. Charts for determining the thickness of full depth asphalt concrete pavements were developed by limiting the total permanent

deformation in the asphalt concrete layer to 12.5 mm. From these charts it can be concluded that a thinner pavement is required if recycled asphalt is used in a full depth asphalt pavement instead of virgin asphalt, when both are designed to withstand the same number of load repetitions. Also, the economy in the thickness of the recycled asphalt concrete is more significant when high numbers of load repetitions are considered. Hence, for roads with high traffic volumes, the limited tests conducted in the present study indicate that considerable savings can be achieved by using recycled asphalt concrete pavements.

6. There are limitations in the basic equations (5.4) and (5.5) for both virgin and recycled asphalt concretes. They over predict the permanent deformation in thin asphalt concrete pavements, particularly when the subgrade is weak. The reason for this is that equations (5.4) and (5.5) do not accurately predict the permanent deformation under very high lateral stresses. This in turn, is due to the inability to simulate very high lateral stresses (i.e. both tension and compression) in the laboratory with conventional tri-axial test equipment.

7.2 Recommendations

The following suggestions are recommended for future research work:

1. The test equipment can be further improved by using more sensitive deformation measurement devices and a high pressure tri-axial cell.

2. The testing program used in the present study can be expanded to determine the behavior of asphalt concrete with respect to rutting under different temperatures.

3. More tests involving different asphalt concrete mixtures have to be carried out so that more general charts can be developed for determining the pavement thickness based on rutting.

4. The variation of the temperature with depth in the pavement and the variation of resilient modulus with stress level should be considered in order to produce more accurate thickness design charts.

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APPENDIX A1

REGRESSION RESULTS OF THE LOG-LINEAR RELATIONSHIP BETWEEN PERMANENT STRAIN AND THE NUMBER OF LOAD REPETITIONS AT DIFFERENT STRESS LEVELS FOR BOTH VIRGIN AND RECYCLED ASPHALT CONCRETES

FORMAT OF THE REGRESSION EQUATION: $Log(\varepsilon_p) = C_0 + C_1 Log(N)$

PLASTIC STRAIN VS LOAD CYCLES VUC1.VERT

FILE NONAME (CREATION DATE = 03/19/86) 03/19/86

* * * * M U L T I P L E   R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = Log(\varepsilon_p)$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X = Log(N)

MULTIPLE R	0.96417
R SQUARE	0.92963
ADJUSTED R SQUARE	0.92750
STANDARD ERROR	0.06882

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE
REGRESSION	1	2.06454	2.06454
RESIDUAL	33	0.15628	0.00474

F = 435.95704 SIG F = 0.0000

------------------------------ VARIABLES IN THE EQUATION ------------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X	0.28076	0.01345	0.96417	20.880	0.0000
(CONSTANT)	-3.32243	0.05726		-58.028	0.0000

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $Log(\varepsilon_p) = C_0 + C_1 \, Log(N)$

PLASTIC STRAIN VS LOAD CYCLES VUC2.VERT

FILE NONAME (CREATION DATE = 03/25/86) 03/25/86

* * * * M U L T I P L E   R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = Log(\varepsilon_p)$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X = Log(N)

MULTIPLE R	0.96239
R SQUARE	0.92620
ADJUSTED R SQUARE	0.92299
STANDARD ERROR	0.07267

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE
REGRESSION	1	1.52435	1.52435
RESIDUAL	23	0.12146	0.00528

F = 288.64547 SIG F = 0.0000

---------------------------- VARIABLES IN THE EQUATION ----------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X	0.28788	0.01694	0.96239	16.990	0.0000
(CONSTANT)	-3.47985	0.07298		-47.683	0.0000

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $Log(\varepsilon_p) = C_0 + C_1 \, Log(N)$

PLASTIC STRAIN VS LOAD CYCLES VUC3.VERT

FILE NONAME (CREATION DATE = 03/25/86) 03/25/86

* * * * M U L T I P L E R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = Log(\varepsilon_p)$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X = Log(N)

MULTIPLE R	0.97250	ANALYSIS OF VARIANCE				
R SQUARE	0.94575		DF	SUM OF SQUARES	MEAN SQUARE	
ADJUSTED R SQUARE	0.94429	REGRESSION	1	1.94510	1.94510	
STANDARD ERROR	0.05491	RESIDUAL	37	0.11156	0.00302	

F = 645.08624 SIG F = 0.0000

------------------------------ VARIABLES IN THE EQUATION ------------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X	0.26140	0.01029	0.97250	25.399	0.0000
(CONSTANT)	-3.73858	0.04462		-83.783	0.0000

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $\mathrm{Log}(\varepsilon_p) = C_0 + C_1\ \mathrm{Log}(N)$

PLASTIC STRAIN VS LOAD CYCLES VCC2.VERT

FILE NONAME (CREATION DATE = 05/07/86) 05/07/86

* * * * M U L T I P L E   R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = \mathrm{Log}(\varepsilon_p)$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. $X = \mathrm{Log}(N)$

MULTIPLE R	0.96054
R SQUARE	0.92264
ADJUSTED R SQUARE	0.91158
STANDARD ERROR	0.96352E-02

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE
REGRESSION	1	0.00775	0.00775
RESIDUAL	7	0.00065	0.00009

F = 83.48072 SIG F = 0.0000

---------------------------- VARIABLES IN THE EQUATION ----------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X	0.04808	0.00526	0.96054	9.137	0.0000
(CONSTANT)	-3.28007	0.02290		-143.212	0.0000

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $\text{Log}(\varepsilon_p) = C_0 + C_1 \text{Log}(N)$

PLASTIC STRAIN VS LOAD CYCLES VCC3.VERT

FILE NONAME (CREATION DATE = 05/07/86) 05/07/86

* * * * M U L T I P L E   R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = \text{Log}(\varepsilon_p)$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. $X = \text{Log}(N)$

MULTIPLE R	0.85381
R SQUARE	0.72899
ADJUSTED R SQUARE	0.71964
STANDARD ERROR	0.03195

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE
REGRESSION	1	0.07965	0.07965
RESIDUAL	29	0.02961	0.00102

F = 78.00584 SIG F = 0.0000

---------------------------- VARIABLES IN THE EQUATION ----------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X	0.08435	0.00955	0.85381	8.832	0.0000
(CONSTANT)	-3.08395	0.04234		-72.833	0.0000

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $\text{Log}(\varepsilon_p) = C_0 + C_1 \text{Log}(N)$

PLASTIC STRAIN VS LOAD CYCLES VT1.VERT

FILE NONAME (CREATION DATE = 04/02/86) 04/02/86

* * * * M U L T I P L E   R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = \text{Log}(\varepsilon_p)$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X = Log(N)

		ANALYSIS OF VARIANCE			
			DF	SUM OF SQUARES	MEAN SQUARE
MULTIPLE R	0.99658				
R SQUARE	0.99316				
ADJUSTED R SQUARE	0.99274	REGRESSION	1	3.42475	3.42475
STANDARD ERROR	0.03839	RESIDUAL	16	0.02358	0.00147

F = 2324.10727 SIG F = 0.0000

------------------------------ VARIABLES IN THE EQUATION ------------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X	0.52791	0.01095	0.99658	48.209	0.0000
(CONSTANT)	-4.61024	0.04352		-105.932	0.0000

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

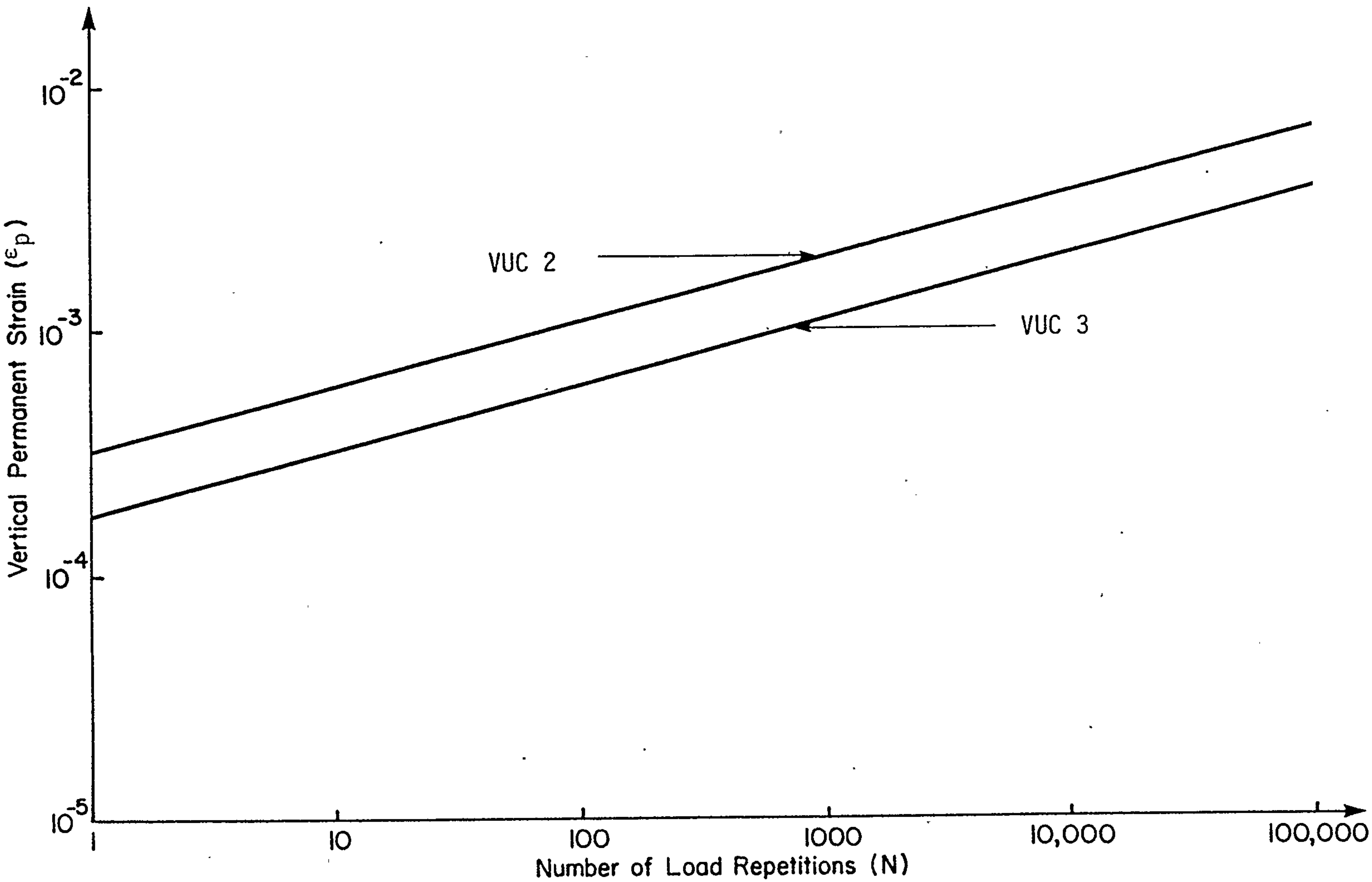

Figure A 1.1 Relationship Between Vertical Permanent Strain and the Number of Load Repetitions in Virgin Asphalt Concrete (under uni-axial stress conditions)

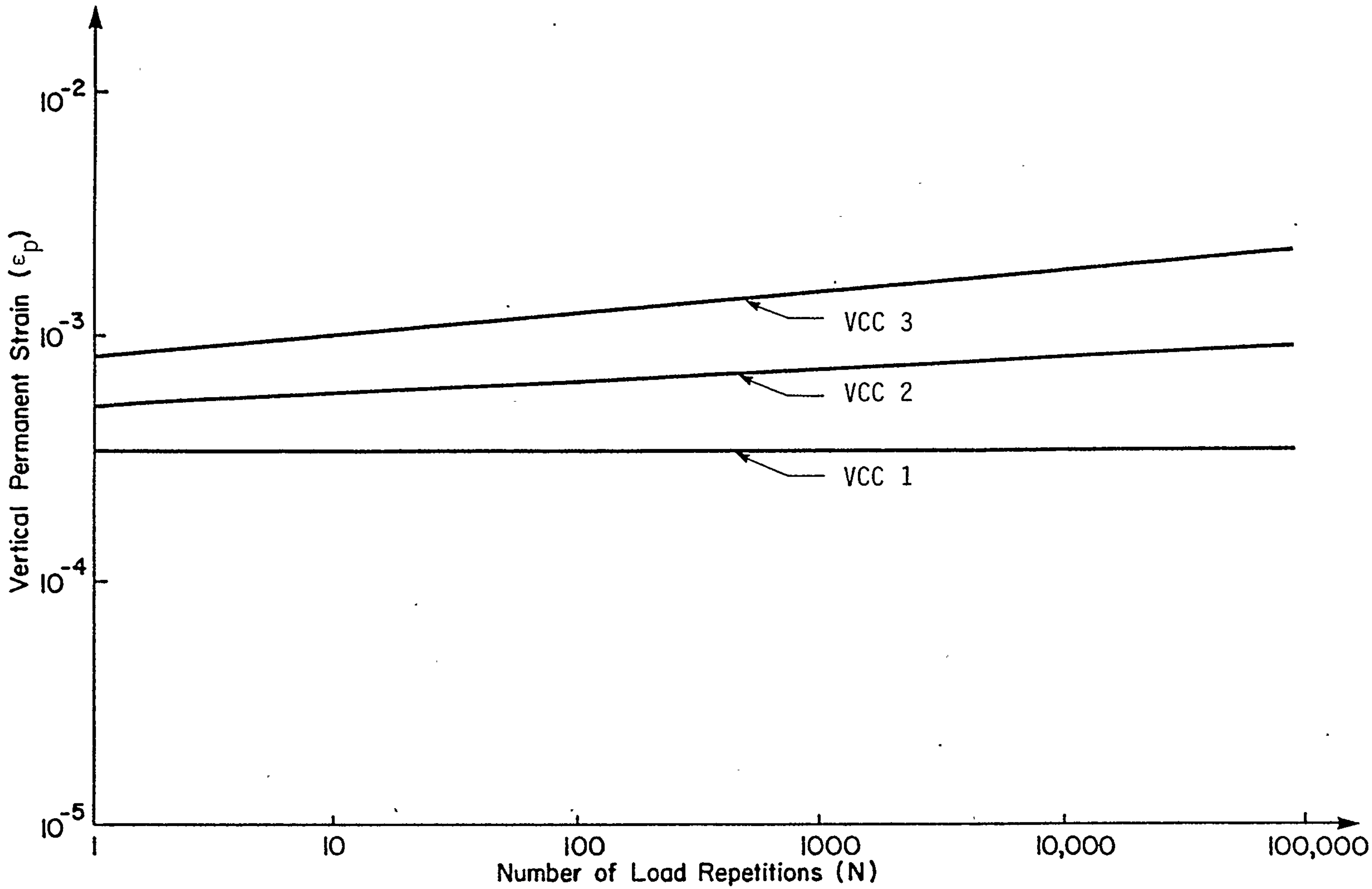

Figure A 1.2 Relationship Between Vertical Permanent Strain and the Number of Load Repetitions in Virgin Asphalt Concrete (under compressive vertical and lateral stresses)

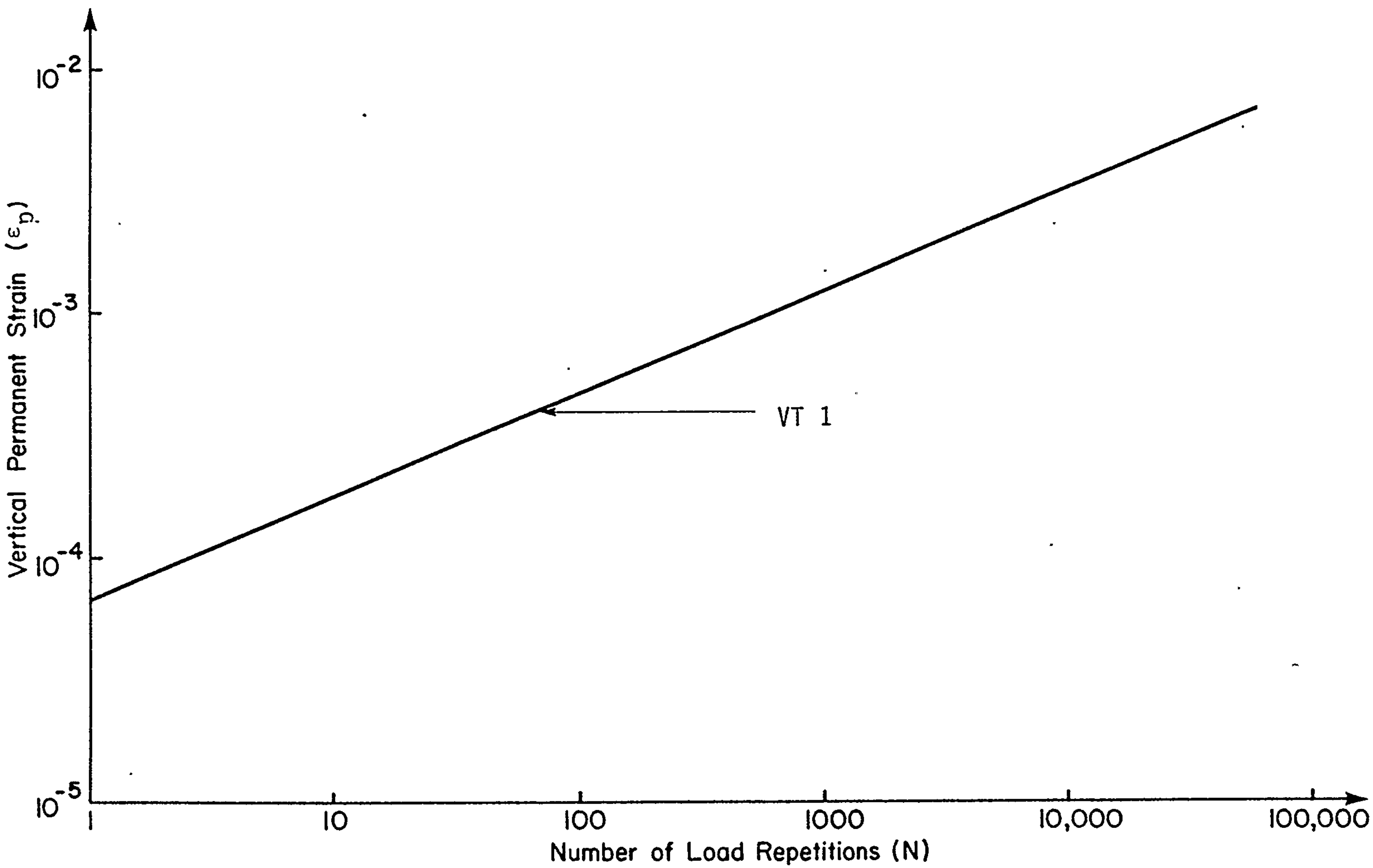

Figure A 1.3 Relationship Between Vertical Permanent Strain and the Number of Load Repetitions in Virgin Asphalt Concrete (under compressive vertical stress and tensile lateral stress)

FORMAT OF THE REGRESSION EQUATION: $\text{Log}(\varepsilon_p) = C_0 + C_1 \text{Log}(N)$

PLASTIC STRAIN VS LOAD CYCLES RUC1.VERT

FILE NONAME (CREATION DATE = 06/06/86) 06/08/86

* * * * M U L T I P L E R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = \text{Log}(\varepsilon_p)$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X = Log(N)

MULTIPLE R	0.95834
R SQUARE	0.91842
ADJUSTED R SQUARE	0.91672
STANDARD ERROR	0.07382

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE
REGRESSION	1	2.94468	2.94468
RESIDUAL	48	0.26156	0.00545

F = 540.38917 SIG F = 0.0000

---------------------------- VARIABLES IN THE EQUATION ----------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X	0.28292	0.01217	0.95834	23.246	0.0000
(CONSTANT)	-3.35729	0.05242		-64.047	0.0000

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $\text{Log}(\varepsilon_p) = C_0 + C_1 \text{Log}(N)$

PLASTIC STRAIN VS LOAD CYCLES   RUC2.VERT

FILE NONAME   (CREATION DATE = 06/08/86)   06/08/86

* * * * M U L T I P L E   R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = \text{Log}(\varepsilon_p)$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X = Log(N)

MULTIPLE R	0.96154
R SQUARE	0.92455
ADJUSTED R SQUARE	0.92291
STANDARD ERROR	0.05775

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE
REGRESSION	1	1.87968	1.87968
RESIDUAL	46	0.15340	0.00333

F = 563.67613   SIG F = 0.0000

------------------------------ VARIABLES IN THE EQUATION ------------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X	0.23003	0.00969	0.96154	23.742	0.0000
(CONSTANT)	-3.28400	0.04143		-79.273	0.0000

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $\text{Log}(\varepsilon_p) = C_0 + C_1 \text{Log}(N)$

PLASTIC STRAIN VS LOAD CYCLES RUC3.VERT

FILE NONAME (CREATION DATE = 06/08/86) 06/08/86

* * * * M U L T I P L E R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = \text{Log}(\varepsilon_p)$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. $X = \text{Log}(N)$

MULTIPLE R	0.89980
R SQUARE	0.80964
ADJUSTED R SQUARE	0.80369
STANDARD ERROR	0.10144

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE
REGRESSION	1	1.40058	1.40058
RESIDUAL	32	0.32931	0.01029

F = 136.09903 SIG F = 0.0000

---------------------------- VARIABLES IN THE EQUATION ----------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X	0.23503	0.02015	0.89980	11.666	0.0000
(CONSTANT)	-3.61705	0.08494	.	-42.582	0.0000

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $\text{Log}(\varepsilon_p) = C_0 + C_1 \text{Log}(N)$

PLASTIC STRAIN VS LOAD CYCLES   RCC1.VERT

FILE NONAME   (CREATION DATE = 06/08/86)   06/08/86

* * * * M U L T I P L E   R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = \text{Log}(\varepsilon_p)$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X = Log(N)

MULTIPLE R	0.94817
R SQUARE	0.89902
ADJUSTED R SQUARE	0.88219
STANDARD ERROR	0.01100

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE
REGRESSION	1	0.00646	0.00646
RESIDUAL	6	0.00073	0.00012

F = 53.41938   SIG F = 0.0003

---------------------------- VARIABLES IN THE EQUATION ----------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X	0.05022	0.00687	0.94817	7.309	0.0003
(CONSTANT)	-3.55944	0.02908		-122.382	0.0000

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $\text{Log}(\varepsilon_p) = C_0 + C_1 \text{Log}(N)$

PLASTIC STRAIN VS LOAD CYCLES   RCC2.VERT

FILE NONAME   (CREATION DATE = 06/08/86)   06/08/86

* * * * M U L T I P L E   R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = \text{Log}(\varepsilon_p)$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X = Log(N)

MULTIPLE R	0.97850
R SQUARE	0.95747
ADJUSTED R SQUARE	0.95497
STANDARD ERROR	0.01399

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE
REGRESSION	1	0.07488	0.07488
RESIDUAL	17	0.00333	0.00020

F = 382.72435   SIG F = 0.0000

---------------------------- VARIABLES IN THE EQUATION ----------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X	0.11175	0.00571	0.97850	19.563	0.0000
(CONSTANT)	-3.32163	0.02454		-135.379	0.0000

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $\text{Log}(\varepsilon_p) = C_0 + C_1 \text{Log}(N)$

PLASTIC STRAIN VS LOAD CYCLES   RCC3.VERT

FILE NONAME   (CREATION DATE = 06/08/86)   06/08/86

* * * * M U L T I P L E   R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = \text{Log}(\varepsilon_p)$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X = Log(N)

MULTIPLE R	0.85570	ANALYSIS OF VARIANCE			
R SQUARE	0.73221		DF	SUM OF SQUARES	MEAN SQUARE
ADJUSTED R SQUARE	0.72265	REGRESSION	1	0.21755	0.21755
STANDARD ERROR	0.05331	RESIDUAL	28	0.07956	0.00284

F = 76.56138   SIG F = 0.0000

---------------------------- VARIABLES IN THE EQUATION ----------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X	0.14681	0.01678	0.85570	8.750	0.0000
(CONSTANT)	-3.10738	0.07339		-42.338	0.0000

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $\text{Log}(\varepsilon_p) = C_0 + C_1 \text{Log}(N)$

PLASTIC STRAIN VS LOAD CYCLES RT1.VERT

FILE NONAME (CREATION DATE = 06/09/86) 06/09/86

* * * * M U L T I P L E R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = \text{Log}(\varepsilon_p)$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X = Log(N)

MULTIPLE R	0.92183
R SQUARE	0.84977
ADJUSTED R SQUARE	0.84226
STANDARD ERROR	0.19577

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE
REGRESSION	1	4.33578	4.33578
RESIDUAL	20	0.76653	0.03833

F = 113.12677 SIG F = 0.0000

---------------------------- VARIABLES IN THE EQUATION ----------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X	0.55528	0.05221	0.92183	10.636	0.0000
(CONSTANT)	-4.66694	0.19433		-24.016	0.0000

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

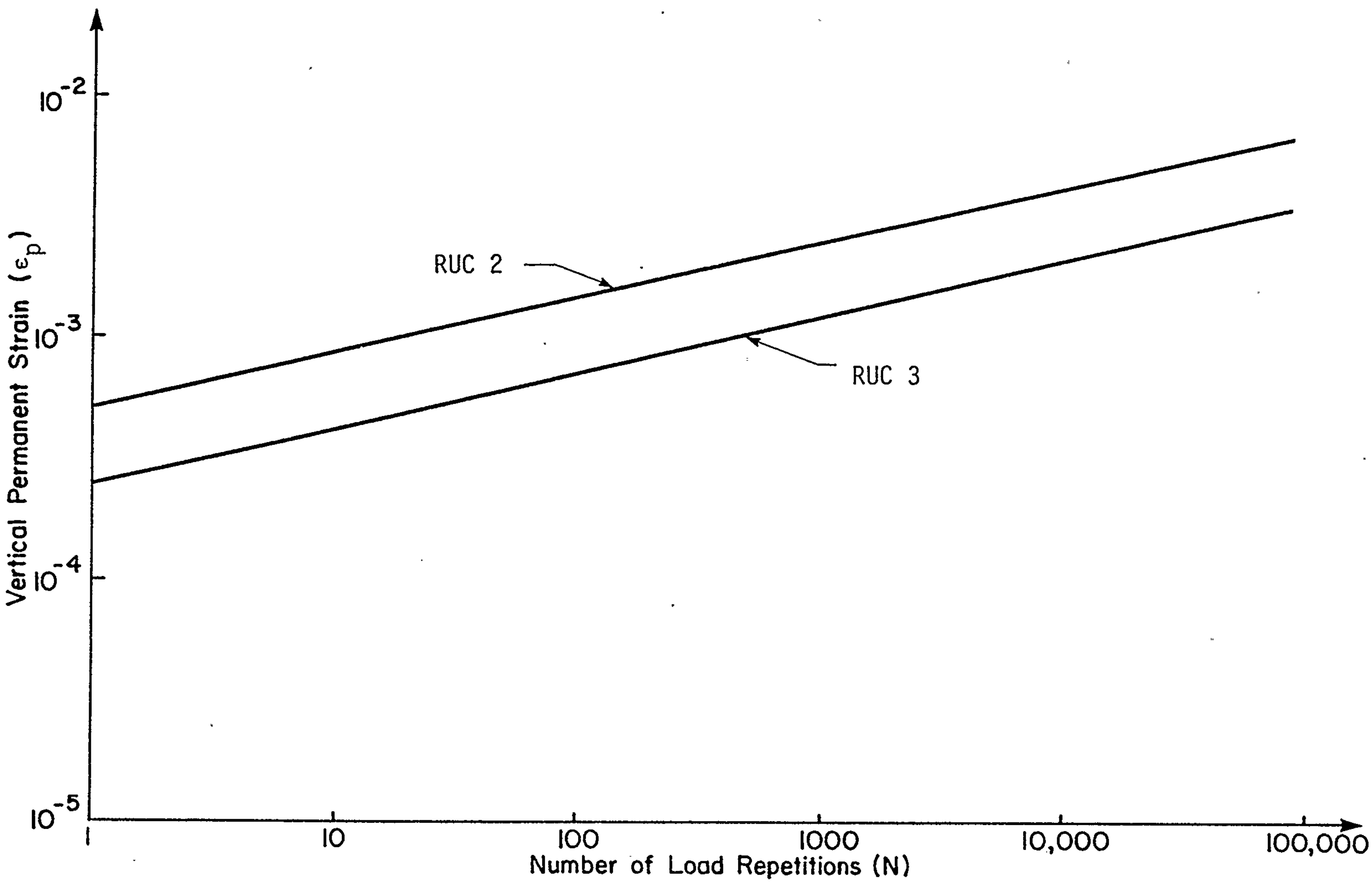

Figure A 1.4 Relationship Between Vertical Permanent Strain and the Number of Load Repetitions in Recycled Asphalt Concrete (under uni-axial stress conditions)

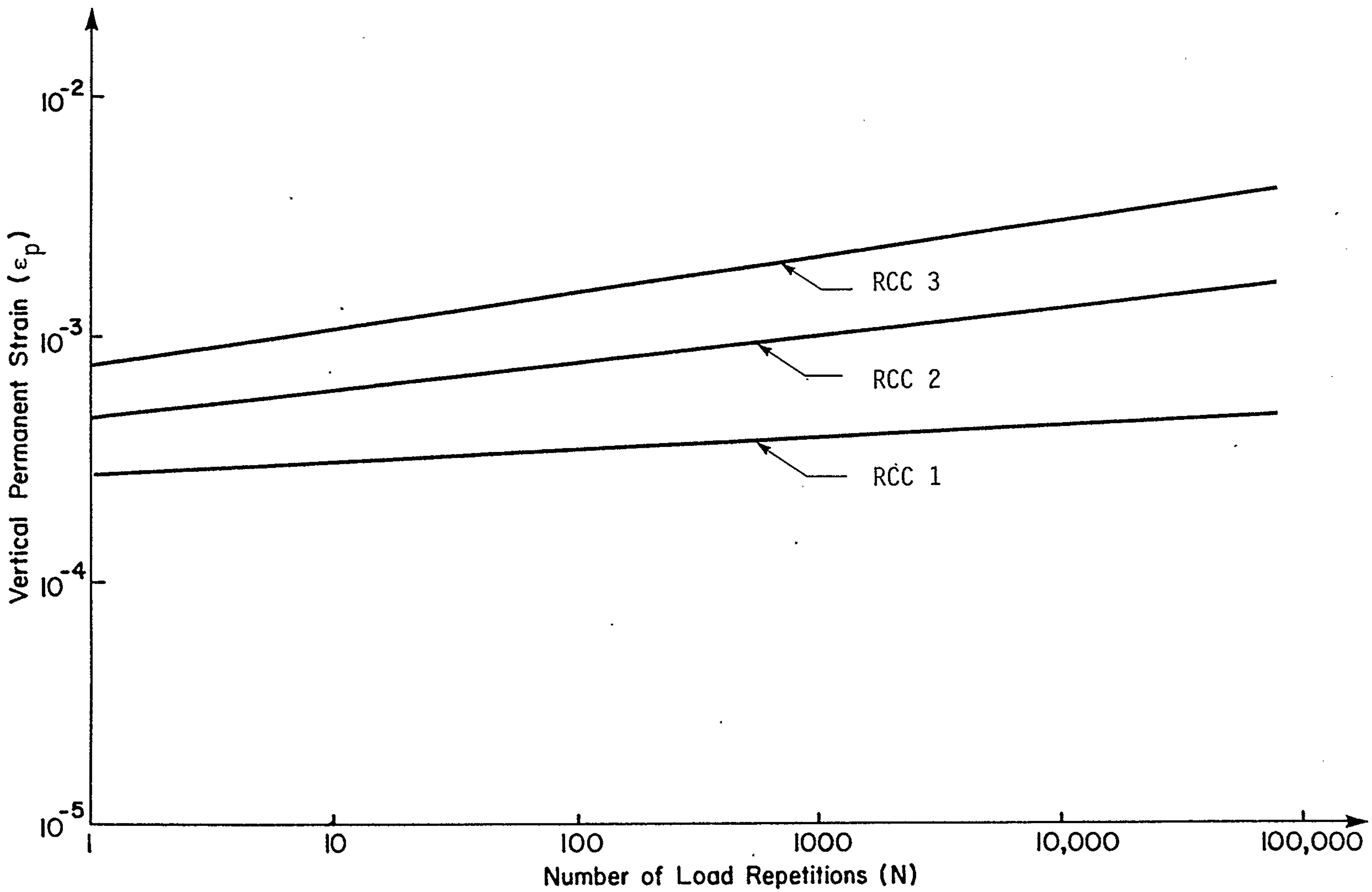

Figure A 1.5 Relationship Between Vertical Permanent Strain and the Number of Load Repetitions in Recycled Asphalt Concrete (under compressive vertical and lateral stresses)

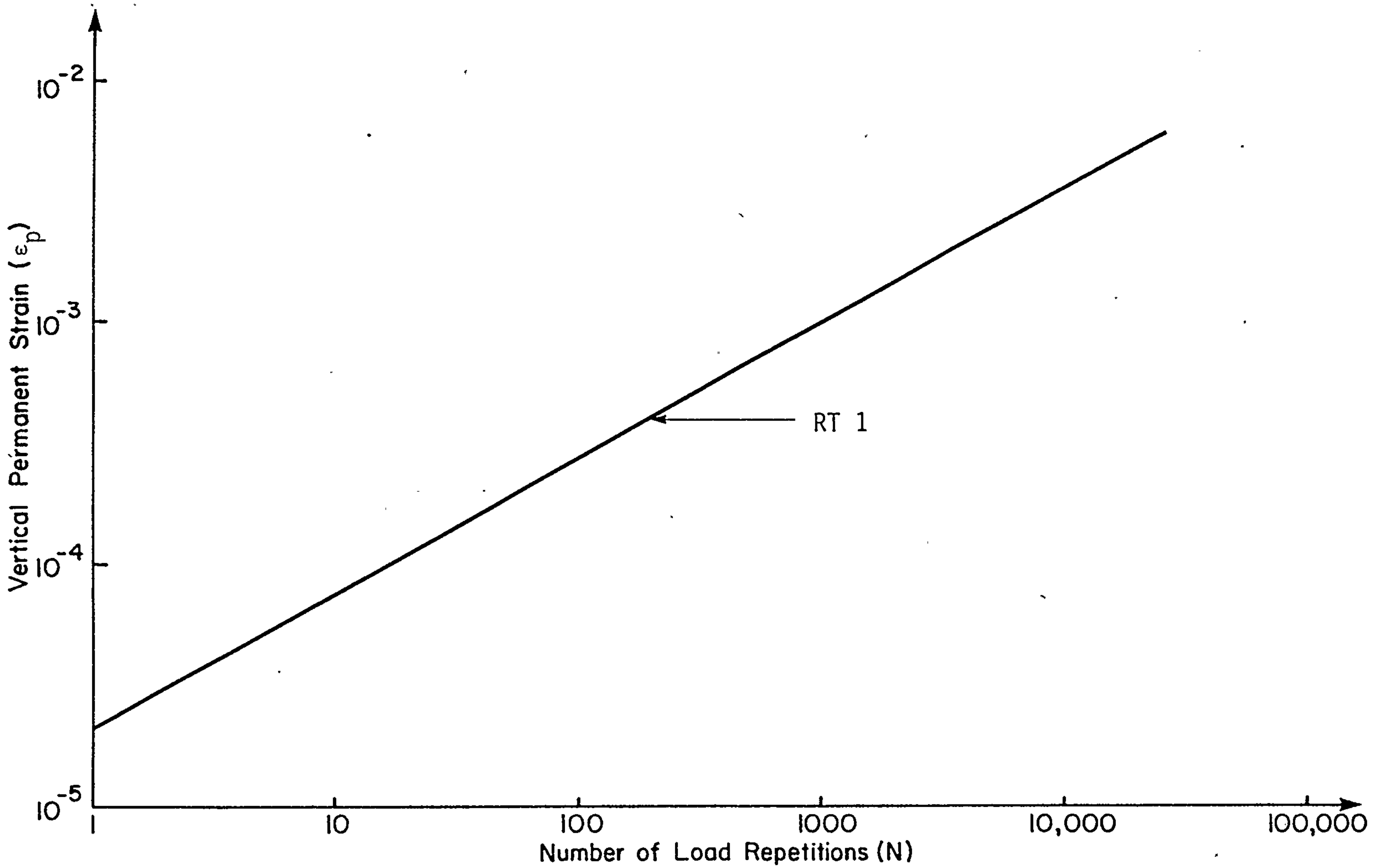

Figure A 1.6 Relationship Between Vertical Permanent Strain and the Number of Load Repetitions in Recycled Asphalt Concrete (under compressive stress and tensile lateral stress)

APPENDIX A2

REGRESSION RESULTS OF THE RELATIONSHIPS BETWEEN C_0, C_1 AND APPLIED STRESSES FOR BOTH VIRGIN AND RECYCLED ASPHALT CONCRETES

FORMAT OF THE REGRESSION EQUATION: $C_0 = a_1 + a_2(p) + a_3(q)$

VIRGIN ASPHALT PREDICTIVE RELATIONSHIP

FILE NONAME (CREATION DATE = 05/31/86)

* * * * M U L T I P L E   R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = C_0$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X1 = p
2. Z1 = q

MULTIPLE R	0.89188
R SQUARE	0.79546
ADJUSTED R SQUARE	0.69318
STANDARD ERROR	0.27794

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE
REGRESSION	2	1.20166	0.60083
RESIDUAL	4	0.30900	0.07725

F = 7.77782 SIG F = 0.0418

---------------------------- VARIABLES IN THE EQUATION ----------------------------

VARIABLE	B	SE B	BETA	T	SIG T
Z1	0.00215	0.7959E-03	0.65120	2.703	0.0539
X1	0.00259	0.7132E-03	0.87382	3.628	0.0222
(CONSTANT)	-4.45061	0.25251		-17.625	0.0001

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $C_1 = b_1 + b_2(p)$

VIRGIN ASPHALT PREDICTIVE RELATIONSHIP

FILE NONAME (CREATION DATE = 05/31/86)

* * * * M U L T I P L E   R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = C_1$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X1 = p

MULTIPLE R	0.93221	ANALYSIS OF VARIANCE			
R SQUARE	0.86901		DF	SUM OF SQUARES	MEAN SQUARE
ADJUSTED R SQUARE	0.84281	REGRESSION	1	0.17452	0.17452
STANDARD ERROR	0.07253	RESIDUAL	5	0.02631	0.00526
		F = 33.17096		SIG F = 0.0022	

------------------------------ VARIABLES IN THE EQUATION ------------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X1	-0.00101	0.1747E-03	0.93221	-5.759	0.0022
(CONSTANT)	0.40340	0.04296		9.390	0.0002

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $C_0 = a_1 + a_2(p) + a_3(q)$

RECYCLED ASPHALT PREDICTIVE RELATIONSHIP

FILE NONAME (CREATION DATE = 06/09/86)

* * * * M U L T I P L E   R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 2. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 2.

DEPENDENT VARIABLE. $Y = C_0$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X1 = p
2. Z1 = q

MULTIPLE R	0.82740
R SQUARE	0.68459
ADJUSTED R SQUARE	0.52689
STANDARD ERROR	0.35603

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE
REGRESSION	2	1.10049	0.55025
RESIDUAL	4	0.50703	0.12676

F = 4.34096 SIG F = 0.0995

---------------------------- VARIABLES IN THE EQUATION ----------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X1	0.00233	0.9136E-03	0.76260	2.549	0.0633
Z1	0.00231	0.00102	0.67761	2.265	0.0862
(CONSTANT)	-4.42063	0.32346		-13.667	0.0002

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.

FORMAT OF THE REGRESSION EQUATION: $C_1 = b_1 + b_2(p)$

RECYCLED ASPHALT PREDICTIVE RELATIONSHIP

FILE NONAME (CREATION DATE = 06/09/86)

* * * * M U L T I P L E   R E G R E S S I O N * * * *

VARIABLE LIST NUMBER 1. LISTWISE DELETION OF MISSING DATA

EQUATION NUMBER 1.

DEPENDENT VARIABLE. $Y = C_1$

BEGINNING BLOCK NUMBER 1. METHOD: ENTER

VARIABLE(S) ENTERED ON STEP NUMBER 1. X1 = p

MULTIPLE R	0.85872
R SQUARE	0.73739
ADJUSTED R SQUARE	0.68487
STANDARD ERROR	0.09220

ANALYSIS OF VARIANCE

	DF	SUM OF SQUARES	MEAN SQUARE
REGRESSION	1	0.11935	0.11935
RESIDUAL	5	0.04250	0.00850

F = 14.03981 SIG F = 0.0133

------------------------------ VARIABLES IN THE EQUATION ------------------------------

VARIABLE	B	SE B	BETA	T	SIG T
X1	-0.83223E-30	0.2221E-03	-0.85862	-3.747	0.0133
(CONSTANT)	0.38782	0.05461		7.102	0.0009

FOR BLOCK NUMBER 1 ALL REQUESTED VARIABLES ENTERED.