AN ABSTRACT OF THE THESIS OF

Ok-Kee Kim for the degree of Doctor of Philosophy in Civil Engineering

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Title: The Effect of Mixing Moisture, Oxidative Aging and Tire Pressures on the Performance of Asphalt Mixes

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Dr. Chris A. Bell

The research undertaken consisted of the study of three topics affecting the performance of asphalt concrete mixtures, that is, mixing moisture, oxidative aging and higher tire pressures and axle loads. This thesis presents the findings of the research comprising these three topics.

In the first study, the effects of mixing moisture (0,1,3%) and additives (lime, Pavebond Special) on performance of asphalt mixtures were evaluated using repeated load diametral testing of laboratory samples in terms of resilient modulus, fatigue life, and permanent deformation. To evaluate the long-term durability of mixes, the modified Lottman conditioning procedure was used. Test results show that inferior performance occurred for mixes with 3% moisture and the addition of lime resulted in distinct improvement of performance for moist mixtures.

In the second study, the effects of oxidative aging on asphalt mixtures used in the construction of three projects in Oregon were evaluated. The repeated load diametral test for mixtures and Fraass test for asphalt cements were used. A modified Pressure Oxidation Bomb

(POB) laboratory accelerated aging method with pure oxygen was adopted. As evaluation parameters, the modulus ratio and Fraass breaking temperature are good indicators of the aging rate of mixtures and asphalt cements, respectively.

The third study, the effect of increased axle loads and tire pressures of trucks on the performance of asphalt concrete pavements, included a survey of existing truck operating characteristics in Oregon and an investigation of the current mix design criteria. In particular, stability of asphalt mixtures was evaluated. Six different aggregate gradations including the Fuller maximum density gradation using aggregate from four different sources were used. The correlation analyses between creep behavior and mix design criteria of asphalt mixtures were made. The results of the survey showed that 87% of the tires were of radial construction, and the average measured tire pressures of radial and bias are 102 and 82 psi, respectively. Theoretical equivalency factors taking into account the effect of tire pressures were developed. A 25% increase in tire pressure could result in a 40 to 60% increase in equivalency for a dual tired single axle of 18 kips and a tandem axle of 34 kips.

THE EFFECT OF MIXING MOISTURE, OXIDATIVE AGING AND TIRE PRESSURES ON THE PERFORMANCE OF ASPHALT MIXES

by

Ok-Kee Kim

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Redacted for privacy

Head of Department of Civil Engineering

Redacted for privacy

Dean of Graduate School

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Typed by author for Ok-Kee Kim.

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"A man's mind plans his way, but the Lord directs his steps" (Proverbs 16:9).

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PREFACE

This dissertation is a compilation of five articles written for separate publication. These articles are derived from two HP & R (Highway Planning and Research) studies entitled, "Effect of Moisture and Aging on Asphalt Pavement Life", Part 1 - Effect of Moisture, and, Part 2 - Effect of Aging, and "Procedures for Controlling the Effect of Increased Tire Pressure on Asphalt Concrete Pavement Damage." These studies were conducted by the Oregon State Highway Division and Oregon State University in cooperation with the Federal Highway Administration. These projects were performed independently, and the five resulting papers are presented in chapters 2 through 6. Chapters 4, 5, and 6 present the papers written for the tire pressure study, and some repetition may be noted. Citations in the text refer to references listed at the end of each chapter. These references are collected into a comprehensive bibliography at the end of the text.

In chapter 2, Dr. R.G. Hicks is listed as a co-author because he was a project leader of the moisture effect study. Dr. Bell's most important contributions were as my guide, idea man, and editor. Also, Dr. Bell was a project leader of the studies on aging and higher tire pressure. Messrs. Wilson and Boyle provided mix design, laboratory samples and core samples.

Finally, it should be noted that the interactive effects of moisture, aging, and higher tire pressure are not considered within the scope of this dissertation.

Ok-Kee Kim

November, 1987

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1.0 INTRODUCTION

1.1 PROBLEM STATEMENT

The performance of asphalt pavements is affected by several factors as illustrated in Figure 1.1. These factors include mix design based on material properties, environmental conditions, traffic characteristics, and pavement structure. However, several changes have occurred over the last two decades in asphalt pavement materials and asphalt paving technology, including construction practices and equipment developments as presented in Table 1.1.

New asphalt sources have been brought on line, introducing changes in asphalt properties. The properties, and hence behavior, of asphalts from different suppliers as well as asphalts from the same supplier over a period of time are likely to vary (1, 2, 3). Not only economic constraints but also environmental and land use restrictions have resulted in increasing use of lower quality aggregate.

Equipment changes include the introduction of drum mix plants. Moisture contents are likely to be higher in mixes produced from drum mix plants than in mixes prepared in batch plants. Lower mixing temperatures to reduce energy costs in either type of plant can have a significant effect on the viscosity of the asphalt cement in the mix during laydown.

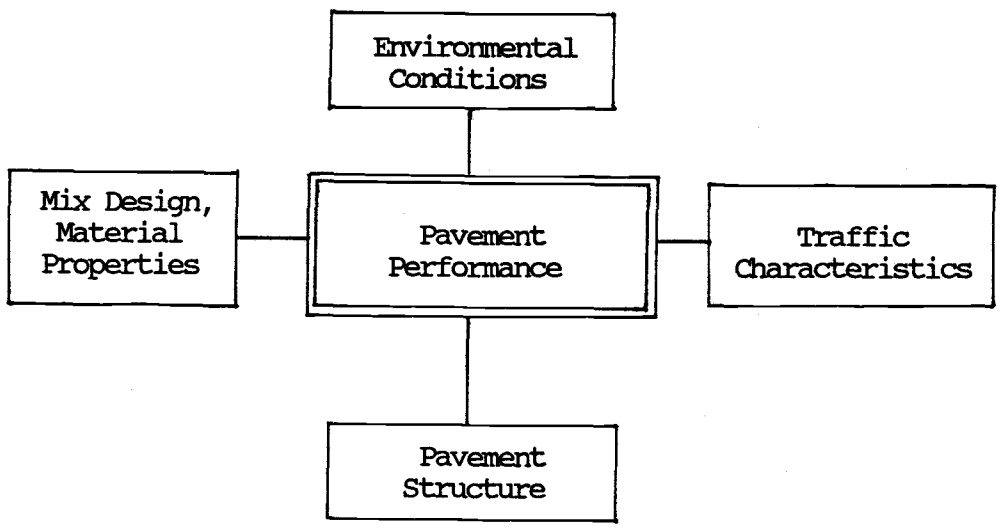

Figure 1.1 Factors Affecting the Performance of Asphalt Pavements.

Table 1.1 Major Changes since 1970 (After Ref. 1).

A. MATERIALS

1. Rapid escalation of asphalt cost caused by 1973-74 Arabian oil embargo
2. Variable crude oil supplies
3. Switch from penetration grading to viscosity grading of asphalts
4. Lack of uniformity in paving asphalt specifications
5. Increase use of modifiers and additives
6. Increase use of aggregate of marginal quality

B. CONSTRUCTION

1. Reduced highway funds available for construction
2. Loss of experienced personnel
3. Introduction of drum mix plants
4. Higher moisture content in mixes
5. Lower mix temperatures
6. Use of baghouse fines in mixes
7. Use of hot mix storage silos
8. Replacement of pneumatic compactors with vibratory compactors
9. Emphasis on thin lift construction
10. Pavement recycling

C. DESIGN

1. Wider acceptance of elastic-layered thickness design procedures
2. Higher truck axle loads and tire pressures
3. Shift from agency design to contractor design of mixes
4. Trend toward leaner mix design

A major change in construction practice since 1970 was the wide spread replacement of pneumatic compactors with vibratory compactors. This represents a significant deviation from the compaction process used prior to 1970. Economic considerations and the shift from new construction to rehabilitation have led to a great deal of thin construction.

Meanwhile, the traffic characteristics on highways including vehicle size, traffic volume, speed, axle loads and tire pressure have changed significantly.

The motor carrier industry requested Congress to increase the gross weight limitation for trucks from 73,280 to 80,000 pounds, in order to alleviate the impact caused by the dramatic increases in fuel costs resulting from the 1973 oil embargo (4). In 1982, the federal government permitted 80,000 pounds gross vehicle weight and 34,000 pounds tandem axle weights on Interstate highways. This allowed a potential 12,000 pound load on the steering axle. As the axle load limitation increases, the use of higher tire pressures becomes more popular in the truck market. The increased axle loads and tire pressures, no doubt, deteriorate the asphalt pavements more rapidly than prior levels.

These changes described above may require modifications and/or additions to existing specifications, mix design criteria (the Marshall or Hveem method), and test procedures to evaluate mix properties or new methodology to replace existing procedures to simulate service conditions.

The studies presented in this thesis include investigations into the effects of mixing moisture, oxidative aging and increased axle loads and tire pressures on the asphalt pavements and the improvement of test methods to evaluate the properties of asphalt mixes. The results of the

studies will contribute to better understanding of premature failure or poor performance of asphalt pavements constructed by using current practices.

1.2 PURPOSE

The purpose of the studies presented in this thesis is to obtain a better understanding of the causes of the pavement problems associated with mixing moisture, oxidative aging, and higher axle loads and tire pressures.

Since each study has been performed independently, the interactive effects of moisture, aging and higher tire pressures are not within the scope of this thesis.

Specific objectives of each study are:

1. To evaluate the effect of mixing moisture on mechanical properties of asphalt mixtures, in terms of resilient modulus, fatigue life, and permanent deformation.
2. To evaluate the effect of additives for reducing the damage from moisture.
3. To provide guidelines to minimize the effects of moisture on pavement performance.
4. To evaluate the effect of accelerated aging in the laboratory on mechanical properties of asphalt mixtures including resilient modulus, fatigue life, and Fraass breaking temperature of asphalt cements.
5. To provide guidelines to minimize the effects of aging on pavement performance.
6. To determine existing operating characteristics of Oregon's trucks, including levels of tire pressures.

7. To develop the use of a simple method of creep testing to predict deformation in asphalt pavements.
8. To evaluate the effectiveness of existing asphalt concrete mixture methods in limiting excessive deformation caused by higher axle loads and tire pressures, and
9. To develop theoretical equivalency factors for typical Oregon pavements taking into account tire pressures.

1.3 ORGANIZATION OF THESIS

The thesis consists of three major parts. Following this chapter, the next five chapters are reproductions of papers prepared by the author in cooperation with several colleagues.

The first paper, "The Effect of Moisture on the Performance of Asphalt Mixtures" was published by the American Society for Testing and Materials (ASTM) following a symposium in December 1984 and is presented in Chapter 2. The second paper, "Development of Laboratory Oxidative Aging Procedures for Asphalt Cements and Asphalt Mixtures," was presented by the author at the Annual Meeting of the Transportation Research Board (TRB) in January 1987 and will be published by TRB in a 1987 Transportation Research Record. This paper is presented in Chapter 3. The papers titled, "Measurement and Analysis of Truck Tire Pressures in Oregon," and "Study on Mix Design Criteria for Controlling the Effect of Increased Tire Pressure on Asphalt Pavement" have been submitted to the Transportation Research Board for publication in 1988 and are presented in Chapter 4 and 5, respectively. The final paper, "Effect of Increased Truck Tire Pressure on Asphalt Concrete Pavement" will be submitted to American Society of Civil Engineers (ASCE) for publication

in Journal of Transportation Engineering and is presented in Chapter 6.

Chapter 7 presents the summary of these studies and common conclusions and recommendations from these studies.

In addition to the articles presented in the text of the thesis, the author was principal author of a paper "Effect of Mix Conditioning on Properties of Asphaltic Mixtures" published by TRB in 1985. It's topic relates closely to testing procedures used in these studies. This article was based solely on work done by the author as a student in the Master program and is therefore included only as an Appendix.

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2.0 THE EFFECT OF MOISTURE ON THE PERFORMANCE OF ASPHALT MIXTURES

by

Ok-Kee Kim

C.A. Bell

and

R.G. Hicks

Department of Civil Engineering
Oregon State University
Corvallis, OR 97331

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ABSTRACT

This paper presents the results of a laboratory study to investigate the effects of mixing moisture on mechanical properties of asphalt mixtures. The potential benefits of lime and Pavebond Special were also evaluated. The repeated load diametral test device was used to measure the mixture performance in terms of the resilient modulus, fatigue, and permanent deformation characteristics of laboratory specimens prepared with and without moisture (0, 1, and 3%) and with and without lime (1%) and Pavebond (0.5%). Mixtures were prepared that were representative of two projects for which considerable field data were available. One project utilized low quality and high absorptive aggregate and the other good quality aggregate. To evaluate the long-term durability of mixtures, they were tested before and after conditioning using the Lottman approach.

The test results showed that inferior performance occurred for mixtures with 3% moisture but was most pronounced in mixtures with higher air-void contents. However, the mixtures with low quality and high absorptive aggregate showed improved performance at 1% moisture content, associated with their lower air-void contents, which may be due to absorbed moisture preventing asphalt absorption and the higher asphalt content of these mixtures. The addition of lime resulted in distinct improvement of performance for moist specimens from the project, which had good quality aggregate, but high air-void contents. However, neither additive showed substantial benefit for moist samples from the project with low quality aggregate and low air-void content.

2.1 INTRODUCTION

2.1.1 Background

The Highway Division of the Oregon Department of Transportation and Oregon State University have collaborated for six years in evaluating the effects of mix variations on asphalt pavement life (1-3). To date, the results have led to improved specifications, but although mix moisture is now limited at or below 0.7% at the plant, this value is not based on rational research findings. Furthermore, no firm guide lines are available for selection of additives that may reduce the subsequent damage to mixtures containing moisture as a result of mixing with damp aggregate, a phenomenon primarily associated with the increased use of drum mixers (2,3).

The increased presence of residual moisture in asphaltic concrete mixtures caused by changes of materials and equipment is well known and has been the subject of considerable discussion recently. In the conference held during the Highway Research Board Annual Meeting in January 1974 (4), there were many questions and considerable discussion about residual moisture. Among them were the following:

1. Are the moisture controls set at too low a level?
2. Is percent moisture a good measure of moisture effect?
3. Is the basic problem moisture or workability?
4. In the present situation, can we continue to say the drier the better?
5. If not, how can we go and what should we control?

This study addresses some of the above questions and also considers the effect of two widely used additives. Mixtures were prepared that were representative of two projects about five years old in the State of

Oregon for which considerable field data were available.

2.1.2 Purpose

The purpose of this study reported in this paper was to obtain a better understanding of the causes of the pavement problems associated with moisture, to develop relationships between pavement performance and the mixing moisture contents considering possible use of additives, and to develop mix design procedures that consider the moisture content and aggregate quality of asphalt mixtures. Such information could be useful in providing for limiting the moisture content of fresh mixtures and for selection of additives that may reduce the subsequent damage to mixtures containing moisture as a result of mixing with damp aggregates.

The specific objectives of this study were

(1) to evaluate the effect of moisture on mechanical properties of asphalt mixtures, such as resilient modulus, fatigue life, and permanent deformation,

(2) to evaluate the effect of additives for reducing the damage from moisture, and

(3) to provide guidelines to minimize the effect of moisture on pavement performance.

2.1.3 Research Approach

The research included tests on laboratory-prepared specimens representing two projects, North Oakland-Sutherlin and Warren-Scappoose. Following the standard Oregon Department of Transportation procedure (5), specimens 10.2 cm (4.0 in.) in diameter by 6.3 cm (2.5 in.) high were fabricated. Some modifications to this procedure were developed to

incorporate moisture in some of the specimens. All specimens were tested in the diametral mode in accordance with ASTM Method for Indirect Tension Test for Resilient Modulus of Bituminous Mixtures (D4123) for resilient modulus, fatigue life, and permanent deformation to evaluate the effects of different moisture contents (0, 1, and 3%) and additives (lime, Pavebond Special). Both as-compacted and conditioned asphalt mixtures were tested. Conditioned mixtures used the Lottman procedure (6), with subsequent testing providing a good indication of long-term durability.

2.2 EXPERIMENT DESIGN

2.2.1 Projects Evaluated

2.2.1.1 North Oakland-Sutherlin: This project is a section of Interstate 5 located approximately 19 km (12 miles) north of Roseburg, OR. Its overall length is 5.1 km (3.2 miles). The optimum asphalt content from the original mix design (1978) was 6.9% of an AR 8000 asphalt cement treated with 0.85% "Pavebond Special." The mix design was repeated for the laboratory tests, and because of differences in asphalt and aggregate, five years after construction, a different optimum asphalt content resulted as will be described below. The asphalt concrete base on this project was paved in October through December 1978, and showed problems of raveling and potholing shortly thereafter. An investigation performed by the Oregon Department of Transportation (ODOT) suggested that the reduced quality of the paving was basically the result of using varying amounts of low-quality aggregate in the mix (2). The aggregate used in this project was a crushed submarine basalt containing seams of sulfate compounds of

calcium, sodium, and magnesium. This aggregate showed high water absorption (up to 7% by weight). Soundness test results for produced aggregate used in the paving ranged from 4 to 39% loss for coarse aggregate and 11 to 48% loss for fine aggreggate. Oregon currently (1984) limits the soundness test loss to 18%. Unfortunately, there were no data concerning the moisture content of cores. However, there was also some deviance from the specified aggregate gradation limits as shown in Figure 2.1.

2.2.1.2 Warren-Scappoose: This project is a section of the Columbia River Highway, located in Columbia county, OR. The overall length is 8 km (5 miles). The base course was constructed in 1979 and the wearing surface in 1980. The optimum asphalt content from the original mix design (1979) was 5.1% for the wearing surface and 5.7% for the base course, which used a good quality gravel aggregate. Again, the mix design was repeated for the laboratory tests, and because of differences in asphalt and aggregate, five years after construction, a different optimum asphalt content resulted as will be described below. The asphalt grade recommended was an AR 4000. Progressive pavement raveling and potholing were noticed in the base course during the months following construction (3). The core data obtained for this project suggested that the reduction in pavement life resulted from high air-void content (10 to 15% for the wearing course and 6 to 11% for the base course) and variability in aggregate gradation as shown in Figure 2.2. Also, the core data showed that the water content ranged from 0.64 to 1.0%, and poor adhesion or stripping was observed. Poor adhesion or stripping might have resulted from this water content.

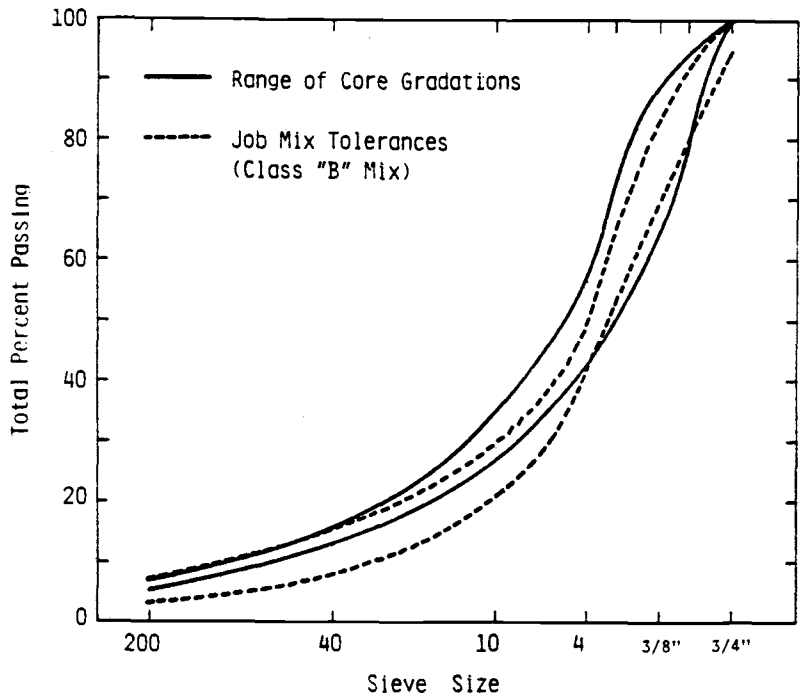


Figure 2.1 Core Gradation for Base Layers: North Oakland-Sutherlin.

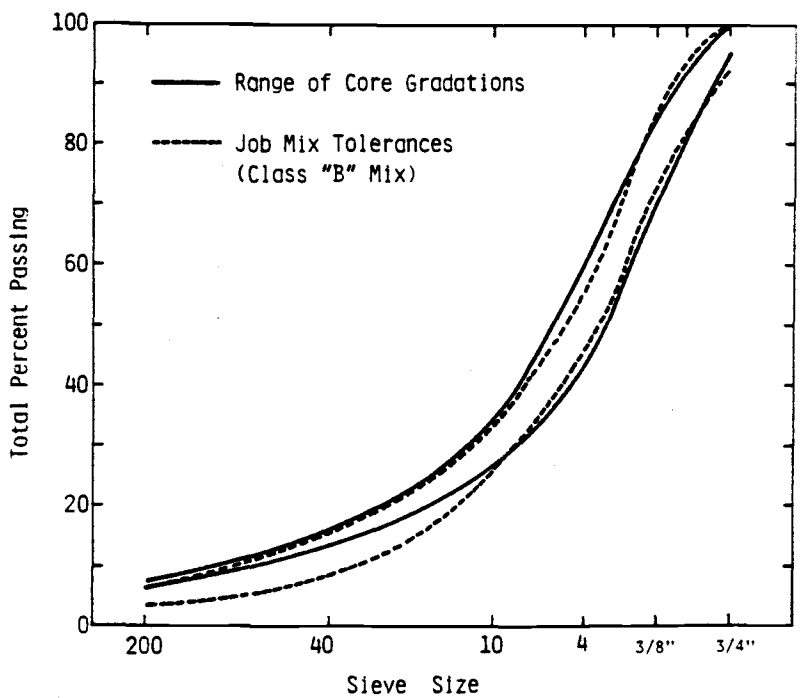


Figure 2.2 Core Gradation for Base Layers: Warren-Scappoose.

2.2.2 Test Program and Methods

The variables considered in this study are presented in Table 2.1. All specimens were prepared to a target density of 92% of theoretical maximum, with the asphalt content and aggregate gradation established in the mix designs. The mix designs were repeated, since the aggregates and asphalts available, five years after the projects were constructed, were different to those used in the projects. The mix designs and specimen preparation for the laboratory study are outlined in subsequent sections.

A minimum of 12 specimens were prepared for each project for each one of the mix variables presented in Table 2.1 (a total of 168 specimens). Six specimens were tested as compacted, and six were tested after conditioning. All specimens were tested for resilient modulus, fatigue, and permanent deformation. All tests were run at three different initial tensile strain levels ranging from 50 to 200 microstrain, resulting in two samples being used at each strain level.

The sample conditioning procedure was based on the moisture-induced damage test defined by Lottman (6). In summary,

1. vacuum (66-cm [26-in.] mercury) saturate the specimens for 2 h,
2. place the saturated specimens in a freezer at -18°C (0°F) for 15h,
3. place the frozen, saturated specimens in a warm water bath (60°C [140°F]) for 24 h,
4. place the specimens in a water bath at room temperature for 3 h,
5. dry the specimen at room temperature for 2 h, and
6. run the diametral test.

The resilient modulus, fatigue, and permanent deformation tests

Table 2.1 Range of Mix Variables Considered in This Study, X's.

Conditioning:	As-Compacted			Conditioned		
Antistrip Agent:	None	1% Lime	0.5% Pavebond	None	1% Lime	0.5% Pavebond
Moisture:						
Standard (none)	X	X	X	X	X	X
1% Moisture	X			X		
3% Moisture	X	X	X	X	X	X

were performed using the repeated load diametral test apparatus. The test procedures employed are essentially the same as used in previous studies (2,3). During the tests, the dynamic load duration was fixed at 0.1 s and the load frequency at 60 cpm. A static load of 4.5 kg (10 lb) was applied to hold the specimen in place. The tests were carried out at 22.5 ± 1.5°C (72.5 ± 2.7°F) and at 19.8 ± 1.5°C (67.6 ± 2.7°F) for the Warren-Scappoose and the North Oakland-Sutherlin projects, respectively.

The maximum load applied and the horizontal elastic deformation were recorded to determine the resilient modulus. The resilient modulus M_R, initial horizontal elastic tensile strain ϵ_t, and vetrtical permanent strain ϵ_c were calculated from the equations suggested by Kennedy (7) using the specimen diameter of 10.2 cm (4.0 in.) and assumed Poisson's ratio of 0.35. Fatigue life is characterized by the number of load applications required to cause failure of the specimen. Attempts to relate the number of load applications to the specimen state of stress and strain showed that the best correlation exists between the tensile strain and the number of load applications, according to the following model (8,9)

$$N_f = K(1/\epsilon_t)^m \qquad [1]$$

where

N_f = number of load repetitions to failure,

K, m = regression constants, and

ϵ_t = initial horizontal elastic tensile strain.

The number of load repetitions to fatigue failure was defined as the number of repetitions required to cause a vertical crack approximately 0.65 cm (1/4 in.) wide in the specimens. To stop the test at

specified level of specimen deformation, a thin aluminum strip was attached to the sides of the specimens, along a plane perpendicular to the plane formed by the load platens. When the specimen deformation exceeded a certain level, the aluminum strip broke and opened a relay, which shut off the test. Proper calibration of the length of the aluminum strip cause the test to stop for a specific specimen crack width of 0.65 cm (1/4 in.).

For each test the accumulation of vertical permanent strain was also monitored during the fatigue life test using the controlled-load test method described by Monismith [10]. The relationship between vertical permanent strain and number of load repetitions may be expressed as follows

$$N = I(1/\epsilon_c)^s \qquad [2]$$

where

ϵ_c = compressive permanent vertical strain,

I, s = constants determined by regression analysis, and

N = number of load repetitions.

2.2.3 Mix Design

Standard mix designs for the laboratory study were carried out by Oregon State Highway Division Materials Section. The mix design method is a version of the Hveem approach (ASTM Test Method for Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus [D 1560]), modified for Oregon conditions. Because of the availability of materials five years after construction, the mix designs for the laboratory study of moisture effects were different from the original mix designs for two projects described previously. As a result

the asphalt contents used were 5.0 and 6.2% for the Warren- Scappoose and North Oakland-Sutherlin projects, respectively.

An Oregon Type B aggregate mix gradation (19-mm [3/4-in.] nominal size) was used for this study (Table 2.2), the same as used in both projects (2,3). The quality of the aggregates was similar to that used in the projects, viz, low soundness and high absorption characterized the North Oakland-Sutherlin project, and good quality gravel aggregate was used for the Warren-Scappoose project.

2.2.4 Specimen Preparation

The desired moisture contents for the asphalt concrete used in this study were from 0% moisture to a value of 3%, which has been demonstrated to be detrimental to asphalt concrete pavement. Hence, asphalt concrete specimens were prepared at 0, 1, and 3% moisture content, and with antistrip agents as indicated in Table 2.1. The asphalt concrete moist specimen preparation procedure was based on trial and error tests performed by the Oregon State Highway Materials Section. The following are the main steps:

1. Prepare specimen aggregate (1100 g).
2. Obtain the dry weight.
3. Soak aggregate in water overnight.
4. Heat the wet aggregate until the desired initial moisture content is obtained.
5. Add the asphalt cement required.
6. Mix and compact the specimen following ASTM Method for Preparation of Bituminous Mixture Test Specimens by Means of California Kneading Compactor (D 1561) procedure.

Table 2.2 Mix Design for Laboratory Prepared Specimens:

Aggregate Gradation, Class B.

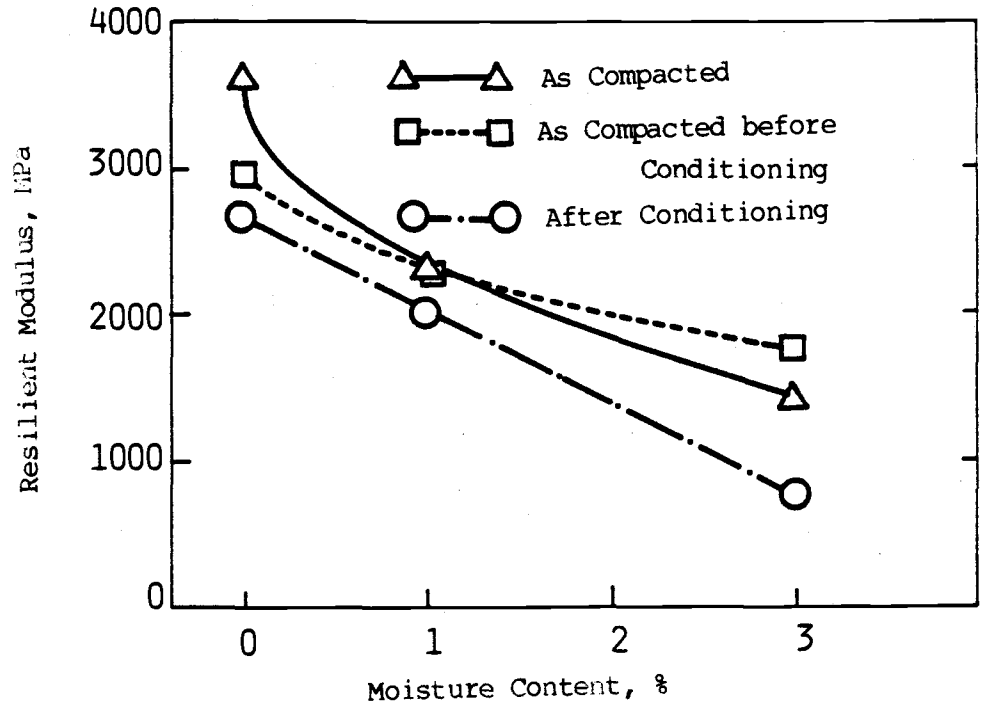
Sieve Size	Recommended Aggregate Gradation, %
1"	100
3/4"	98
1/2"	87
3/8"	79
1/4"	65
#10	33
#40	14
#200	5.0

2.3 RESULTS AND DISCUSSION

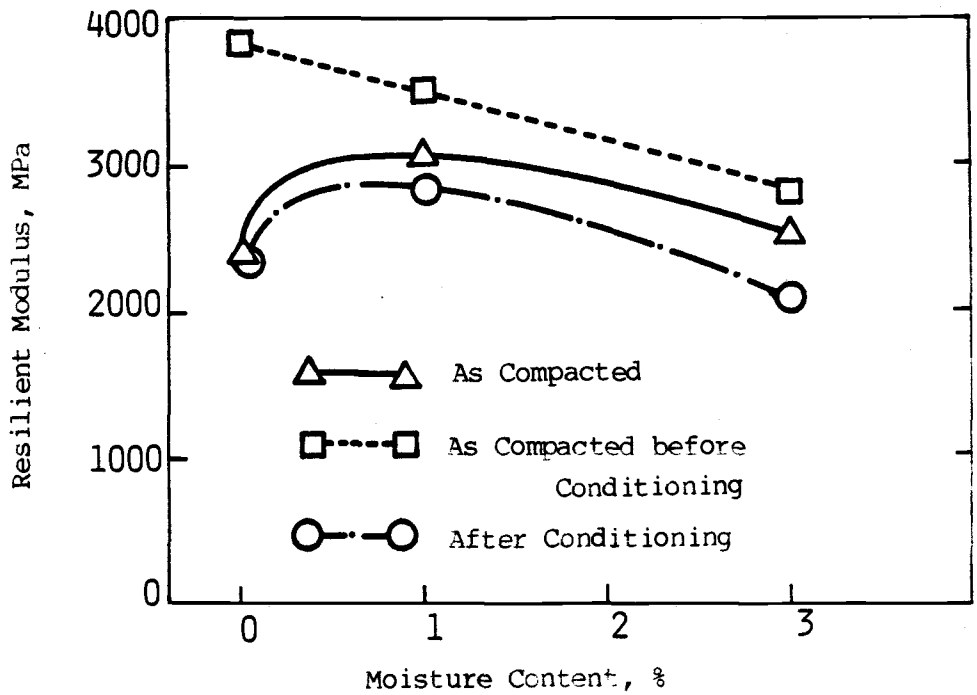
The results are presented and discussed below in two major parts: the effects of moisture on specimens with no additives and the effects of additives on specimens with and without moisture. It should be noted that the moisture content refers to the amount incorporated during mixing, whereas "conditioning" refers to a test procedure (6) to evaluate durability of mixtures. In evaluating the results, the effect of moisture in "as-compacted" specimens (those involving no conditioning) is of much less significance than in "conditioned" specimens. The effect of conditioning is assessed by comparing properties measured with "as-compacted" and "conditioned specimens" such as retained modulus value.

2.3.1 Effect of Moisture for Mixtures without Additives

2.3.1.1 Modulus Results: Six specimens were tested as compacted and six specimens were tested both before and after conditioning for each project for each mix variable set presented in Table 2.1. Because there was a difference of about seven weeks between tests with as-ccompacted specimens and the conditioning tests, two moduli values of as-compacted specimens per each mix variable set were obtained. The effect of moisture on modulus for both projects is shown in Figure 2.3. For the Warren-Scappoose project, modulus decreases with increasing moisture content. The average modulus of as-compacted specimens with 3% moisture content is 40% of the average modulus of as-compacted specimens with no moisture. Also, the retained modulus ratio (modulus after conditioning /modulus before conditioning) of specimens with 3% moisture content is only 0.43.



(a) Warren-Scappoose



(b) North Oakland-Sutherlin

Figure 2.3 Effect of Moisture on Resilient Modulus.

For the North Oakland-Sutherlin project, Figure 2.3 shows that the average modulus of the as-compacted and conditioned specimens increases as the moisture content increases up to 1% and then decreases with increasing moisture. This behavior is due in part to the higher air voids (8.34%) for specimens without moisture compared to low air voids (5.72%) for specimens with 1% moisture, as shown in Table 2.3.

For the North Oakland-Sutherlin project, which used low quality crushed rock aggregate, the average retained modulus ratio of specimens with no moisture (0.62) is lower than that for the Warren-Scappoose project (0.90), which used good quality gravel aggregate. Similar results were obtained in the previous study (2,3). However, with 3% moisture content, the retained modulus ratio for the North Oakland-Sutherlin project (0.74) is about double that for the Warren-Scappoose project (0.42). This trend is attributed to different asphalt cement contents (11), resulting from the mix designs, (5.0% for the Warren-Scappoose project and 6.2% for the North Oakland-Sutherlin project), as well as the large difference of air voids shown in Table 2.3.
The large difference in asphalt contents is due to the substantially different aggregates used in the projects, with the gravel aggregate in the Warren-Scappoose project requiring a much lower asphalt content.

The above results may be explained by considering the effects of asphalt quality and aggregate quality on mixture resilient modulus. High stiffness (resilient modulus) is achieved by a high aggregate density, with pronounced inter-particle friction and interlock.
Aggregate density increases to a maximum (dependent on gradation) as the fluid content in a mixture increases until a maximum is reached, after which further addition of fluids (moisture and asphalt cement) cause a

Table 2.3 Specific Gravity and Air Voids of Laboratory Specimens.

(a) Warren-Scappoose

	Max. Sp. Gr.	Sp. Gr.	Air Voids (%)	Water Intrusion* (%, by weight)
0% Moisture	2.487	2.284	8.16	-
1% Lime	2.492	2.309	7.34	1.77
0.5% Pavebond	2.496	2.293	8.13	1.78
1% Moisture	2.507	2.314	7.69	1.81
3% Moisture	2.505	2.287	8.70	2.08
3% Moisture/ 1% Lime	2.493	2.315	7.14	1.19
3% Moisture/ 0.5% Pavebond	2.508	2.256	10.05	1.82

$$\text{*Water Intrusion} = \frac{\text{wt. after vacuum saturation} - \text{wt. before vacuum saturation}}{\text{wt. before vacuum saturation}} \times 100$$

Table 2.3 Specific Gravity and Air Voids of Laboratory Specimens (Continued).

(b) North Oakland-Sutherlin

	Max. Sp. Gr.	Sp. Gr.	Air Voids (%)	Water Intrusion* (%, by weight)
0% Moisture	2.493	2.285	8.34	1.60
1% Lime	2.485	2.305	7.24	0.74
0.5% Pavebond	2.503	2.335	6.71	0.47
1% Moisture	2.501	2.358	5.72	0.42
3% Moisture	2.494	2.415	3.17	0.20
3% Moisture/ 1% Lime	2.478	2.388	3.63	0.47
3% Moisture/ 0.5% Pavebond	2.486	2.430	2.25	0.17

$$\text{*Water Intrusion} = \frac{\text{wt. after vacuum saturation} - \text{wt. before vacuum saturation}}{\text{wt. before vacuum saturation}} \times 100$$

reduction in density. Such behavior is similar to that for soil and water combinations where maximum density occurs at an optimum water content. Maximum aggregate density may not correspond to maximum stiffness since asphalt contributes to mixture stiffness adversely once an aggregate density is reached that will mobilize interparticle friction and aggregate interlock. Addition of further asphalt will reduce aggregate interlock and interparticle friction. The presence of moisture at the time of mixing will contribute to additional fluids in the mixture and will therefore reduce the compactive effort necessary to achieve maximum aggregate density. For absorptive aggregates, free moisture in the aggregate leaves more asphalt available for coating the aggregate (12). However, the bond between an asphlat cement and aggregate will usually be adversely affected by moisture, the severity depending on the aggregate and asphalt chemistry. This bond may not be very influential on mixture stiffness once the mixture becomes dry and stays dry (11); however, the durability of the mixture may be significantly affected by wet-dry cycles or freeze-thaw cycles such as in the conditioning procedure used in this study (6).

The phenomena outlined above are exhibited in mixtures from both projects. For the mixtures from the Warren-Scappoose project, the modulus decreases with increased fluids content caused by loss of interparticle friction and interlock, since there was no increase in density of the mixture (Table 2.3), as would be expected with a gravel aggregate. The performance of this mixture becomes much worse as more moisture was present at mixing because of weak bonding, low asphalt content, and high voids in the mix.

For the mixtures from the North Oakland-Sutherlin project the

tendency for increased modulus with a small addition of moisture is probably due to increased aggregate density affected by improved workability, as would be expected with crushed rock aggregate with greater potential for improved packing than a gravel aggregate. Additional moisture increases the aggregate density and mix density (Table 2.3), but reduces interparticle interlock and friction and therefore the resilient modulus. This mixture is more durable than that from the Warren-Scappoose project because of the increased density and higher asphalt content affected by improved workability.

An additional reason for the different behavior of the mixtures from the two projects is aggregate absorption. The marginal aggregate from the North Oakland-Sutherlin project was much more absorptive (up to 7% by weight), and this could be an advantage in mixtures using moist aggregates where water might prevent asphalt absorption and thus render more asphalt available for coating. It should be noted that following the conditioning procedure, mixtures from the Warren-Scappoose project had free water on the broken surface, whereas there was none on the surface of mixtures from the North Oakland-Sutherlin project, indicating higher aggregate absorption in the latter but lower mixture absorption caused by low air voids.

2.3.1.2 Fatigue Results: After the resilient modulus was measured, the fatigue life was determined at three different fixed initial tensile strain levels between 50 and 200 microstrains. The test was based on the controlled-load test method described in Ref 10. The fatigue life of asphalt mixtures is a function of initial tensile strain and may be expressed by Eq 1, where the constants K and m shown in Table

Table 2.4 Fatigue Data ($N_f = K[1/\epsilon_t]^m$).

(a) Warren-Scappoose

	As-Compacted			Conditioned		
	K	m	r^2	K	m	r^2
0% Moisture	1.492×10^{-6}	2.387	.995	5.636×10^{-4}	1.790	.925
1% Lime	3.101×10^{-9}	3.080	.972	7.133×10^{-9}	3.010	.978
0.5% Pavebond	1.268×10^{-8}	2.903	.978	3.845×10^{-6}	2.303	.959
1% Moisture	2.729×10^{-6}	2.344	.912	1.270×10^{-7}	2.718	.929
3% Moisture	6.821×10^{-5}	1.932	.953	4.908×10^{-9}	3.044	.919
3% Moisture/1% Lime	4.989×10^{-8}	2.921	.968	5.412×10^{-6}	2.321	.911
3% Moisture/0.5% Pavebond	9.186×10^{-5}	1.922	.975	1.004×10^{-2}	1.474	.941

r^2: Coefficient of Determination.

Table 2.4 Fatigue Data ($N_f = K[1/\epsilon_t]^m$, Continued).

(b) North Oakland-Sutherlin

	As-Compacted			Conditioned		
	K	m	r^2	K	m	r^2
0% Moisture	2.451×10^{-7}	2.658	.999	1.637×10^{-16}	5.083	.976
1% Lime	6.541×10^{-9}	3.109	.941	1.295×10^{-8}	3.023	.905
0.5% Pavebond	2.498×10^{-11}	3.724	.968	4.797×10^{-20}	6.055	.960
1% Moisture	6.717×10^{-11}	3.621	.944	4.617×10^{-14}	4.464	.826
3% Moisture	3.604×10^{-18}	5.599	.861	3.102×10^{-14}	4.712	.927
3% Moisture/1% Lime	1.716×10^{-18}	5.675	.944	2.982×10^{-4}	2.075	.880
3% Moisture/0.5% Pavebond	4.210×10^{-17}	5.307	.973	1.735×10^{-14}	4.797	.917

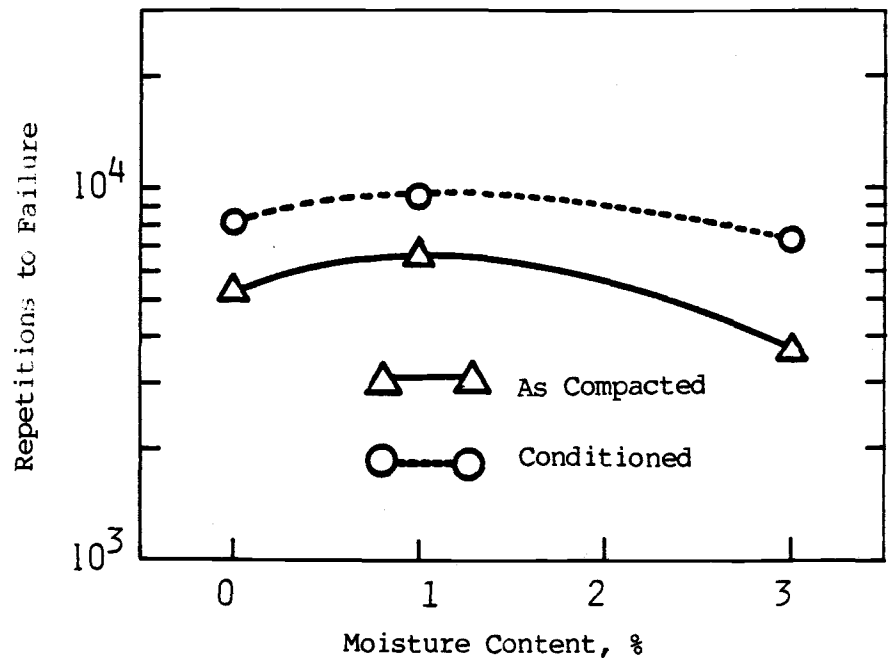
r^2: Coefficient of Determination.

2.4 were determined by linear regression analysis.

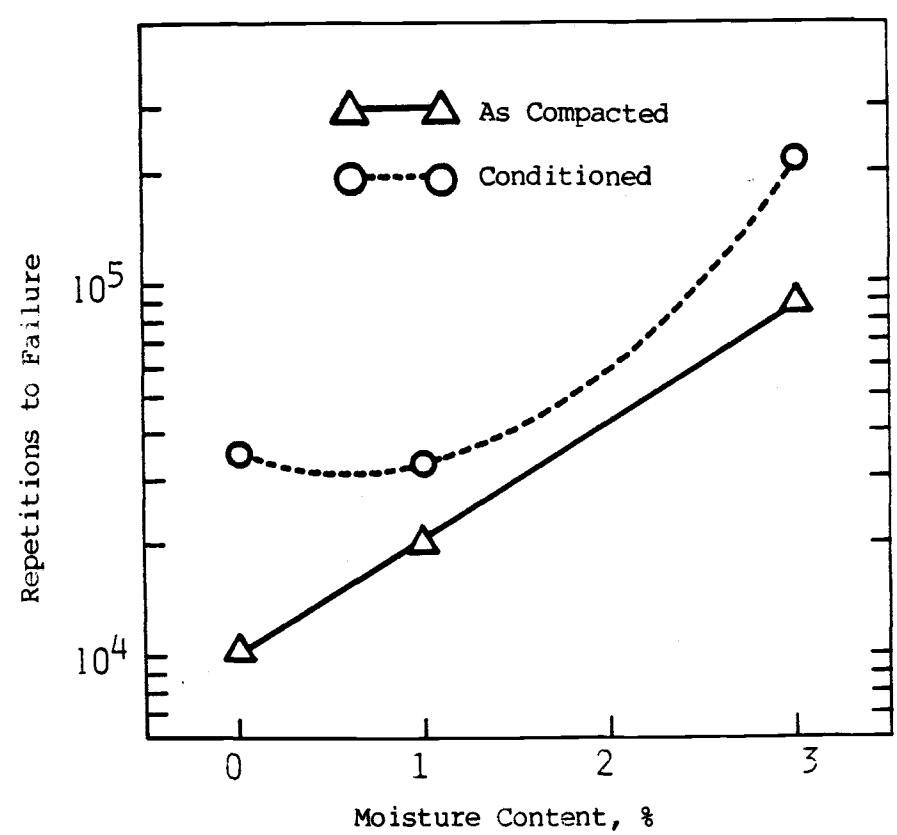
Figure 2.4 shows that the number of load repetitions to failure at a measure strain of 100 microstrain for each project. These illustrate the results given in Table 2.4, and show quite clearly an "optimum" moisture content of 1% for the Warren-Scappoose project, for as compacted and conditioned specimens. For the North Oakland-Sutherlin project, the fatigue life of as-compacted specimens decreases as moisture content increases up to 1% and then increases with increasing moisture. This fatigue result is the inverse of the modulus result presented in Figure 2.3.b. After conditioning, however, the fatigue life increases as moisture content increases.

The above results can be explained in terms of the stiffness and air-void contents of the materials, which in turn are influenced by the aggregate type and asphalt content. For the Warren-Scappoose project stiffness decreased with increasing moisture content (Figure 2.3), and the air-void content was lowest at 1% moisture (Table 2.3). The substantial loss in modulus should lead to longer fatigue lives at higher moisture content on the basis of the controlled-load test, but the air voids effect plus the probable weaker bond between asphalt and aggregate (caused by the free water between them) lead to the shorter fatigue lives at higher moisture content for this project.

For the North Oakland-Surtherlin project, the much longer fatigue lives at higher moisture content are a result of lower air void contents. For both projects, fatigue lives were longer after conditioning for all moisture conditions (Figure 2.4) because of the reduced stiffness that ocurred in all cases (Figure 2.3).



(a) Warren-Scappoose

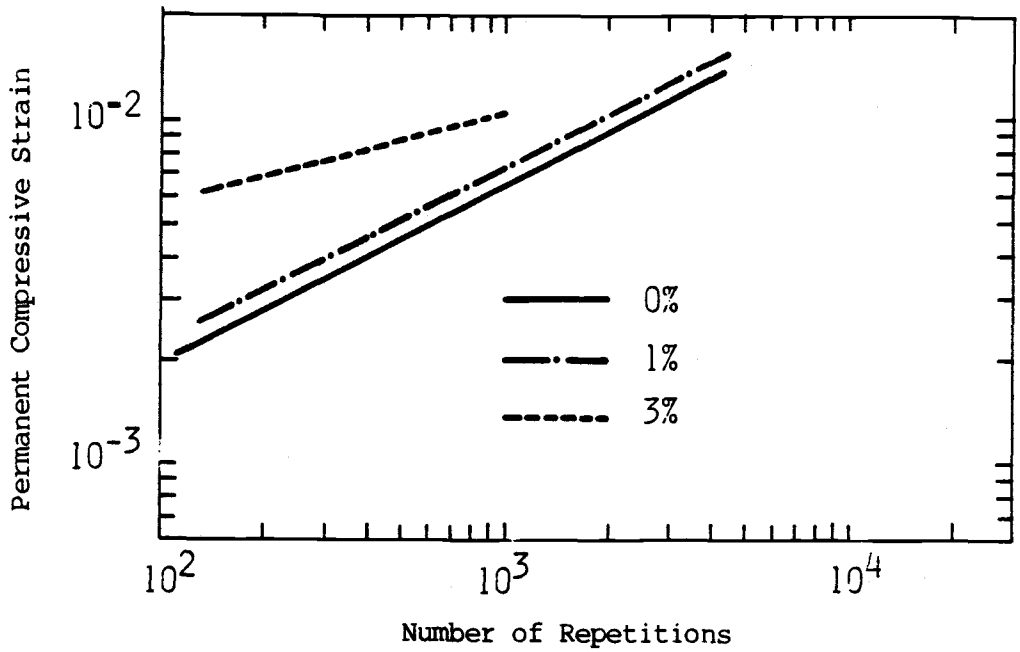


(b) North Oakland-Sutherlin

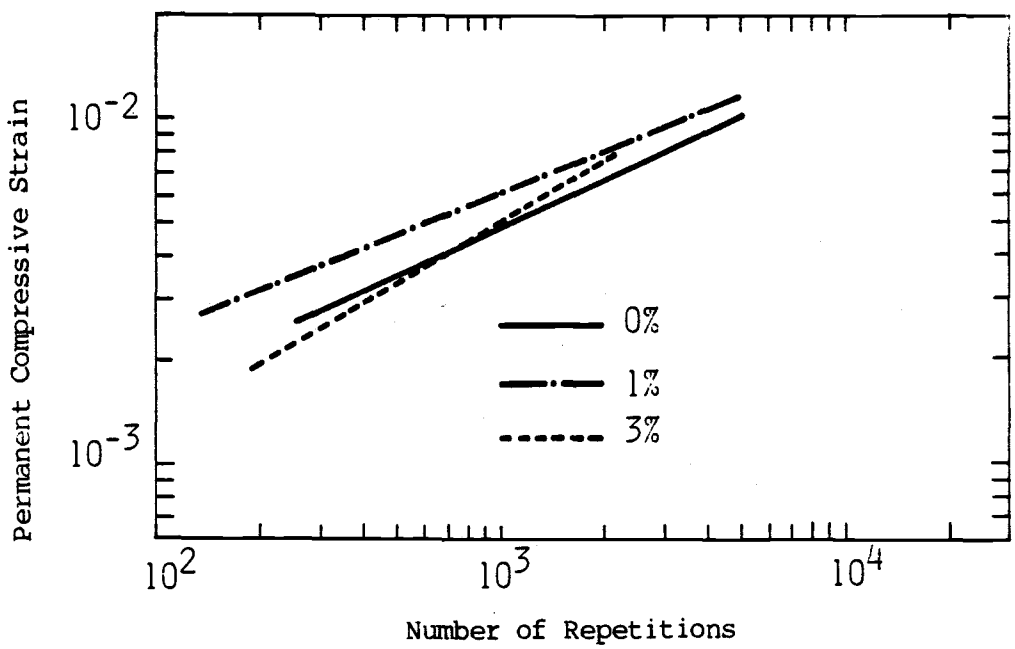
Figure 2.4 Fatigue Life with Moisture at ϵ_t of 100 Microstrain.

2.3.1.3 Deformation Results: The effect of moisture on permanent deformation of mixtures at initial tensile strain of 100 microstrain for the Warren-Scappoose project and 150 microstrain for the North Oakland-Sutherlin project is presented in Figures 2.5 and 2.6, respectively. For the Warren-Scappoose project, the as-compacted specimens with 3% moisture again performed the worst (less repetitions are required to reach a fixed compressive strain), and specimens with 0% moisture performed the best (more repetitions are required to reach a fixed compressive strain) as shown in Figure 2.5. After conditioning, specimens with 1% moisture gave the worst results. For the North Oakland-Sutherlin project, as-compacted specimens with 1% moisture (showing the highest modulus in Figure 2.3.b) performed best, while those with 3% moisture performed worst in the high compressive strain range, as demonstrated in Figure 2.6. After conditioning, the results show that specimens with 1% moisture again gave the best results while those with 0% moisture performed the worst.

The above results are again related to the stiffness, air voids, and asphalt content of the mixtures. Stiffness is a major influence on permanent deformation resistance and therefore the trend of decreasing resistance with decreasing stiffness is to be expected. However, this was not the case for all of the results. The performance of the conditioned specimens from the Warren-Scappoose project was slightly better than for the as-compacted specimens although they were of lower stiffness (Figure 2.3). This anomaly may due to the comparison method on the strain level and the diametral mode of the testing, which is most suitable for stiffness and fatigue testing, but after a large number of repetitions the permanent strain is significantly influenced by various

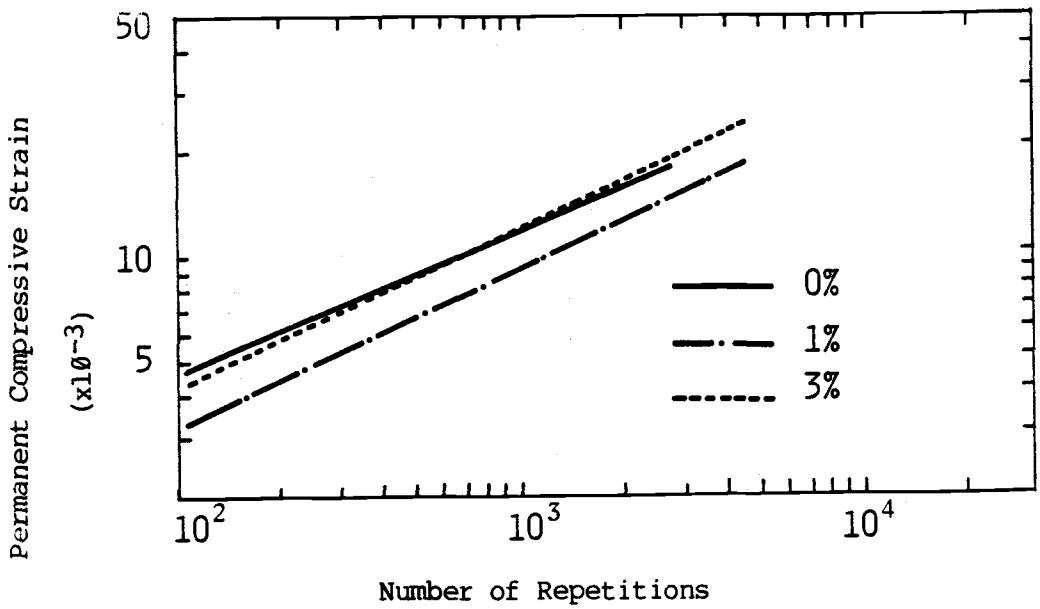


(a) As Compacted

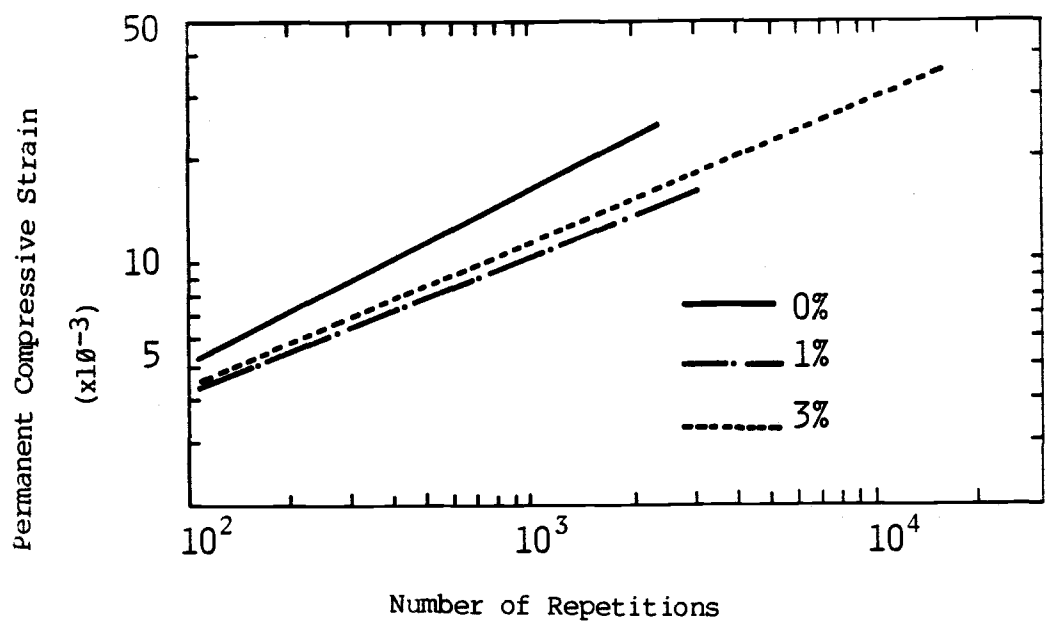


(b) Conditioned

Figure 2.5 Effect of Moisture on Permanent Deformation of Warren-Scappoose Project at ϵ_t of 100 Microstrain.



(a) As Compacted



(b) Conditioned

Figure 2.6 Effect of Moisture on Permanent Deformation of North Oakland-Sutherlin Project at ϵ_t of 150 Microstrain.

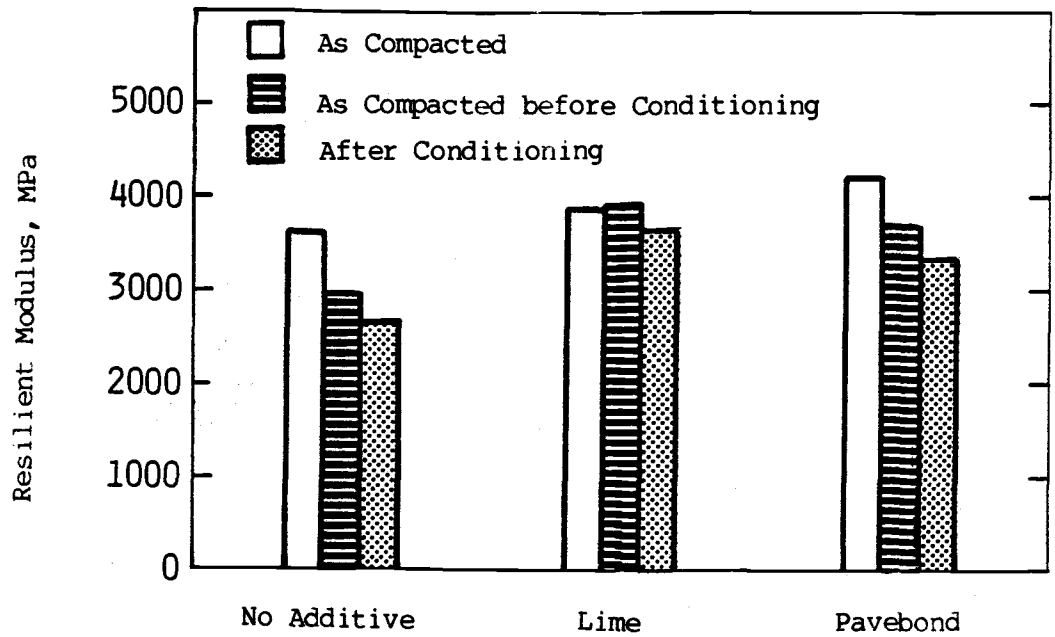
geometric and loading factors as well as the mixture variables. The behavior of the mixtures from the North Oakland-Sutherlin project is as expected with the stiffness mixtures performing best.

2.3.2 Effect of Additives

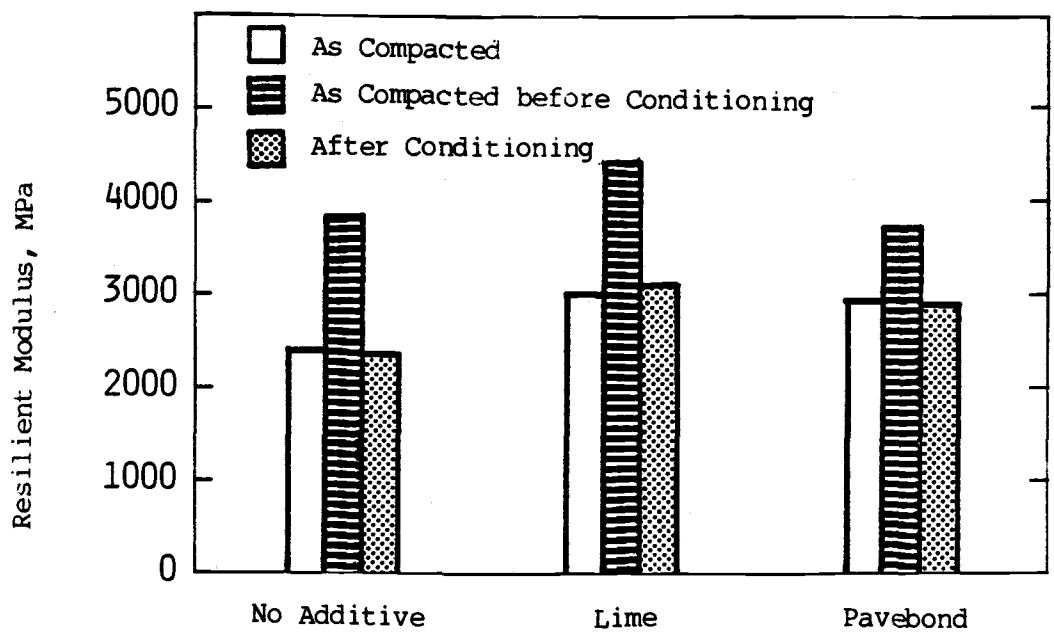
2.3.2.1 Modulus Results: A major part of the testing program was aimed at evaluating the effect of lime (1%) and Pavebond Special (0.5%) on the performance of asphalt mixtures. The effect of additives on modulus for mixtures with no moisture is presented in Figure 2.7. For both projects, introduction of additives increases the modulus of as-compacted specimens very slightly. After conditioning, specimens with additives obtained a much higher modulus than the "no treatment" specimens and specimens with 1% lime obtained the highest modulus.

With 3% moisture, the increase of modulus and retained modulus ratio by lime is substantial for the Warren-Scappoose project, and there is an improvement in retained modulus as shown in Figure 2.8. For the North Oakland-Sutherlin project the moduli of as-compacted and conditioned specimens with no additives are about the same as those with lime or Pavebond Special.

For the Warren-Scappoose project, without moisture there is little benefit from use of additives except with conditioned specimens. However, for specimens with moisture, the effect of lime is superior to that of Pavebond Special. For the North Oakland-Sutherlin project, there is little benefit shown use of additives. Iahai and Craus (13) emphasize that the contribution of the hydrated lime to adhesion is mobilized only in the presence of water. The above results confirm their conclusions since the best improvement of performance was obtained

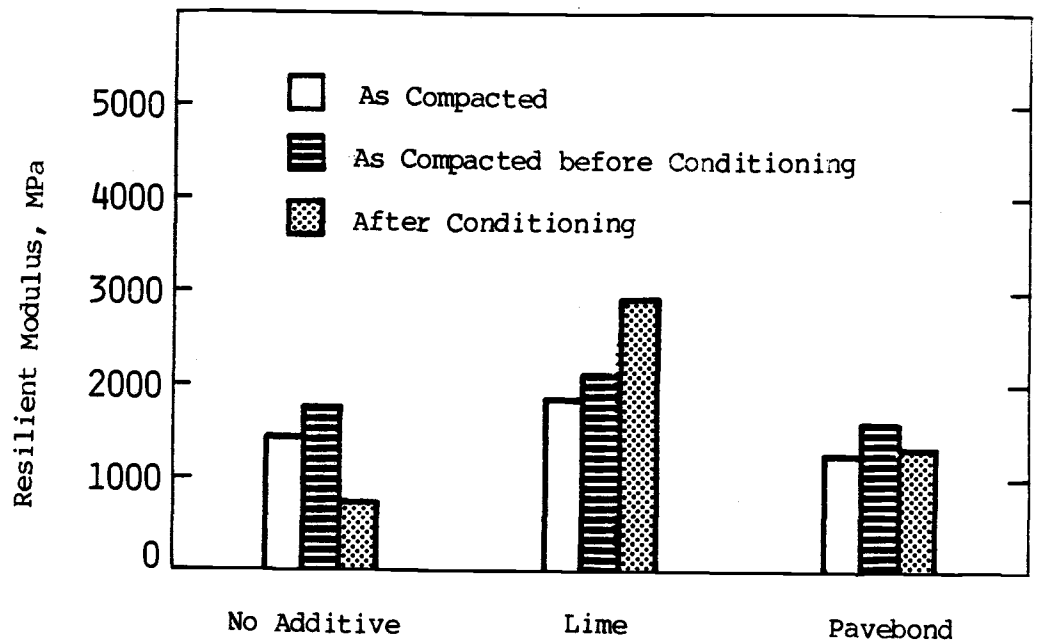


(a) Warren-Scappoose

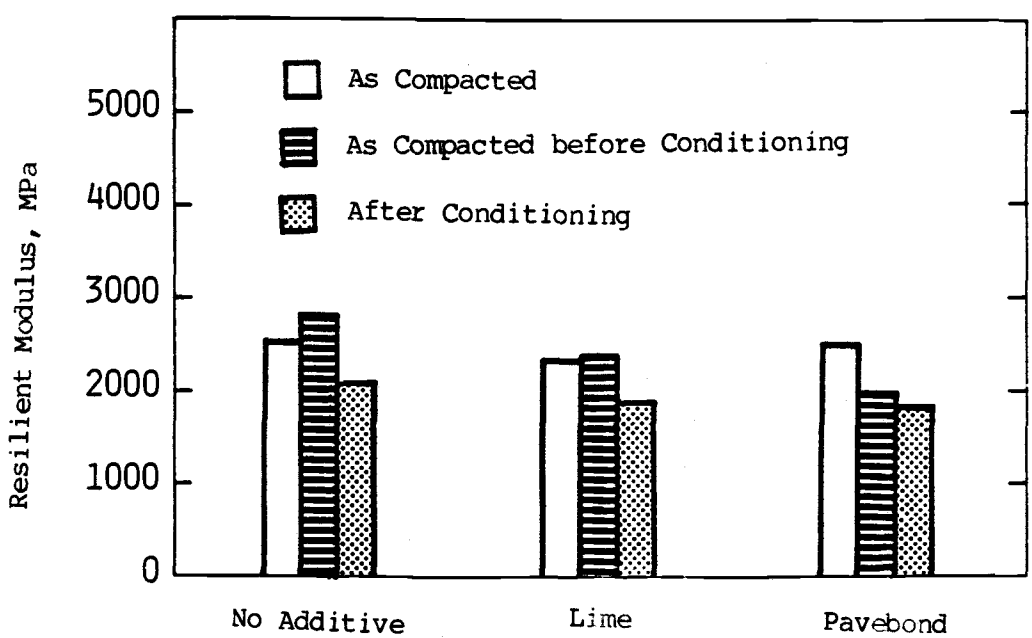


(b) North Oakland-Sutherlin

Figure 2.7 Effect of Additives without Moisture on Resilient Modulus.



(a) Warren-Scappoose



(b) North Oakland-Sutherlin

Figure 2.8 Effect of Additives with Moisture (3%) on Resilient Modulus.

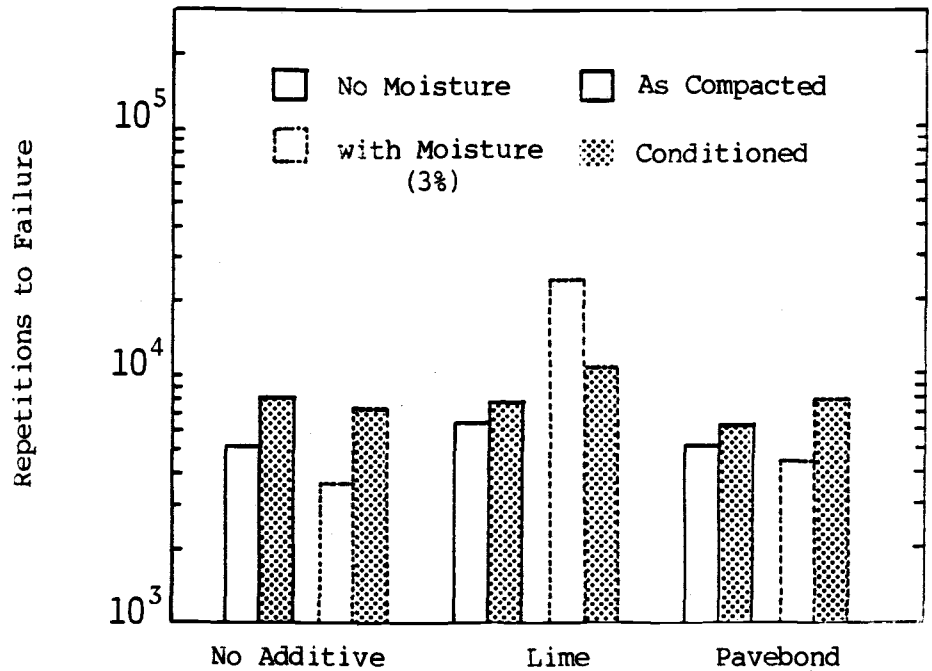
in the Warren-Scappoose specimens with 3% moisture. There was little benefit in the North Oakland-Sutherlin project because of the superior durability of the mixture.

It can be seen that there is a partial stiffening effect on the specimens without moisture for the North Oakland-Sutherlin project and with moisture for the Warren-Scappoose project during a period of seven weeks between as-compacted and conditioned tests.

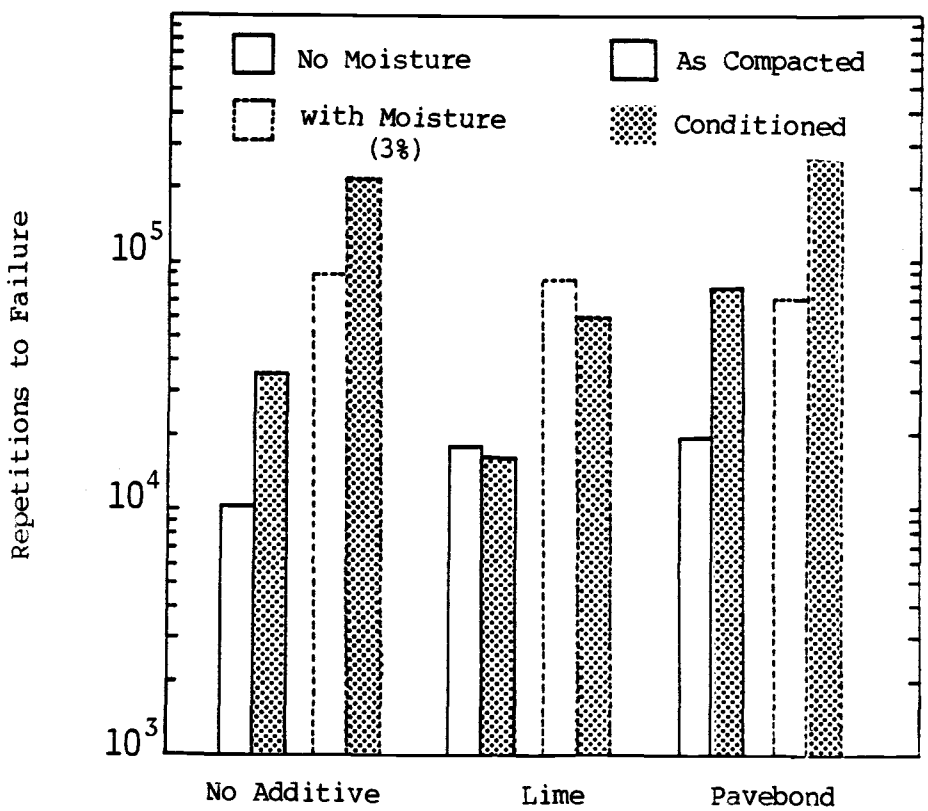
2.3.2.2 Fatigue Results: The effects of additives on fatigue life of specimens with and without moisture are summarized in Table 2.4 for both projects. A comparison between fatigue of as-compacted and conditioned specimens mixed with additives at 100 microstrain is shown in Figure 2.9. It is clear that for the Warren-Scappoose mixtures with no moisture, additives show no benefit, but, when moisture is present, lime shows significant benefit.

For the North Oakland-Sutherlin project, there is clear benefit from both additives, particularly Pavebond Special, for specimens with no moisture. However, for this project, with 3% moisture, there is no benefit shown with lime and very little benefit with Pavebond Special when examining the results for as-compacted or conditioned specimens. Results at strain levels other than 100 microstrain are similar to those shown in Figure 2.9.

The above results may be explained, in similar manner to that done earlier for mixtures without additives, in terms of modulus, air voids, and degree of coating of the mixtures. Figures 2.7 and 2.8 show moduli for the specimens with and without additives, and Table 2.3 shows the air-void contents. Considerations of these data together with Table 2.4



(a) Warren-Scappoose



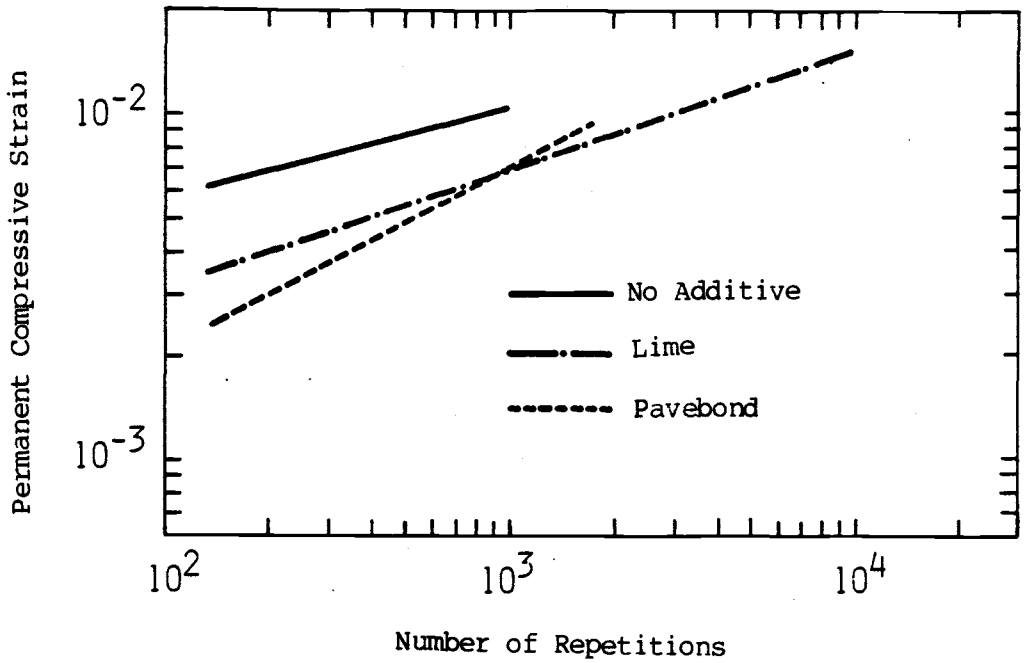
(b) North Oakland-Sutherlin

Figure 2.9 Fatigue Life with Additives at ϵ_t of 100 Microstrain.

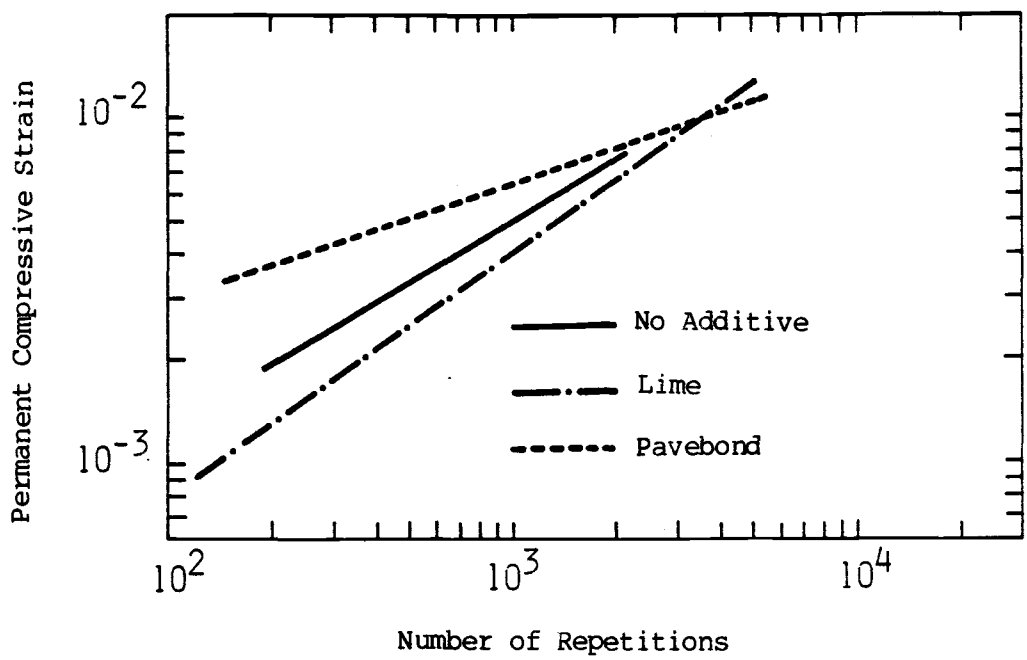
and Figure 2.9 confirms the previous observation that the additives are only likely to be of significant benefit in mixtures with questionable durability, that is, those with high air voids and low asphalt content. Hence, lime shows a significant advantage for the Warren-Scappoose project, but neither additive shows an advantage for the North Oakland-Sutherlin project. It is also significant to note that much higher fatigue lives were achieved by maximum from the North Oakland-Sutherlin project under all circumstances.

2.3.2.3 Permanent Deformation: The effect of additives on permanent deformation of mixtures without and with moisture based on the initial tensile strain of 100 microstrain are presented in Figures 2.10 and 2. 11 for the Warren-Scappoose project and in Figures 2.12 and 2.13 for the North Oakland Sutherlin project at the initial tensile strain of 150 microstrain. For the Warren-Scappoose project, as-compacted specimens with no additive perform the worst as shown in Figure 2.10. Mixtures with Pavebond Special at low compressive strains and lime at high compressive strains perform best. After conditioning, mixtures with lime deformed the most. For this project, the behavior of as-compacted specimens with moisture but no additive is similar to that of conditioned specimens without moisture as shown in Figure 2.10. After conditioning, specimens mixed with Pavebond Special again performed the worst, while specimens mixed with no additive performed better than those with lime.

For the North Oakland-Sutherlin project, without moisture, specimens mixed with lime show the best performance for both as-compacted and conditioned specimens as shown in Figure 2.12. With moisture, specimens

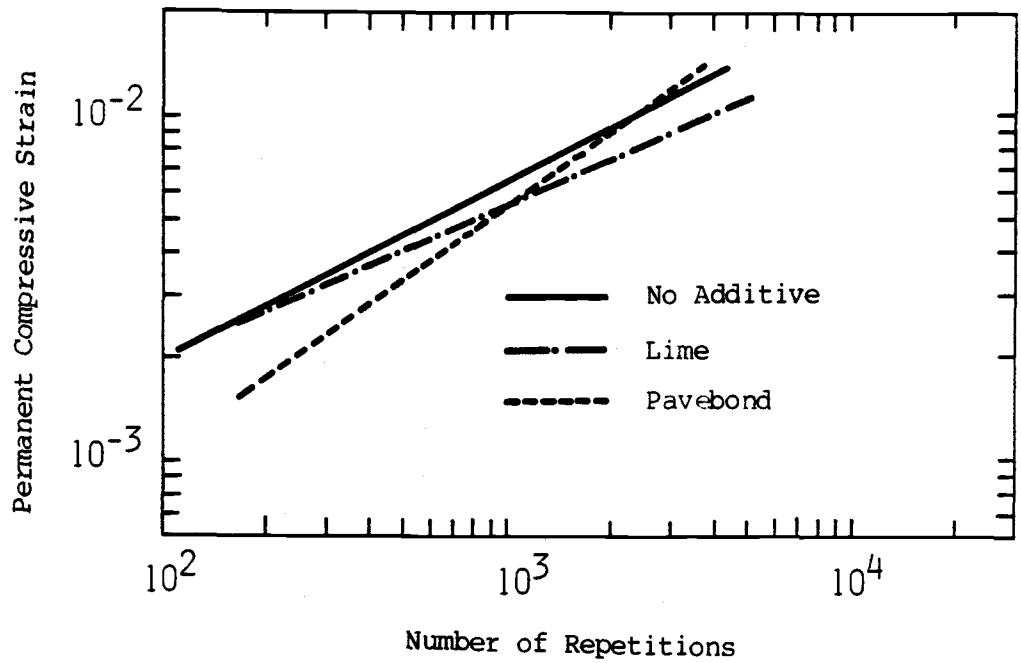


(a) As Compacted

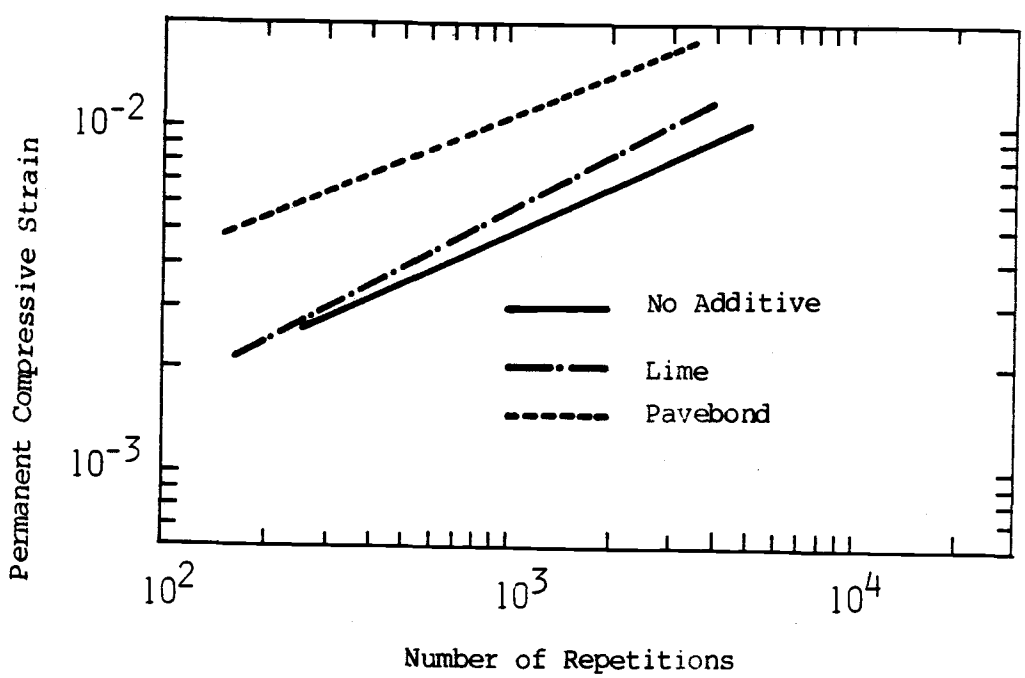


(b) Conditioned

Figure 2.10 Effect of Additives with Moisture (3%) on Permanent Deformation of Warren-Scappoose Project at ϵ_t of 100 Microstrain.

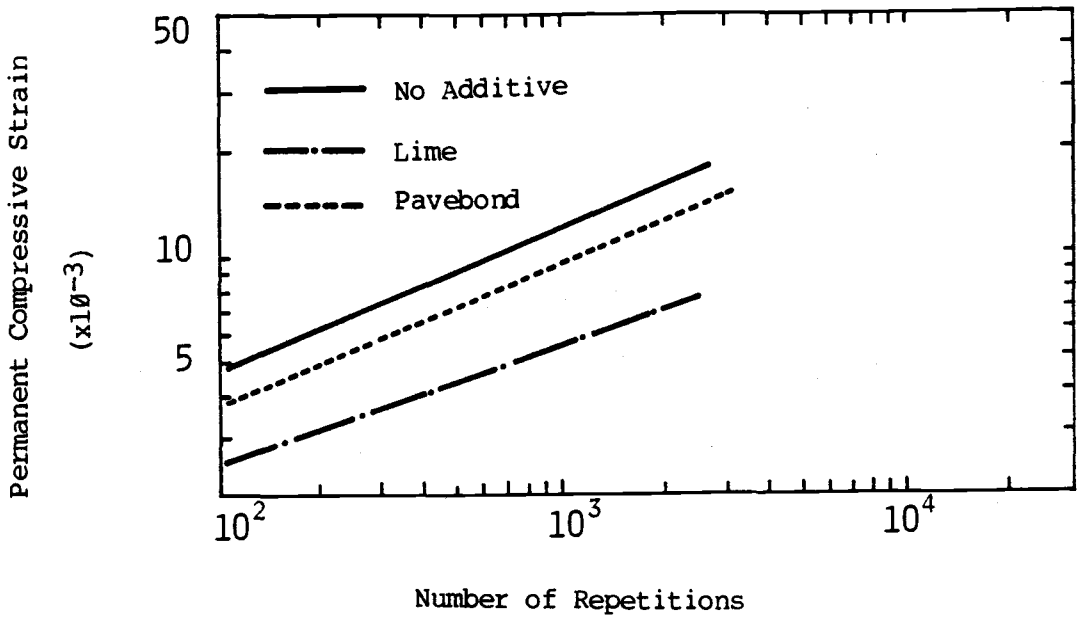


(a) As Compacted

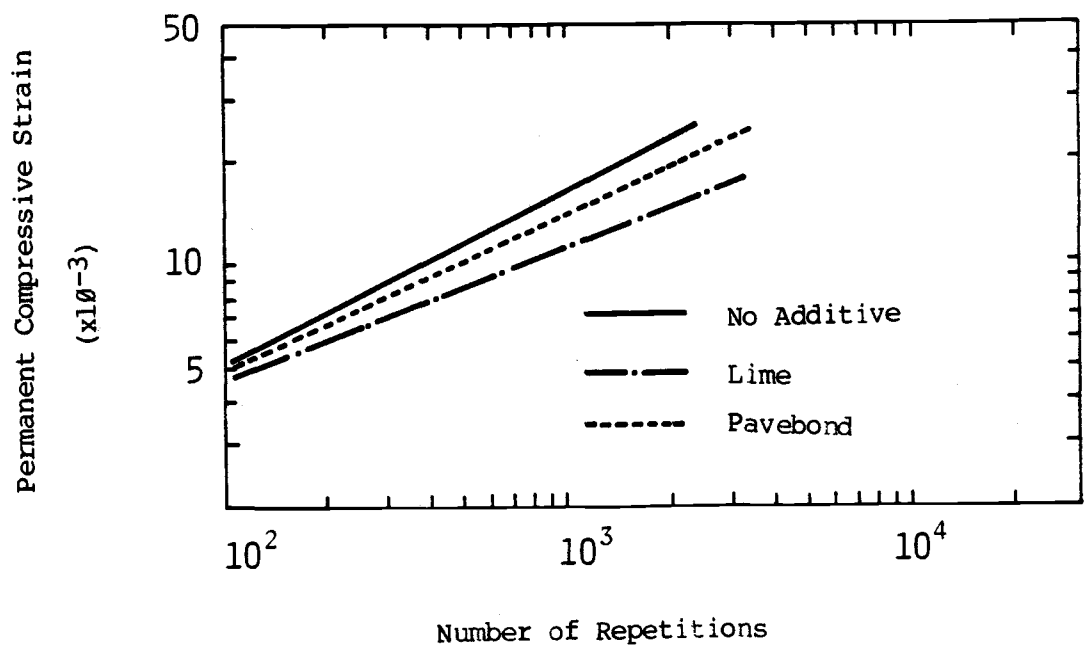


(b) Conditioned

Figure 2.11 Effect of Additives without Moisture on Permanent Deformation of Warren-Scappoose Project at ϵ_t of 100 Microstrain.

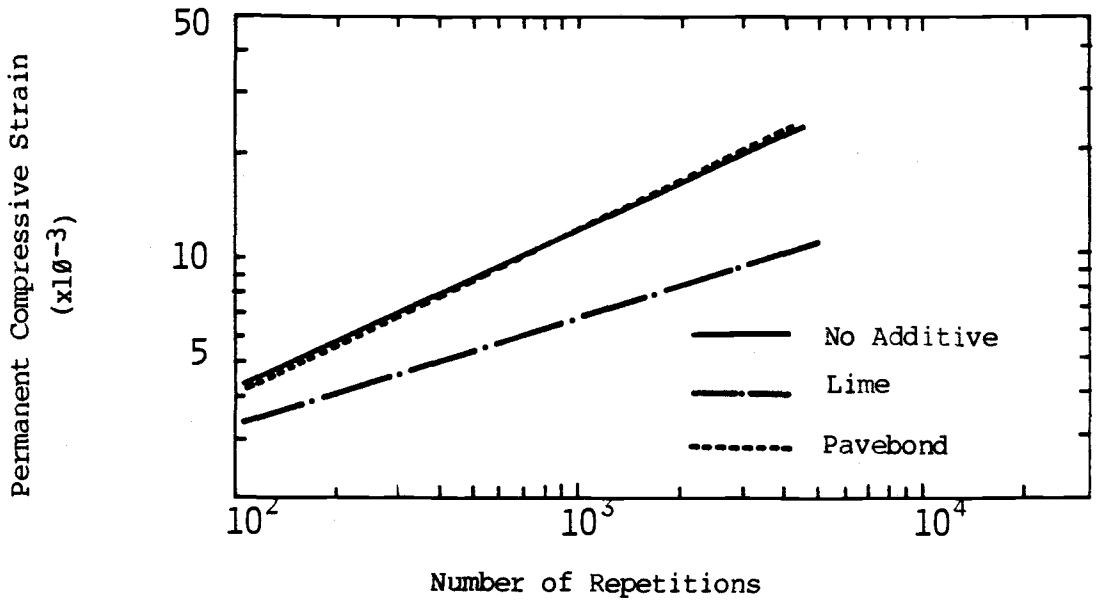


(a) As Compacted

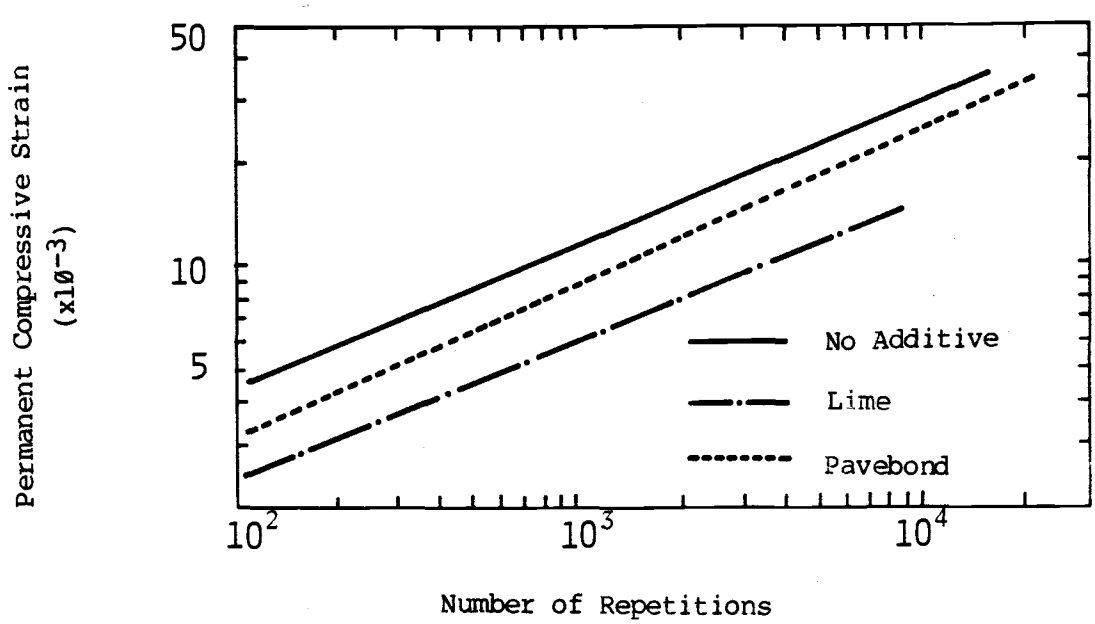


(b) Conditioned

Figure 2.12 Effect of Additives without Moisture on Permanent Deformation of North Oakland-Sutherlin Project at ϵ_t of 150 Microstrain.



(a) As Compacted



(b) Conditioned

Figure 2.13 Effect of Additives with Moisture (3%) on Permanent Deformation of North Oakland-Sutherlin Project at ϵ_t of 150 Microstrain.

For the North Oakland-Sutherlin project, without moisture, specimens mixed with lime show the best performance for both as-compacted and conditioned specimens as shown in Figure 2.12. With moisture, specimens mixed with lime again show the best performance for both as-compacted and conditioned specimens as shown in Figure 2.13. With and without moisture specimens mixed with no additives show the worst performance. As mentioned earlier, deformation results at high strain levels should be treated with caution when obtained with a diametral device. Indeed, as in the evaluation of results for mixtures without additives, the deformation results do not compare well with other data.

2.3.3 General Discussion

The above discussion indicates that fatigue and deformation results may be explained in terms of the stiffness results, with a reduction in stiffness tending to cause prolonged fatigue life (because of the comparison based on strain level) but a decrease in deformation resistance. It is therefore essential to recognize that the fatigue results should not be considered by themselves as an indicator of the effect of moisture and additives on pavement life, since they do not reflect differences in mixture stiffness. Rather a study such as that reported by Bell et al. (14) should be done to determine the effect of change in stiffness on the distress parameters associated with pavement failure. In particular, determination of critical tensile strain in the asphalt layer can be used in conjunction with fatigue data and the number of load repetitions to failure assessed accordingly. Such a study is outside the scope of this paper.

The results of this study partially confirm the assessment of the

performance of the projects that was observed after construction (2,3). In particular, the raveling problems exhibited in the Warren-Scappoose project were attributed to high air void content following construction (10 to 15% for the wearing course and 6 to 11% for the base course) and poor adhesion or stripping caused by the water content (0.64 to 1.0%). This study confirms that air voids contributed to poorly performing mixtures particularly if mix moisture was initially high.

For the North Oakland-Sutherlin project, the laboratory mixtures performed better than expected compared to the in-service performance. This was associated with the excellent durability of the mixtures caused by high asphalt content and low air voids, and good mechanical properties, even with mix moisture exceeding what is now regarded as tolerable amount in Oregon (0.7%). The problems observed in service may not have been associated with the low quality aggregate used, but rather the fairly high air voids in the mixture (2).

The results of this study are in keeping with the philosophy that good mix design and high compaction (low air voids) result in a high performance mix. The results of a previous study at Oregon State University (1) confirmed this philosophy, as do those of the current study.

2.4 CONCLUSIONS AND RECOMMENDATIONS

2.4.1 Conclusions

The following major conclusions are drawn from the findings of this laboratory study:

1. The effect of mix moisture and additives on asphalt pavement performance are best explained by the resilient modulus. A

lowering of modulus tends to increase fatigue life, but decrease deformation resistance.

2. The results for mixtures with no additives showed that excess mixing moisture (3%) was detrimental for both project, particularly with regard to reduction in modulus after conditioning in the higher air-void content mixtures (Warren-Scappoose project).
3. For the North Oakland to Sutherlin project that utilize a low-quality crushed rock aggregate, 1% moisture resulted in improved performance because of the substantial improvement in compaction that resulted.
4. For mixtures with additives, the detrimental effect of 3% moisture on the resilient modulus was substantially reduced by use of lime (1%) in the Warren-Scappoose project, with Pavebond Special (0.5%) showing limited benefit. Neither additive showed substantial benefit in the North Oakland-Sutherlin project.
5. Additives are of limited benefit in mixtures with high density and which achieve good performance without additives, such as the North Oakland-Sutherlin project. Although, this project used a low-quality aggregate, the mix design produced a high performance mixture. Additives, particularly lime, were of substantial benefit in the Warren-Scappoose project, which had a high air-void content and low retained modulus without the additives.

2.4.2 Recommendations

The following recommendations can be made from the results of this phase of the study:

1. Since the limit of acceptable moisture content could vary from one project to another (depending on the absorption of the aggregate), limiting values could also be established after a regular mix design by additional tests at the optimum asphalt content with moisture contents varying up to the maximum absorption of the aggregate.
2. It is recommended to use as much asphalt cement as possible to maximize durability and minimize the damage from moisture, while ensuring adequate resistance to permanent deformation.
3. Mix density should be maximized, commensurate with good mix design pracitce, to achieve good mechanical properties and durability.
4. Additives should be selected after appropriate tests have established values such as the retained modulus ratio.
5. It is necessary to establish test methods to determine the effect of residual moisture on mixtures. The procedure employed in this study together with the specimen preparation techniques are a good starting point.
6. Since the effect of moisture and effectiveness of additives is a complex function of asphalt, aggregate, and additive interactions, it is necessary to establish a simple test to determine the compatibility of these components.

2.5 REFERENCES

1. Puangchit, P., Hicks, R.G., Wison, J.E., and Bell, C.A., "Development of Rational Pay Adjustment Factors for Asphalt Concrete," Transportation Research Record 911, Transportation Research Board, Washington, DC, 1983, pp. 70-79.

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3.0 DEVELOPMENT OF LABORATORY OXIDATIVE AGING PROCEDURES FOR ASPHALT CEMENTS AND ASPHALT MIXTURES

Ok-Kee Kim
Graduate Research Assistant
Department of Civil Engineering
Oregon State University

Chris. A. Bell
Associate Professor of Civil Engineering
Oregon State University

James E. Wilson
Assistant Engineer of Materials
Oregon Department of Transportation

and

Glenn Boyle
Group Leader - Asphalt Mix Design
Oregon Department of Transportation

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ABSTRACT

This paper presents an evaluation of an oxidative aging procedure for asphalt materials. Test results and the effectiveness of the aging device used are presented. The study was performed by Oregon State University and the Oregon Department of Transportation. This study involved laboratory tests on field core samples as well as laboratory mixture samples and asphalt cements used for three projects constructed in Oregon.

The procedure selected for aging laboratory mixtures involved using a Pressure Oxidation Bomb (POB), a sealed container in which asphalt mixtures and/or asphalt samples were subjected to pure oxygen at 100 psi pressure at 60°C, for periods of up to 5 days. Resilient modulus and fatigue tests were performed to measure the properties of cores and laboratory mixtures (before and after aging). The asphalt samples were aged on a Fraass plaque to achieve minimum disturbance of the sample, and the degree of aging was assessed by changes in the Fraass breaking temperature.

The results of this study showed that the POB was an effective means of producing measurable changes in both mixtures and asphalt samples. However, the mixture properties were substantially different from those measured for the field core samples, while the asphalt properties were similar. As evaluation parameters, the modulus ratio and Fraass breaking temperature are good indicators of aging rate of mixtures and asphalt cement, respectively. The study also indicated that aging rate is a function of the air voids in the mixture and asphalt properties.

3.1 INTRODUCTION

3.1.1 Background

Premature failure or poor performance of asphalt pavements often results from weakening of the adhesive bond between asphalt cement and aggregate particles by the action of moisture and/or mechanical stresses, and/or aging of asphalt cement. Aging is the change of properties of asphalt pavements with time and usually is accompanied by hardening of the asphalt cement. Petersen (1) indicated that three fundamental composition-related factors govern the changes that could cause hardening of asphalts in pavements:

1) loss of the oily components of asphalt by volatility or absorption by porous aggregates;
2) changes in the chemical composition of asphalt molecules from reaction with atmospheric oxygen; and
3) molecular structuring that produces thixotropic effects (steric hardening).

Oxidation of asphalt is generally considered a major factor contributing to the hardening and embrittlement of asphalt pavement. This oxidation occurs in the preparation and laydown of hot mix pavements as well as due to environmental aging while in service. Excessive hardening of the asphalt cement is undesirable because it often leads to problems associated with pavement embrittlement and cracking. The rate of hardening is affected by the chemical composition of asphalt, light, aggregate properties, and the ambient temperature (2-5). After the development of the recovery method by Abson in 1933 provided a means of recovering the asphalt from hot plant mixtures immediately and after periods of aging in the pavement, numerous methods to evaluate the

properties of "aged" asphalt cement have been developed. Many aging methods attempt to correlate short-term laboratory aging with the change of asphalt properties occurring after long exposure in the pavement as well as during mixing operations.

Methods used by various investigators include use of high temperatures, light, chemical oxidation agents and oxidation in solution under oxygen pressure. Reference 6 summarizes the various test conditions, such as temperature and exposure time, used for aging methods along with evaluation parameters. Most of the methods represent the short term aging due to the mixing process rather than the long term aging. Most of the evaluation parameters consist of measurements of consistency of the asphalt cement, such as penetration, viscosity, and ductility. For asphalt mixtures, Pauls and Welborn (7) used the compressive strength of the weathered mixtures as the evaluation parameter. Kemp and Predoehl (8) used the resilient modulus of the weathered briquettes as the evaluation parameter.

3.1.2 Purpose

As discussed above, the majority of aging procedures developed previously are for short term rather than long term effects. The major objective of the study reported in this paper was to develop a laboratory procedure to simulate long term aging effects. The approach selected was one similar to the Lottman test for moisture effects, which has been used extensively by the authors (9, 10). This approach is to measure fundamental properties (such as tensile strength, resilient modulus, fatigue life and permanent deformation) of the asphalt mixture before and after conditioning (i.e., either by moisture or

oxygen). It was also desired to evaluate asphalt cements as well asphalt mixtures before and after oxygen conditioning, i.e., aging in oxygen. The purpose of this paper is to (a) present an oxidative aging procedure adopted for a recent aging study by Oregon State University and the Oregon Department of Transportation, and (b) discuss the test results and effectiveness of the aging device.

3.2 SELECTION OF AGING AND EVALUATION METHODS

After review of the various aging procedures used previously, it was decided that the method adopted in the aging study should attempt to reproduce oxidative aging occurring after construction of a pavement. The research approach included comparison of artificially aged laboratory prepared samples with cores aged in the field. The extent to which aging of laboratory mixtures could be achieved with the developed methods is emphasized in this paper. Both laboratory mixtures and asphalt samples were subjected to aging. Asphalt sample aging was included in the test program to see if the methods adopted would indicate asphalt aging susceptibility. A major goal of this research was to utilize an aging device suitable for both types of samples.

The pressure oxidation bomb (POB), originally developed in Britain (11) and recently used by Edler et al. (12) in South Africa, was selected as the most suitable aging device and modified in the aging study. This modified device can contain two mixtures or one mixture and several Fraass samples. As reported by Thenoux et al.(13), the use of Fraass samples for aging asphalt has the advantage of minimal disturbance to the asphalt which is tested on the "container" on which it is aged.

The POB was operated in the aging study with the samples contained in a pure oxygen environment at 100 psi and at 60°C. This compares to 300 psi and 150°C used by Edler et al.(12). These levels were arbitrarily selected after consideration of safety concerns regarding the use of pressurized oxygen and in order to preserve the shape of the mixtures.

3.3 MATERIALS TESTED

Asphalt cements and aggregates for three projects constructed in Oregon [Idylwood Street (1974) Plainview Road-Deschutes River (1980), Arnold Ice Caves-Horse Ridge (1973)] were used for the laboratory simulation aging study. These projects were 5 to 10 years old and in each project original samples of asphalt and representative aggregate were available. For each of the three projects the mix type was B-mix (Table 3.1) and the grade and optimum content of asphalt cement were as follows: AR 4000 of 7.0% for the Idylwood Street, AR 4000 of 5.5% for the Plainview Road-Deschutes River project, and 120/150 Pen. of 6.5% for the Arnold Ice Caves-Horse Ridge project. Laboratory mixtures were prepared using a kneading compactor, and field cores were obtained for each project.

Table 3.1 Aggregate Gradation, Class B Mix.

Sieve Size	Aggregate Gradation, % Passing		
	Idylwood Street	Plainview Road-Deschutes River	Arnold Ice Caves-Horse Ridge
1 in.			100
3/4 in.	100	100	97
1/2 in.	87	87	84
3/8 in.	78	74	74
1/4 in.	63	60	60
# 4	52	52	53
# 10	30	31	32
# 40	12	14	15
#200	5	5	5

3.4 TEST PROGRAM

3.4.1 Cores

The Oregon Department of Transportation (ODOT) testing program included the tests for aggregate gradation, asphalt cement contents, air voids, and recovered asphalt cement properties. The repeated load diametral test for modulus and fatigue life of cores was performed by Oregon State University.

3.4.2 Laboratory Mixtures

The variables considered in the laboratory mixture preparation were:

1) Compaction level: 94% of maximum density (100 blows at 500 psi after 20 blows at 250 psi and leveling load of 12500 lbs for 15 seconds), and 88% of maximum density (30 blows at 100 psi and leveling load of 1000 lbs for 15 seconds);
2) Aging period: 0, 1, 2, 3, and 5 days.

Each of the above variables was studied relative to a standard mix consisting of the original mix design used for the projects studied. Following the standard ODOT procedure (14) using a kneading compactor, 4-in. (100 mm) diameter by 2.5-in.(63 mm) high specimens were fabricated for three projects [Idylwood Street (ID-ST), Plainview Road-Deschutes River (PR-DR) and Arnold Ice Caves-Horse Ridge (AIC-HR)] by using the same asphalt cement and same mix design employed at the time of construction. A minimum of 10 specimens for each compaction level were prepared for each of the three projects. All 60 specimens were tested for resilient modulus and fatigue life. All diametral tests were run for tensile stress levels of 20 psi and 40 psi for 88% and 94% compac-

tion level, respectively.

3.4.3 Asphalts

Routine asphalt tests were accomplished by ODOT. Fraass samples were prepared and tested by OSU together with chemical composition tests in accordance with ASTM D-4124.

3.5 DETAILS OF TEST METHODS

For this aging study the POB was used to simulate oxidative aging occurring after construction. The resilient modulus and fatigue life of cores and laboratory mixture samples as well as the Fraass breaking temperature of asphalt cement were measured. The modulus ratio (the ratio of modulus after aging to the modulus before aging) and the Fraass breaking temperature were adopted to measure the changes in the properties of laboratory mixture samples and asphalt cements, respectively.

3.5.1 Aging Procedure

The modified pressure oxidation bomb consists of a cylindrical pressure vessel made of stainless steel fitted with a screw-on cover containing an ASME pressure relief for safety, a pressure gauge, and a stopcock. The pressure oxidation bomb (POB) was sealed by using O-rings. Figure 3.1 shows a diagram of the POB used for this study.

The following are the main steps in the use of the POB:

1. Samples (asphalt mixtures or asphalt cement) are prepared.
2. The samples are placed in a POB.
3. Vacuum [26 in.(66 cm) Hg.] is applied for 20 minutes.
4. The POB is filled via the stop cock from an oxygen

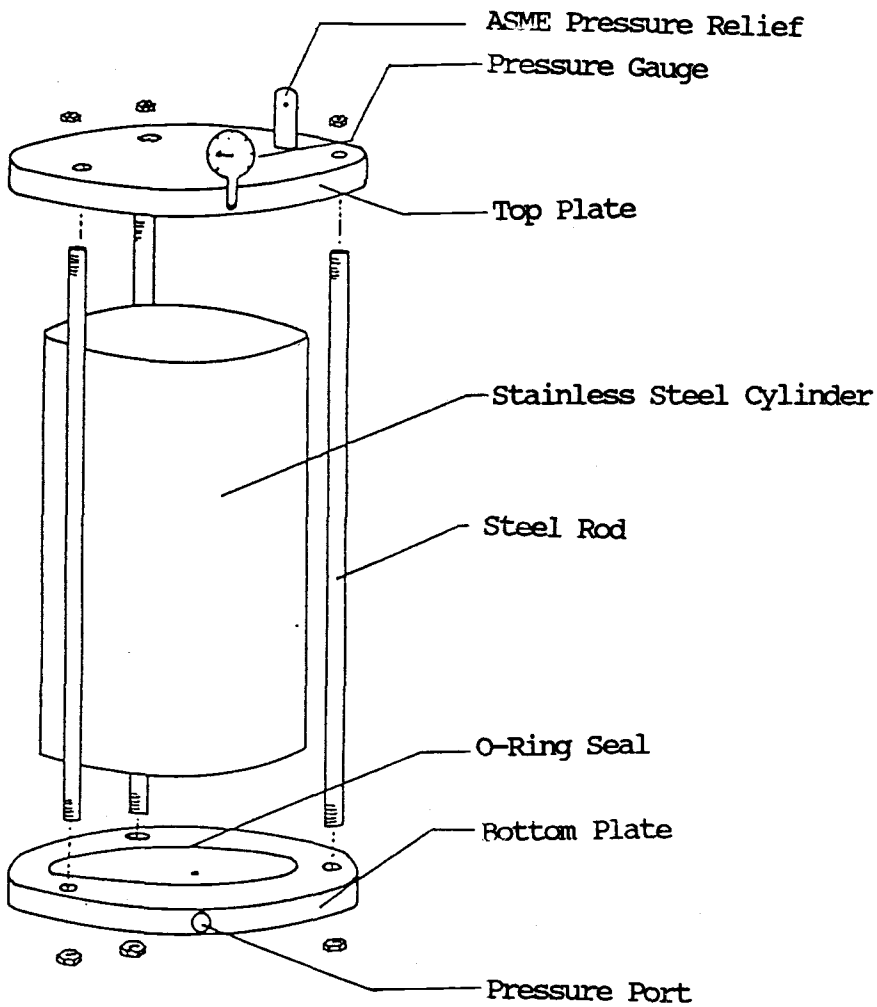


Figure 3.1 Pressure Oxidation Bomb (POB).

cylinder to 100 psi (689.5 kPa). This pressure is held for 30 minutes to ensure leak-free joints.

5. The bomb is then placed in an oven maintained at 60°C (140°F) for a period of time such as 1, 2, 3, and 5 days.
6. At the conclusion of the test the stop cock is opened, the cover is removed, and the aged mixtures and/or asphalt cement samples are cooled for one day and one hour under room temperature, respectively.

3.5.2 The Fraass Brittle Test

The Fraass breaking point (11) is the temperature at which an asphalt first becomes brittle as indicated by the appearance of cracks when a thin film of asphalt on a metal plaque is cooled at the rate of 1°C/min. and flexed at a constant rate. The apparatus and the procedure of sample preparation and test are described in Appendix and Reference 15. The following are the main steps used for this study.

1. The sample (0.4 gr) is prepared.
2. A standard steel plaque [1.6 in. x 0.8 in.(41 mm x 20 mm)] is coated uniformly with a thin layer of asphalt cement [0.02 in. (0.5 mm)].
3. The steel plaque coated with asphalt cement is in a closed chamber, and the temperature of the plaque is lowered steadily at a rate of 1°C/min (1.8°F/min), adding solid carbon dioxide to the acetone bath which surrounds the chamber where the plaque is located.
4. The steel plaque is repeatedly bent to a given extent in a standard time. The temperature at which one or more cracks

appear is recorded as the breaking point ("brittle temperature").

3.5.3 Repeated Load Diametral Test

The resilient modulus and fatigue tests were performed using the repeated load diametral test apparatus. The parameters recorded during the repeated load diametral test were the load applied, the horizontal elastic deformation, and the number of repetitions to failure. During the tests, the dynamic load duration was fixed at 0.1 sec and the load frequency at 60 cycles per minute. The static load 10 pounds (4.5 kg) was applied to hold the specimen in place. The tests were carried out at 70.7±1.6°F (21.5±0.9°C). For this study, the number of load repetitions to fatigue failure was defined as the number of repetitions required to get a vertical crack approximately 0.25 in (0.64 cm) wide in the specimens. To stop the test at the specified level of specimen deformation, a thin aluminum strip was attached to the sides of the specimen, along a plane perpendicular to the plane formed by the load platen. As the specimen deformation exceeds a certain level, the aluminum strip breaks and opens the relay, which shuts off the test. The test procedures employed are described in detail in Reference 15.

3.6 TEST RESULTS AND DISCUSSION

3.6.1 Resilient Modulus

The average moduli values of six cores of each three projects are presented in Table 3.2 including the thickness, air voids, and asphalt contents. For laboratory mixtures, two specimens were tested as-compacted and the other eight specimens were tested both before and

Table 3.2 Summary of Core Data.

Projects	Thickness (in.)	Max. Sp.Gr.	Bulk Sp.Gr.	Air Voids (%)	Asphalt Content (%)	Resilient Modulus (ksi)
Idylwood St.	1.9	2.459	2.17	11.8	5.9	772
Plainview Rd.- Deschutes River	1.4	2.497	2.29	8.3	5.8	569
Arnold Ice Caves - Horse Ridge	1.6	2.444	2.34	4.3	6.7	244

after aging for each compaction level. The aged specimens were cooled for one day before modulus was measured. The modulus test results including bulk specific gravity, air voids, and maximum specific gravity (AASHTO T-209) are summarized in Table 3.3. The aging effect assessed by the modulus ratio (the ratio of modulus after aging to the modulus before aging) is shown in Figure 3.2.

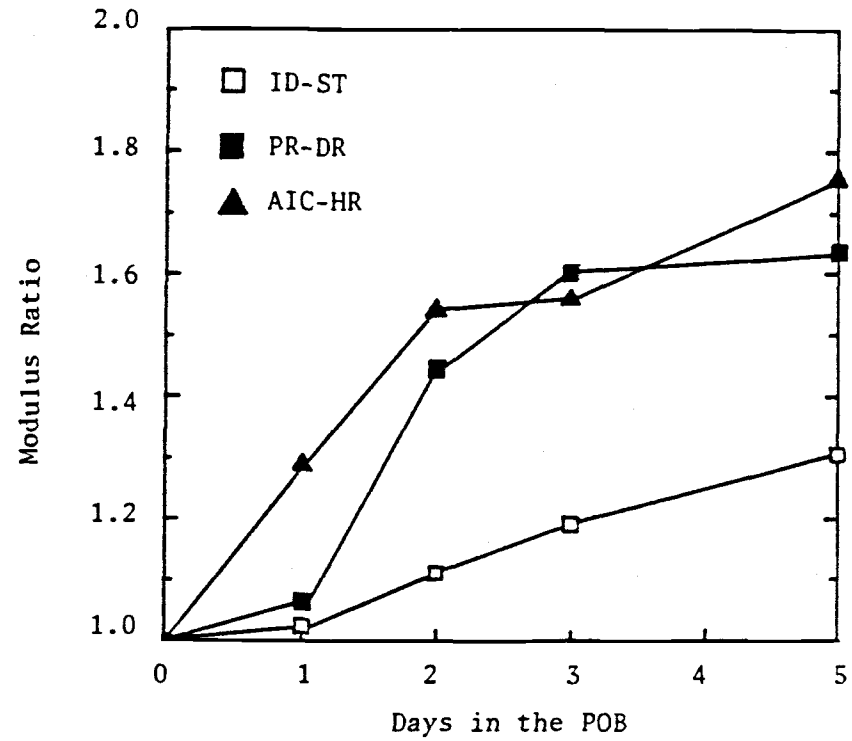
The mixtures for the Idylwood Street project (ID-ST) with 88% compaction level aged at a constant rate, while the mixtures for the other two projects aged very rapidly during the first two or three days. The aging rate (slope in Figure 3.2) of the mixtures for the Plainview Road-Deschutes River project (PR-DR) changed little after the third day while that of the Arnold Ice Caves- Horse Ridge project (AIC-HR) still increased. Even though the same grade of asphalt cement (AR-4000) was used for both the Idylwood Street and the Plainview Road-Deschutes River project the trend of mixture aging is significantly different. The difference is probably because the physical properties of the original and recovered asphalt cement were substantially different as presented in Table 3.4, i.e., the asphalts were probably from a different source.

The results from the mixtures compacted at 94 % level show a slightly different trend. Unlike the mixtures compacted at 88 % level, the mixtures with 94% compaction level for the Idylwood Street and Plainview Road-Deschutes River projects aged little during the first two days and then aged rapidly between the second and third days. This rapid aging was followed by a period of slow aging between the third and fifth days. This result may show that it takes some time for the oxygen to penetrate into mixtures with low air voids and to react with asphalt cement. If so, the permeability of mixture is an important factor in

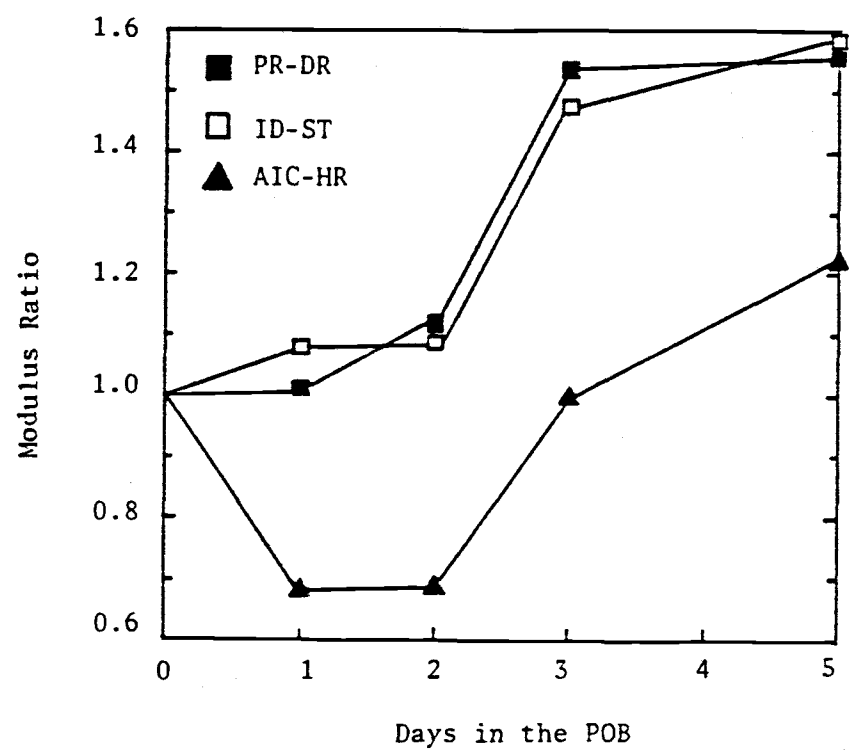
Table 3.3 Laboratory Mixture Aging Test Data.

Project	Asphalt Content (%)	Max. Sp.Gr.	Bulk Sp.Gr.	Air Voids (%)	Days	Resilient Modulus (ksi)
Idylwood	7.0	2.407	2.275*	5.5*	0	74
					1	80
					2	81
					3	110
					5	118
			2.137**	11.2**	0	52
					1	53
					2	58
					3	59
					5	68
Plainview Road-Deschutes River	5.5	2.455	2.292*	6.6*	0	237
					1	241
					2	265
					3	366
					5	373
			2.160**	12.0**	0	149
					1	158
					2	214
					3	238
					5	242
Arnold Ice Caves-Horse Ridge	6.5	2.447	2.349*	4.0*	0	105
					1	72
					2	73
					3	105
					5	128
			2.202**	10.0**	0	53
					1	69
					2	82
					3	83
					5	94

Pure Oxygen Pressure = 100 psi.
Aging Temperature = 60°C.
* : Samples prepared to approximately 94% of maximum density.
**: Samples prepared to approximately 88% of maximum density.



(a) At 88% Compaction Level



(b) At 94% Compaction Level

Figure 3.2 Aging Modulus Ratio.

Table 3.4 Physical Properties of Asphalt Cement.

	Idylwood Street			Plainview Road-Deschutes River			Arnold Ice Caves-Horse Ridge		
	O	RTFOT*	R	O	RTFOT	R	O	RTFOT	R
Penetration									
at 25 C (77 F)	139	66	10	80	46	22	140	66	63
at 4 C (39.2 F)	50		9	20		14	46		32
Penetration Ratio (4 C/25 C)	0.36		0.90	0.25		0.64	0.33		0.51
Absolute Viscosity (60 C, Poises)	1169	4306	225129	1504	3858	13584	762	2524	5542
Kinematic Viscosity (135 C, C.S.)	353	608	3952	368	494	885	236	393	745
Flash Point (Closed Cup, C)	199			252			229		
Loss on Heating (%)	1.77			0.34			0.20		

*RTFOT: After Rolling Thin Film Oven Test
O: Original
R: Recovered

the aging rate as suggested by Goode (16) and Kumar (17). It is noted that aging rate with 94% compaction level for both the Idylwood Street project and the Plainview Road-Deschutes River project are similar while the aging rate of the mixtures with 88% compaction level for these projects are significantly different. One unexpected result for the mixtures with 94% compaction level was that the specimens for the Arnold Ice Caves-Horse Ridge project appeared to soften during the first 3 days. This may have been caused by a slight loss of cohesion of the specimens at the high temperature used in the POB (60°C). However, the aging rate (slope) after the second day increases rapidly and then decreases like the other two projects.

Finally it should be noted that the modulus results from the laboratory accelerated aging procedure (POB) performed for five days under 100 psi and 60°C were not comparable with the modulus results determined from the cores of each project. Figure 3.3 shows the moduli of the laboratory aged samples for both compaction levels after 5 days, and the core moduli values. As can be seen, the core moduli values are approximately twice those of the low air void content laboratory samples except the Idylwood Street project. These results are not surprising, since laboratory compacted samples have been found to have lower moduli than field cores (18). The large difference in moduli values between cores and laboratory mixtures for the Idylwood Street project results in part from the difference of asphalt content of cores (5.8%) and laboratory mixtures (7.0%). In general, mixtures with high asphalt content show low modulus.

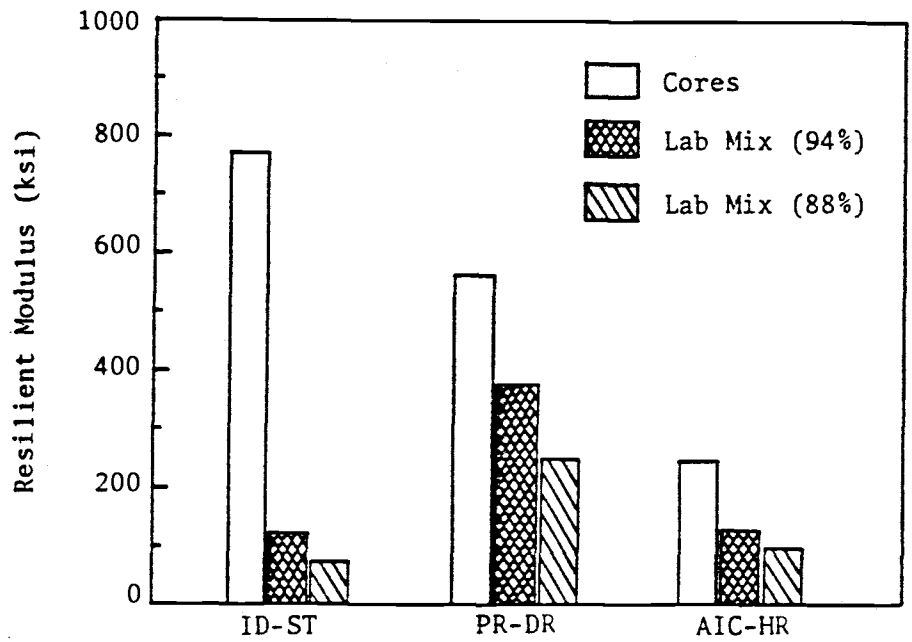


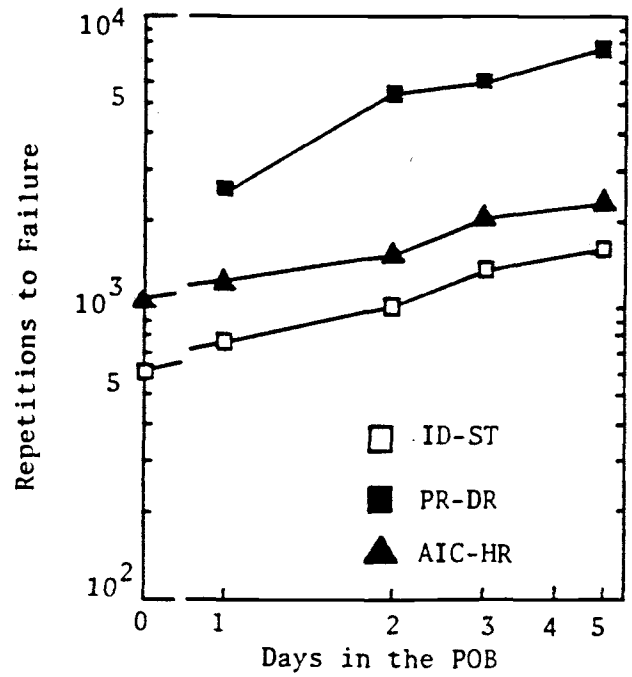
Figure 3.3 Comparison of Modulus between Cores and Aged Mixtures.

3.6.2 Fatigue Life

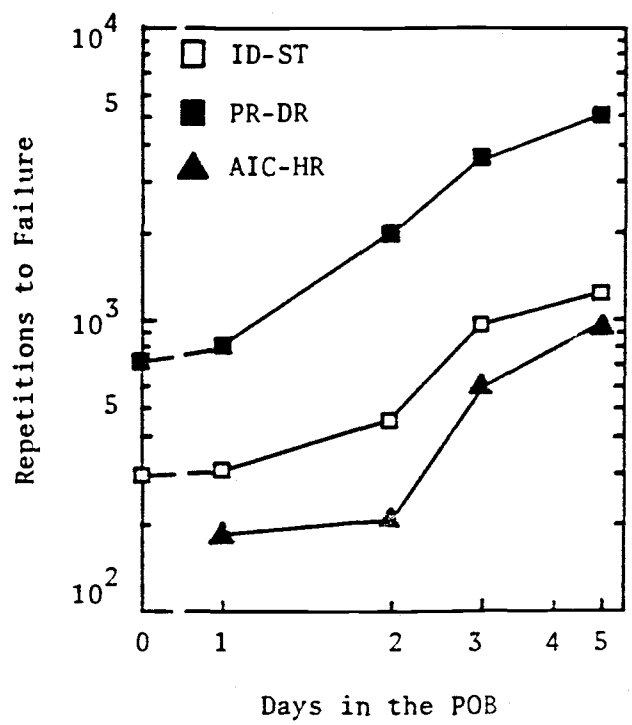
After resilient modulus was measured, the fatigue test was run at fixed tensile stress ranging from 30 psi to 60 psi for cores. For the laboratory mixtures the fatigue test was run with applying 20 psi and 40 psi of tensile stress for 88% compaction level mixtures and 94% compaction level mixtures, respectively. Only the test results of the laboratory mixtures are discussed in this paper.

Fatigue results represent the effect of the differences in modulus presented in Table 3.3 and resulted in a wide variety of fatigue performance as shown in Figure 3.4. The fatigue results show that the aged mixtures of the Plainview Road-Deschutes River project (PR-DR) obtain the longest fatigue life for both compaction levels, since this project has the highest moduli values as presented in Table 3.3. In general, the resistance to fatigue failure (slope in Figure 3.4) of mixtures decreases after the third day regardless of the difference of compaction level.

The fatigue characteristic of mixtures with different compaction levels was changed slightly with aging time. For 88% compaction level, the slopes of each project are relatively constant through time, while slopes of the mixtures compacted at 94% level changed in each time interval. These results show that the air voids of mixtures affect the resistance to fatigue failure through their service period, and that high modulus is an important factor in achieving long fatigue life. The changes of fatigue life of the Idylwood Street and the Arnold Ice Caves-Horse Ridge projects increase slightly during the first two days. This result can be explained by the time for oxygen to penetrate into the mixture as discussed in the modulus section, since the actual air voids



(a) At 88% Compaction Level



(b) At 94% compaction Level

Figure 3.4 Fatigue Life of Aged Specimens.

of the Idylwood Street (5.5%) and the Arnold Ice Caves-Horse Ridge project (4.0%) are much lower than that of Plainview Road-Deschutes River project (6.6%). The rate of change of fatigue life of each project at both compaction levels (except the Plainview Road-Deschutes River project at 88% compaction level) decreases after the third day, that is, the resistance to fatigue failure of mixtures decreases with oxidative aging time.

3.6.3 Fraass Breaking Temperature

The trends of increased Fraass breaking temperature (i.e., more brittle asphalt) for each project (Figure 3.5) are similar to those of increased modulus ratio of mixtures with 88% compaction level (Figure 3. 2). For the Idylwood Street project (ID-ST) the breaking temperature of the asphalt cement (AR- 4000) increases very slowly as the aging period increases. For the Plainview Road-Deschutes River project (PR-DR) which used the same grade of asphalt cement (AR-4000) the breaking temperature of asphalt cement increases much more rapidly during the first two days aging. The original asphalt cement (120/150 Pen.) used for the Arnold Ice Caves-Horse Ridge project (AIC-HR) has almost the same breaking temperature and a similar aging rate as that for the Idylwood Street project. Also, these asphalt cements have almost the same physical properties except for flash point and loss on heating as presented in Table 3.4. Again, it can be seen that even though asphalt cements may have the same grade, they show significant differences in behavior such as for the AR-4000 asphalts used for the Idylwood Street and Plainview RoadDeschutes River projects.

The change in Fraass breaking temperature of an asphalt cement

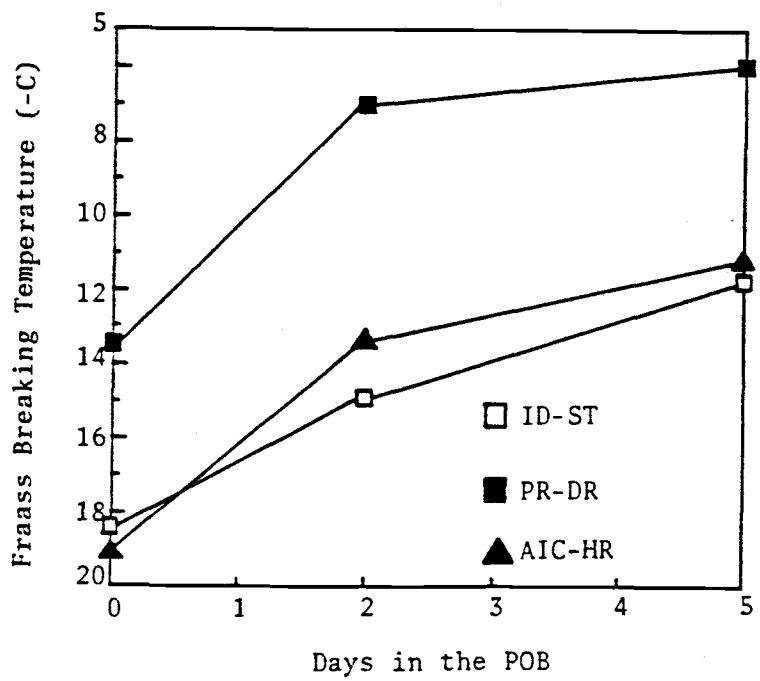
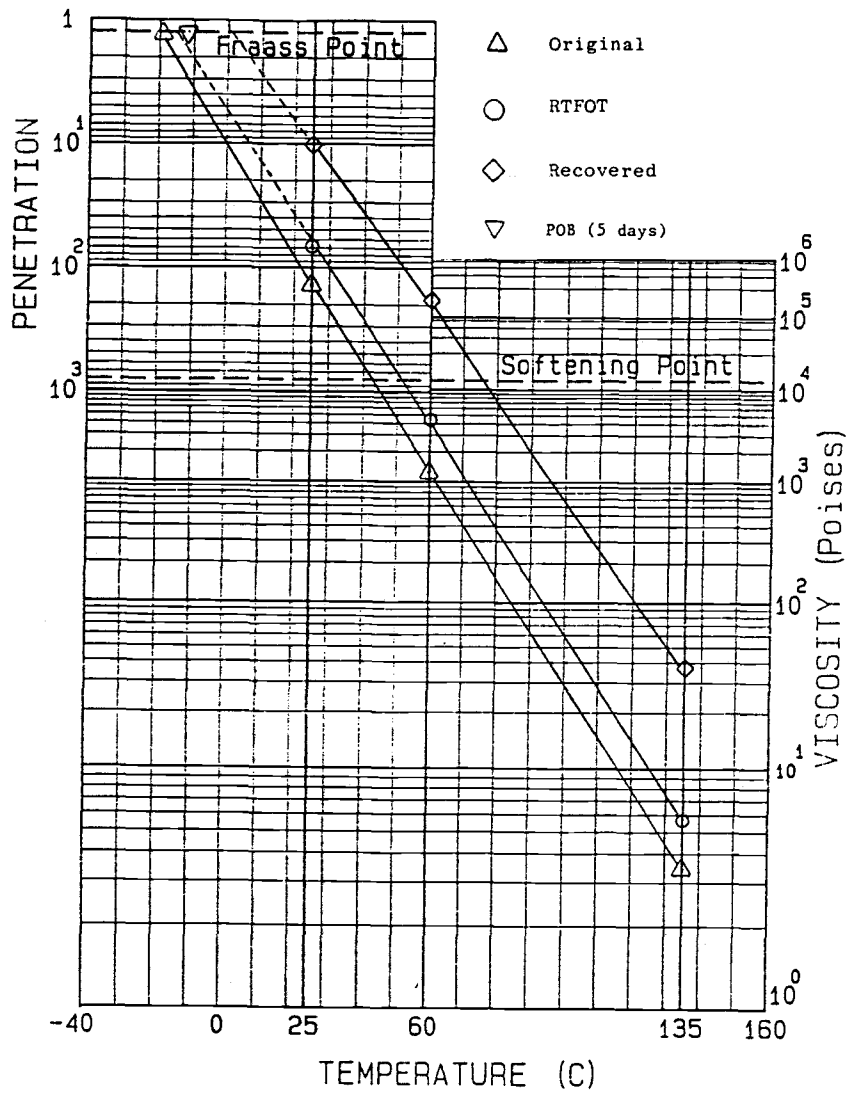
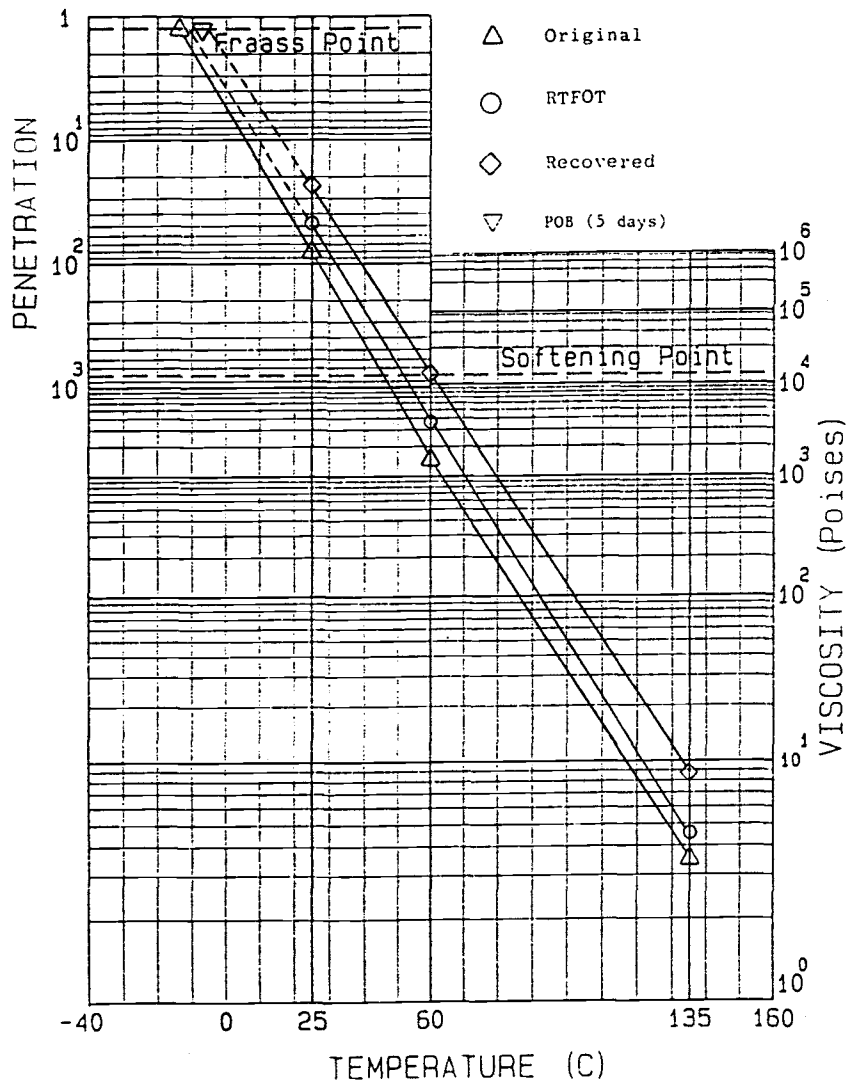


Figure 3.5 Effect of POB Aging on Fraass Temperature of Asphalt Cement.



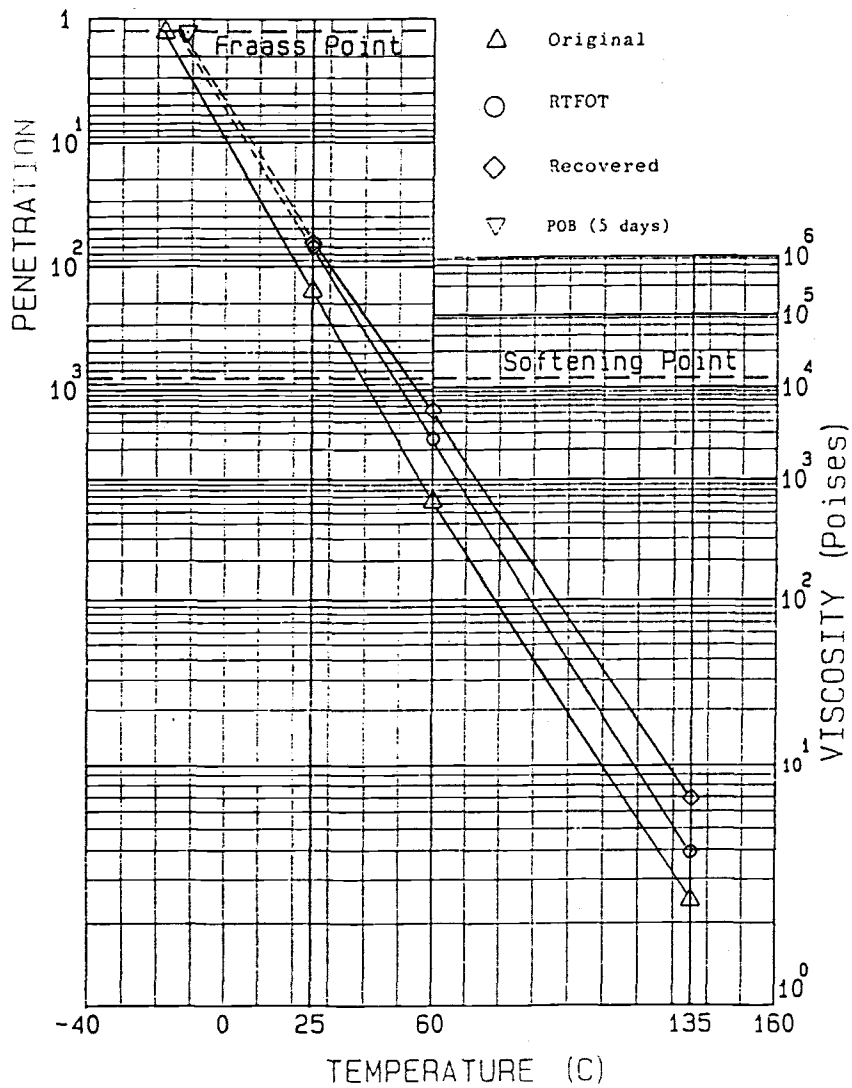
(a) Idylwood Street

Figure 3.6 Asphalt Consistency Data.



(b) Plainview Road-Deschutes River

Figure 3.6 Asphalt Consistency Data (Continued).



(c) Arnold Ice Caves-Horse Ridge

Figure 3.6 Asphalt Consistency Data (Continued).

indicates a general change in the consistency properties of the asphalt cement which can be estimated from a Bituminous Test Data Chart (BTDC, (19)) as shown in Figure 3.6 for each project. If the temperature susceptibility of asphalt cement changed little after aging (i.e. if the lines drawn through the original and aged property data were parallel), it might be possible to predict the long term asphalt properties with the one point of Fraass breaking temperature of asphalt aged in the POB. Hence, the Fraass breaking temperature may be a valuable indicator of the durability of asphalt cement as well as defining a low temperature consistency.

The rolling thin film oven test (RTFOT, ASTM D-2872) is used to measure the anticipated hardening of the asphalt during hot-mix plant operations in several western States, and therefore will probably not give an indication of hardening due to long term aging. The consistency data for asphalt from each project after RTFOT (Figure 3.6) are not adequate to predict the long term asphalt properties due to oxidation. The limited data shown in Figure 3.6 illustrate that asphalt recovered from field cores had higher consistencies to RTFOT aged asphalt, and that POB aging (5 days) may be similar to field aged materials with regard to the Fraass point. Clearly more data is required to support this suggestion. For the Idylwood Street project the Fraass breaking temperature lies between the projected line (dashed line in Figure 3.6) of RTFOT and recovered as shown in Figure 3.6 (a), that is, the asphalt cement aged in the POB for 5 days was aged more than the asphalt from the RTFOT and aged less than that of cores. However, for the Plainview Road-Deschutes River project the Fraass breaking temperature of asphalt aged in the POB for 5 days lies close on the projected line of recovered

asphalt in Figure 3.6 (b). The result of the Arnold Ice Caves-Horse Ridge project is similar to that of the Plainview Road-Deschutes River project as shown in Figure 3.6 (c).

3.6.4 Effectiveness of POB

In general, a major cause of asphalt cement hardening is oxidation, a process that occurs most readily at high temperature and with thin asphalt films. The POB developed in England (11) was modified to age both asphalt cements and asphalt mixtures for the aging study at OSU. The modified POB was used with 100 psi and 60°C. Previously, higher pressures and temperatures were used (11, 12), but the lower levels were adopted because of safety considerations and preservation of the shape of the mixture samples. The modulus ratios obtained from original mixtures and weathered mixtures after four years aging at four different weathering sites in California (8) are presented in Table 3.5. The modulus ratio ranges from 0.67 to 1.67, excluding the high air voids (7 to 12%) mixture using Santa Maria asphalt and non absorbent aggregate. Even though the moduli values of aged laboratory mixtures were substantially different from those of cores tested in the study reported herein, it can be seen that the POB causes similar changes in modulus, i.e., results in similar modulus ratios (Figure 3.2) to those observed in the California study (Table 3.5). The results of asphalt cement chemical composition tests (20) done in cooperation with this aging study, indicate that the use of the POB to age the asphalt cement on Fraass plaques (0.5-mm film thickness) produced a similar chemical composition to that of asphalt extracted from the cores. The results presented in Table 3.6 show that asphalt cement aged in the POB for 5

Table 3.5 Modulus Ratio of Field Weathering Mixtures

(Original and 4 Year Field Weathering, After Ref. 8).

(a) Non Absorbent Aggregate

Asphalt Source	Air Voids (%)	Original*	Weathering Site ** A	B	C	D
Valley	3 - 5	950	0.86	0.76	1.16	0.81
	7 - 9	800	0.85	0.84	1.05	0.82
	10 -12	1000	0.73	0.76	0.81	0.62
LA Basin	7 - 9	740	0.78	0.73	1.11	0.96
Santa Maria	3 - 5	330	1.21	1.21	1.24	1.67
	7 - 9	160	2.31	2.13	2.44	3.13
	10 -12	150	1.93	2.47	2.13	3.40

(b) Absorbent Aggregate

Asphalt Source	Air Voids (%)	Original*	Weathering Site ** A	B	C	D
Valley	3 - 5	810	0.79	0.80	1.12	0.86
	7 - 9	590	1.09	0.83	1.20	1.07
	10 -12	730	0.92	0.70	1.11	0.95
Santa Maria	3 - 5	430	1.28	1.00	1.28	1.61
	10 -12	270	1.37	1.44	1.63	2.07

Original*: Resilient Modulus of Original Mixture, ksi.

Weathering Site**: A; Fort Bragg, B; Sacramento, C; So. Lake Tahoe, D; Indio.

Table 3.6 Chemical Composition of Asphalt Cement.

	Idylwood Street				Plainview Road-Deschutes River				Arnold Ice Caves-Horse Ridge			
	Ø	2 days*	5 days**	R	Ø	2 days	5 days	R	Ø	2 days	5 days	R
Asphaltenes	22.7	27.2	29.0	37.3	16.1	22.4	24.3	25.3	24.9	25.6	27.8	28.0
Saturates	8.4	8.2	7.2	5.6	8.6	7.6	7.6	6.6	10.2	10.3	9.8	10.8
Naphthene-Aromatic	24.5	23.0	23.4	24.7	26.8	25.8	25.1	25.8	25.5	27.4	26.8	19.7
Polar Aromatics	43.3	39.4	37.9	32.4	47.0	42.6	40.8	41.8	38.1	36.0	33.7	39.7
Total	98.9	97.8	97.5	100.0	98.5	98.4	97.7	99.5	98.7	99.3	98.1	98.2

*aged with POB for 2 days
**aged with POB for 5 days
O: Original
R: Recovered

days can have a similar composition to that from the cores. However, as with the consistency data, there is a significant difference in the POB and recovered properties for the Idylwood Street project. The component fractions of asphalt cement samples (aged for 5 days) from the Plainview Road-Deschutes River project were very similar to those obtained for asphalt extracted from the cores. Also, for the Arnold Ice Caves-Horse Ridge project the fractions of asphaltenes and saturates of the asphalt cement aged for 5 days in the POB are close to those of the cores. The chemical composition of the aged asphalt (for 5 days) of the Idylwood Street project shows that the asphalt in the POB was less aged than that of cores as discussed in the previous section. The equivalent aging period in the laboratory to a field may vary with grade and source of asphalt cements as well as mixture properties (particularly air voids and asphalt film thickness) and environmental conditions.

The POB can be used effectively to age asphalt cements in thin films with high pressure and/or temperature to give an indication of the long term asphalt properties with one point of Fraass breaking temperature on the BTDC. However, to effectively evaluate mixture aging, representative mixtures must be tested, and if possible these should be core samples obtained shortly after construction, rather than laboratory compacted mixtures which as shown in this study and others (21, 22) do not represent field mixtures. There may be a large difference between the laboratory mixtures and plant mixtures due to the differences in production method and differences in the compaction method. Reference 11 indicates that one day's aging of asphalts in the POB at a pressure of 20 atmospheres (300 psi) and at temperatures of 50 to 60°C is equivalent to half a year on the road in Holland. However, it is extremely

important to fit an appropriate pressure relief device to the POB when working at these elevated levels. In this study only 100 psi at 60°C was applied after consideration of safety concerns regarding the use of pressurized oxygen and in order to preserve the shape of the mixtures.

3.7 SUMMARY

For the limited number of projects studied, the pressure oxidation bomb (POB) was found to be a very effective device for oxidative aging of both asphalt cements and asphalt mixtures. The test results show that with the exception of one project, and in that case only for modulus, all the properties changed by at least 50 percent after 5 days of POB aging. Higher pressure, temperature and/or longer exposure time could be applied to accelerate the aging, but the user must be aware of safe operating procedures. The modulus ratio and the Fraass breaking point are good indicators to measure the aging rate of mixtures and asphalt cement, respectively. The aging rate of mixtures varies with the air voids of mixtures. The mixtures with higher air voids are aged more rapidly, although the aging rate depends on asphalt properties.

The limited testing done in this study showed that asphalts aged in the POB for 5 days had similar composition to those recovered from field cores of five to ten years old. This lends some confidence to the use of the Fraass samples for oxidative aging procedures.

3.8 RECOMMENDATIONS FOR FUTURE WORK

Although not evaluated in this study, the POB approach for oxygen conditioning could be combined with a Lottman moisture conditioning approach to approximate very harsh environmental conditions. There is

some evidence to suggest that mixtures suffering oxidative aging are moisture susceptible (1) and therefore subjecting a mix to cycles of oxidation and moisture conditioning with modulus and fatigue tests are very appropriate. Clearly, the data collected in this study were insufficient to fully evaluate the aging procedures used. More data should be collected, therefore, to improve confidence in their effectiveness.

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DISCLAIMER

The contents of this paper reflects the views of the authors who are responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the official views or polices of either the Oregon State Highway Division or Federal Highway Administration.

APPENDIX

Sample Preparation and Testing Procedure of Fraass Brittle Test

Figure A gives a general description of the component parts of the Fraass apparatus. The following describes the sample preparation procedure and test procedure. Throughout these procedures temperature and preparation times should be controlled very carefully.

A. Sample Preparation

The following are the steps used for the sample preparation:

1. Heat the asphalt cement until it has become fluid enough to pour from the container in which it is supplied. The temperature required to heat the asphalt cement to this consistency should correspond to a viscosity of 100 ± 10 Poises obtained from a plot of viscosity versus temperature, such as a BTDC. Do not heat the container for more than one hour.
2. Pour the asphalt on waxed paper and spread as thin as possible. Cool the asphalt at 10°C for one hour.
3. Place an amount of the asphalt corresponding to 0.40 ± 0.01 gr. in the solid state on a standard Fraass steel plaque (41 mm x 20 mm) of known tare weight. For each test, prepare at least eight plaques. This process is done at temperature below 10°C, such that the asphalt cement can be cut with a razor.
4. After placing the sample on a steel plaque, heat the plaque on the baffle plate (Figure B) cautiously. The temperature on the heating plate at the edge is the temperature used in step 1. Before heating, the level of the heating plate should be

checked and maintained.

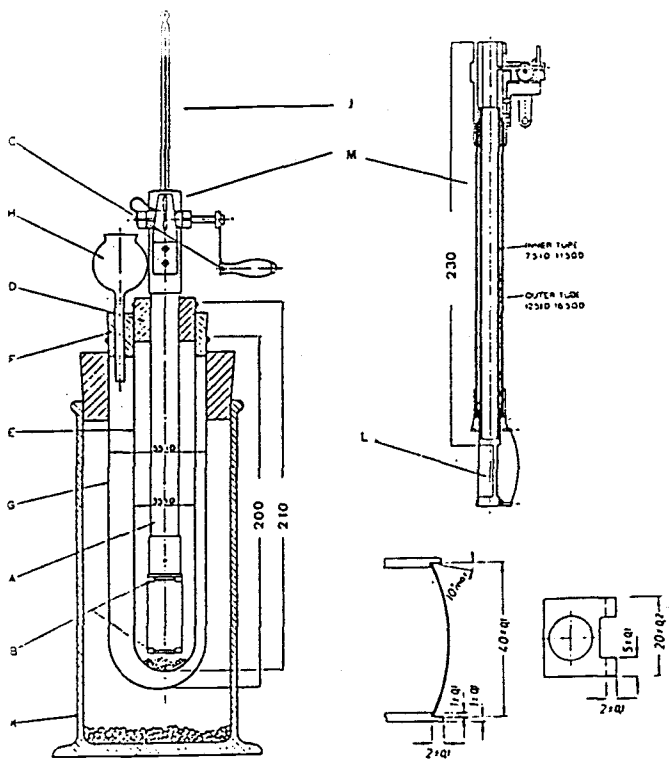
5. Place no more than three plaques around the edge (Figure B) of the heating plate. Using a very thin needle, after the sample has softened, induce the sample towards the edge of the plaque by breaking the surface tension. This step should be done as soon as possible (within 3 minutes).
6. Cool the plaques at room temperature for about one hour then place in a refrigerator until executing the Fraass test.

B. Testing Procedure

The steps of testing are the following:

1. Check the bending apparatus and the distance between the plaque hinges. The distance should be 40 ± 0.1 mm as shown in Figure A.
2. Place a small quantity of calcium chloride or anhydrone in the test tube E and the cylinder K (Figure A).
3. Assemble the test tubes as shown in Figure A, and fill the annular space between E and G to about half its height with acetone.
4. Attach a thermistor behind the plaque and place the plaque between the clips of the bending apparatus, bending the plaque gently to do so, and mount the bending apparatus in the tube E.
5. Add solid carbon dioxide to the acetone at such rate that the temperature falls at rate of 1°C per minute.
6. When the temperature reaches 0°C, start to turn the handle "C" at a rate of one revolution per second for 11 or 12 turns and then unwind the handle at the same rate.

7. Repeat step 6 until one or more cracks appear on the sample. Record the temperature at this point as the Fraass breaking point.



(dimensions in mm)

KEY

A: Concentric cylinder

B: Plaque hinges

C: Rotating handle

D: Supporter rubber bung. Seals chamber one.

E: Test-tube. Air chamber one. Contains small quantity of calcium chloride to absorb moisture.

F: Supported rubber bung

G: Test tube containing acetone

H: Funnel

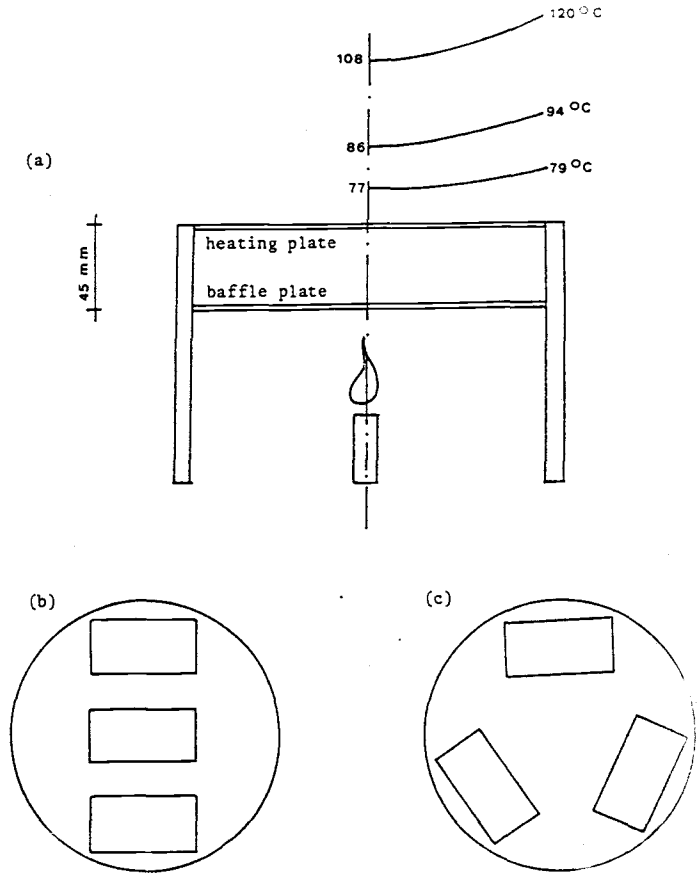
J: Mercury thermometer

K: Glass cylinder. Air chamber two contains small quantity of calcium chloride to absorb moisture.

L: Location of the thermometer bulb

M: Bending apparatus

Figure A. Schematic of Fraass Apparatus (After Ref. 16).



(a) Temperature Gradients on Plate for Three Different Flame Intensities.

(b) Plaque Positioned the Wrong Way.

(c) Better Way of Placing Plaques during Preparation.

Figure B. Schematic of Fraass Sample Preparation (After Ref. 16).

4.0 MEASUREMENT AND ANALYSIS OF TRUCK TIRE PRESSURES IN OREGON

Ok-Kee Kim
Graduate Research Assistant
Department of Civil Engineering
Oregon State University

Chris A. Bell
Associate Professor of Civil Engineering
Oregon State University

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August 1987

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ABSTRACT

As axle loads have increased, the use of higher tire pressures has become more popular in the truck market and radial tires are predominantly used. In order to collect data on tire pressures and types of tires in use, a survey was carried out at a weigh station located near Woodburn, Oregon on Interstate 5 during the summer of 1986.

The data show that 87% of the tires surveyed are of radial construction. The average measured pressures of radial and bias tires are 102 psi and 82 psi, respectively. The survey results show that the difference between the manufacturer's maximum recommended tire pressure and the measured tire pressure is very small, particularly for radial tires. Therefore, if government agencies wish to control tire pressures, it would be expedient to control the manufacturer's maximum recommended pressure rather than the inflation pressure used by truckers. This would ensure reasonable control, since the data collected in this study show that measured and recommended tire pressures are nearly equal.

The survey results show that the size of most tires is 11 in. wide with a rim diameter of 24.5 in. (i.e., 11/80 R 24.5 or 11-24.5) and the average tread depth of radial tires is slightly greater than that of bias tires.

4.1 INTRODUCTION

4.1.1 Problem Statement

The economics of truck transportation have tended to cause the average gross weight of trucks to increase such that the majority of trucks are operating close to the legal gross loads or axle loads (1). Many states, including Oregon (2), also issue permits for trucks to operate above normal legal load limits. As axle loads have increased, the use of radial tires with higher pressures has become more popular in the truck market to support the increased axle loads.

Higher tire inflation pressures decrease the contact area, resulting in reduced tire friction or skid resistance and increased potential for pavement damage under the high stress. The higher tire pressures contribute to greater deformation in flexible pavements, manifested as high severity wheel track rutting. Rutting results in hazardous pavements, since ruts create an uneven pavement where water and ice can accumulate in harsh weather. The higher tire pressures also tend to be accompanied by higher loads, and these will tend to increase the severity of fatigue cracking.

4.1.2 Objectives

In order to determine the levels of tire pressures in use, a survey was carried out at a weigh station located on Interstate 5 during the summer of 1986 by Oregon Department of Transportation (ODOT) and Oregon State University (OSU).

This paper presents a part of the study on procedures for controlling the effect of increased tire pressure of trucks on asphalt pavement damage (3) performed by the ODOT and OSU.

The objectives of this paper are to (a) present existing operating characteristics of Oregon's trucks, particularly levels of tire pressures and tire sizes, and (b) analyze the data surveyed.

4.2 BACKGROUND

Economic incentives that often exceed the expected costs of overweighting to the trucker are a major reason for increasing the cargo weight of trucks. The benefit to a trucker from increasing the load capacity of a truck is increased financial returns.

Table 4.1 indicates that the cash incentive to load 80,000 lb referring to 73,000 lb is $180 and the incentive increases as cargo weight increases (4). This results from decreasing costs per ton-mile as cargo weight increases. Figure 4.1 shows how costs per ton-mile decrease dramatically and costs per mile increase only slightly as the weight of the load increases. While the cost per mile increases only 1.5%, as the weight increases from 10 to 25 tons, the cost per ton-mile decreases 60%. Since fuel cost per mile traveled does not vary proportionately with the weights of trucks, as shown in the Mississippi and Oregon studies (1), the more a truck is loaded the greater financial benefit results.

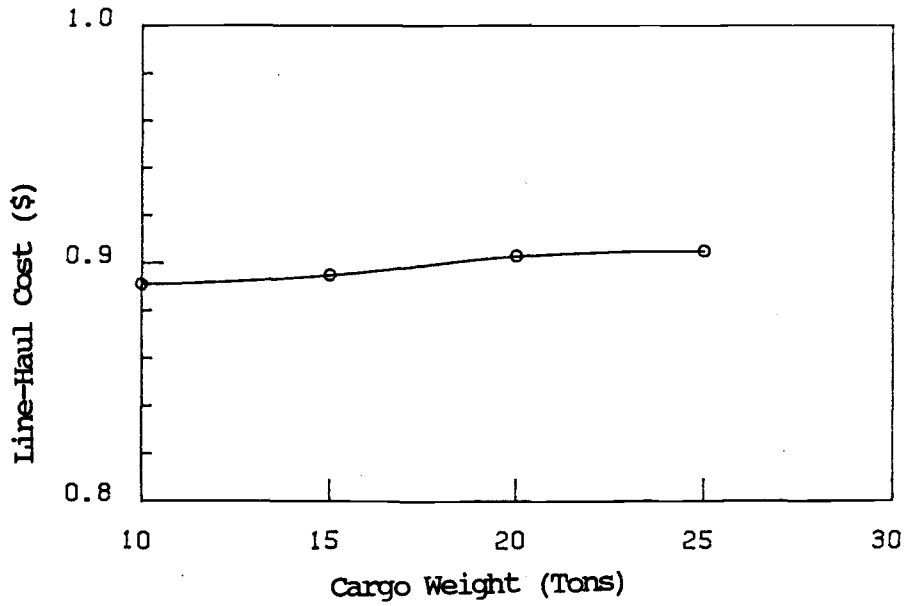
Consequently, the economics of long haul truck transportation has tended to cause the average gross weight of trucks to increase such that the majority of trucks are operating close to or above the legal gross loads or axle loads. In 1982, the federal goverment permitted 80,000 pounds gross vehicle weight, 20,000 pounds single axle weights and 34,000 pounds tandem axle weights on interstate highways.

As axle loads have increased, there has been a trend to use of the

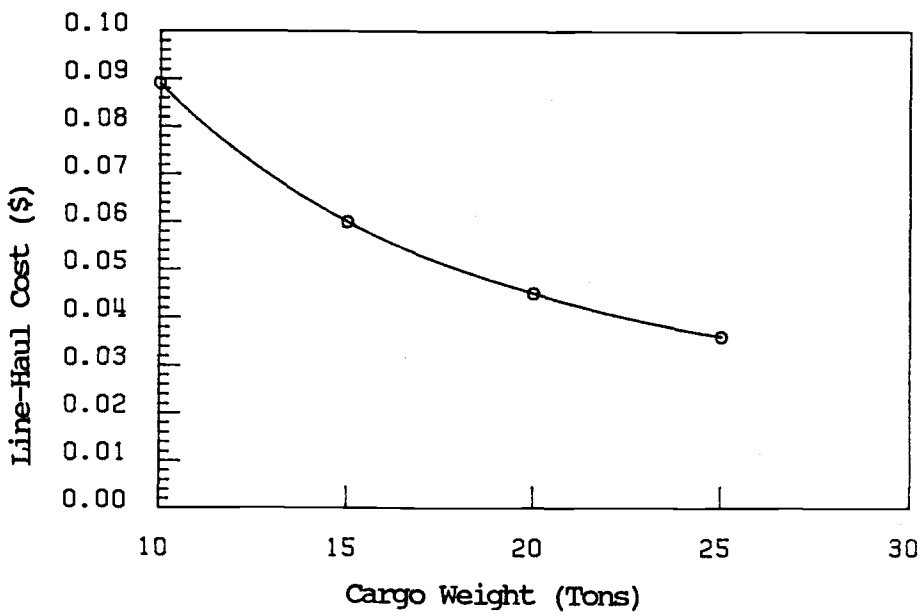
Table 4.1 Incremental Incentives to Overweight (After Ref. 4).

Vehicle Weight (lb)	Cargo Weight (lb)	Rate per Pound* ($)	Resulting Rate ($)	Incentive ($)
73,000	45,000	0.056	2,520	0
75,000	47,000	0.054	2,540	20
80,000	52,000	0.052	2,700	180
90,000	62,000	0.050	3,100	580
100,000	72,000	0.048	3,460	940

* A typical rate is $0.056; the decreases in rate per pound are given in an attempt to account for the rate reduction that might be offered by a trucker planning to overweight.



(a) Per Mile



(b) Per Ton-Mile

Figure 4.1 Cargo Weight vs. Line Haul Cost (After Ref. 4).

increasing tire pressures, due in part to attempts to decrease the contact area between the tire and the pavement, resulting in reduced rolling resistance of vehicles and, therefore, fuel consumption.

Recent study in Texas (5) indicates that trucks typically operate with tire pressures of about 100 psi in that state. The study performed by Roberts and Rosson (6) indicated that the resulting contact pressure between the tire-pavement for a bias tire with an inflation pressure of 125 psi could be as high as 200 psi. This study showed that for legal axle loads, increasing the tire pressures from 75 to 125 psi for a bias ply tire (10.00-20) can cut the life of the typical thin asphalt concrete pavements of Texas by amounts ranging from 30 to 80%.

From the point of view of asphalt pavement design and rehabilitation strategies (particularly overlay design) the increased tire inflation pressures and the axle load configuration are both very important factors to be considered. Adequate consideration of current levels of these factors could result in the refinement of paving mix design, and pavement structure design methods as well as the adjustment of highway user cost. Also, it is necessary to review the maintenance schedules and the remaining life of the existing pavements constructed on the basis of truck tire pressures of about 80 psi.

4.3 RESULTS

A survey to evaluate tire inflation pressures and types of tires in use was carried out at a weigh station located near Woodburn, Oregon on Interstate 5 from July 28 to July 30 and from August 25 to August 31 in 1986. The data for each truck took about 15 to 30 minutes for two or three inspectors to collect, depending on the truck type. The survey

was performed day and night for the survey period mentioned above.

A tire pressure data collection sheet is shown in Figure 4.2. One data collection form represents one truck. The data collection form consists of four parts, as follows:

1) Basic data: date, time (start time and finish time), Public Utility Commission (PUC) safety inspection number, inspector, PUC plate number, and commodity.
2) Weather information, including air temperature and pavement temperature.
3) Truck classification used in Oregon's Weigh-In-Motion study.
4) Tire data: axle number, dual/single tire, tire manufacturer, tire construction (radial/bias), tire size, tread depth, and tire pressure (recommended maximum pressure (cold) by manufacturer, 1st and 2nd measured tire pressure).

The tire manufacturer, tire construction and tire size were read from the tire. As Middleton et al. (5) did in Texas, tire pressure was measured twice. The 1st measured pressure was the inflation pressure measured after the truck was stopped. The 2nd pressure measurement was the last step of collecting the data. Therefore, a time interval between the 1st and the 2nd measurement was 15 to 30 minutes. The reason for the 2nd measurement was to determine whether or not a change in pressure occurred as the tires cooled down.

4.3.1 Truck Types

The total of 270 trucks were classified as shown in Figure 4.2. As presented in Table 4.2, 55.9% were 3-S2 (18-wheelers), 7.4% single unit trucks, and 13% trucks with tractors, semitrailers, and trailers.

Curtis A. Bell
Oregon State University
Dept. of Civil Engineering
Corvallis, OR 97331
503/754-4273

Research Section: 503/378-2518
Permits and Weighmasters: 503/378-2568

TIRE PRESSURE DATA COLLECTION SHEET

BASIC DATA: Test No. (no entry required): _____ Date: _____ Start Time: _____

PUC Safety Inspection No.: _____ Place of Inspection: _____ Inspector: _____

PUC Plate No.: _____ Commodity: _____ Comments: _____

WEATHER: (tick one)

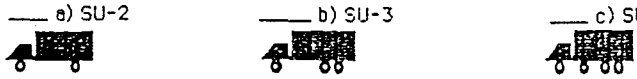
a) Hot & Sunny ___; b) Cool & Sunny ___; c) Hot & Cloudy ___; d) Cool & Cloudy ___; e) Intermittent Showers ___; f) Frequent Showers ___; g) Persistent Rain ___

*Air Temperature ____°F *Pavement Temperature ____°F *Record immediately after start time

TRUCK CLASSIFICATION: (tick one)

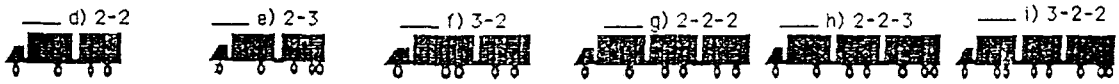
A. Single Units:

___ a) SU-2 ___ b) SU-3 ___ c) SU-4



B. Trucks & Trailers:

___ d) 2-2 ___ e) 2-3 ___ f) 3-2 ___ g) 2-2-2 ___ h) 2-2-3 ___ i) 3-2-2

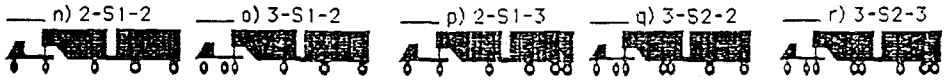


C. Tractors & Semitrailers

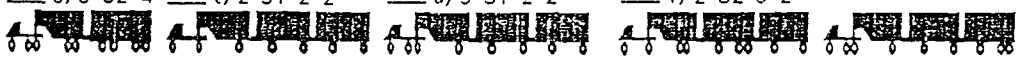
___ j) 2-S1 ___ k) 3-S1 ___ l) 2-S2 ___ m) 3-S2

D. Tractors, Semitrailers & Trailers:

___ n) 2-S1-2 ___ o) 3-S1-2 ___ p) 2-S1-3 ___ q) 3-S2-2 ___ r) 3-S2-3



___ s) 3-S2-4 ___ t) 2-S1-2-2 ___ u) 3-S1-2-2 ___ v) 2-S2-3-2 ___ w) 3-S1-2-3



TIRE DATA:

A. Left Side - Outer Tires

Axle #	Twin/ Single Tire	Name	Mfr. Rec/Max Pressure (psi)	Rad/ Bias (R/B)	Size	Pressure (psi) 1st*/2nd**	Tread Depth†
(1)							
(strg)							
(2)							
(3)							
(4)							
(5)							
(6)							
(7)							
(8)							
(9)							

B. Right Side - Outer Tires

Axle #	Twin/ Single Tire	Name	Mfr. Rec/Max Pressure (psi)	Rad/ Bias (R/B)	Size	Pressure (psi) 1st*/2nd**	Tread Depth†
(1)							
(strg)							
(2)							
(3)							
(4)							
(5)							
(6)							
(7)							
(8)							
(9)							

*measured at beginning of inspection; ** measured at end of inspection; †1/32nd in.

Finish time: _____

Figure 4.2 Tire Pressure Data Collection Sheet.

Table 4.2 Number of Trucks in the Sample.

Truck Type		Frequency (Number)	Frequency (%)
Single Units	SU-2	11	4.1
	SU-3	9	3.3
Trucks and Trailers	2-3	2	0.7
	3-2	16	5.9
	3-3	4	1.5
	3-4	3	1.1
	4-4	1	0.4
Tractors and Semi-Trailers	2-S1	12	4.4
	3-S1	3	1.1
	2-S2	11	4.1
	3-S2	151	55.9
	4-S2	1	0.4
	2-S3	1	0.4
	3-S3	1	0.4
Tractors Semi-trailers and Trailers	2-S1-2	10	3.7
	3-S1-2	11	4.1
	3-S2-2	3	1.1
	3-S2-3	3	1.1
	3-S2-4	1	0.4
	2-S1-2-2	4	1.5
	3-S1-2-2	2	0.7
	2-S1-2-1	1	0.4
Unknown		9	3.3
TOTAL		270	100.0

4.3.2 Tire Construction

The tires surveyed were divided into three groups:

1) single tires used for steering axles,
2) single tires for non-steering axles, and
3) dual tires for non-steering axles.

As presented in Table 4.3 the data collected show that the majority of tires are radials, i.e., 87% of total tires (total is 2704 tires). The radial tires for steering axle is of 91% , which is the greatest percent among the three groups.

4.3.3 Recommended Maximum Tire Pressure

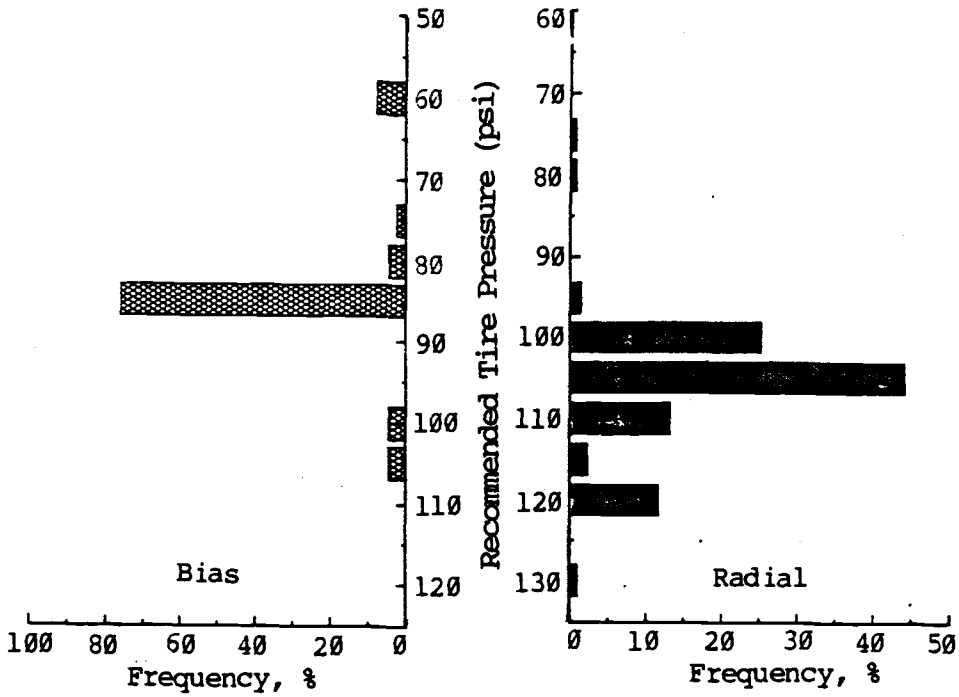
Figure 4.3 shows the distribution of the recommended maximum tire pressure (cold) by manufacturers for three groups of radial and bias tires and Table 4.3 presents the mean value and one standard deviation. The average of recommended maximum pressure for dual radial and bias tires is 101 psi and 81 psi, respectively.

4.3.4 Measured Tire Pressure

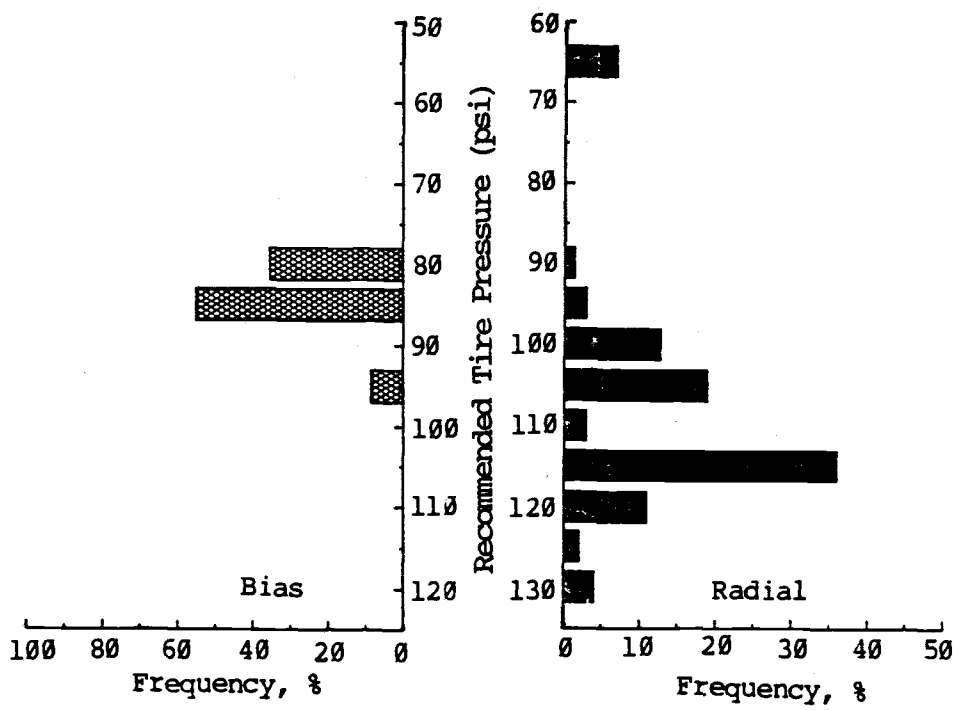
Figure 4.4 shows the distribution of the 1st measured tire pressure for three groups of radial and bias tires. Table 4.3 presents the mean value and one standard deviation of the 1st measured tire pressure and the difference between the 1st and the 2nd measured tire pressure. The average of the 1st measured pressure of dual radial and bias tires are 102 and 82 psi, respectively. The 1st measured tire pressures are slightly higher by 1.2 to 2.4 psi than the 2nd measured pressures.

Table 4.3 Results of Tire Survey.

	Single Tire for Steering Axle		Single Tire for Non-Steering Axle		Dual Tire for Non-Steering Axle	
	Radial	Bias	Radial	Bias	Radial	Bias
A. Tire Construction						
Sample Number	499	46	91	11	1755	292
Sample Frequency, %	91.5	8.5	89.2	10.8	85.7	14.3
B. Recommended Tire Pressure						
Mean (psi)	106	84	108	84	101	81
One Standard Deviation (psi)	7	9	14	4	8	8
Sample Number	495	46	89	11	1735	285
C. 1st Measured Tire Pressure						
Mean (psi)	106	86	107	93	102	82
One Standard Deviation (psi)	10	17	15	10	12	15
Sample Number	498	46	91	11	1755	292
D. (1st Measurement)-(2nd Measurement)						
Mean (psi)	1.5	2.4	1.6	1.5	1.2	1.6
One Standard Deviation (psi)	2.3	2	1.9	0.7	3	2.7
Sample Number	316	18	66	2	1064	202
E. Tread Depth (1/32-in.)						
Mean	13	11	12	12	11	9
One Standard Deviation	3.4	3.7	4.3	3.7	4.9	3.4
Sample Number	496	46	88	11	746	287

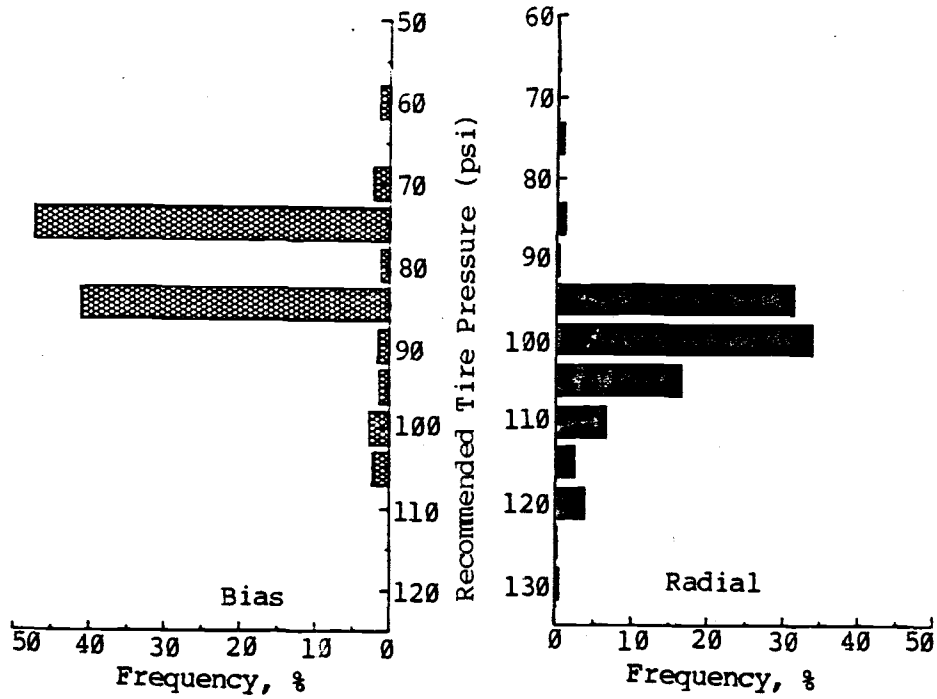


(a) Single Tire, Steering Axle

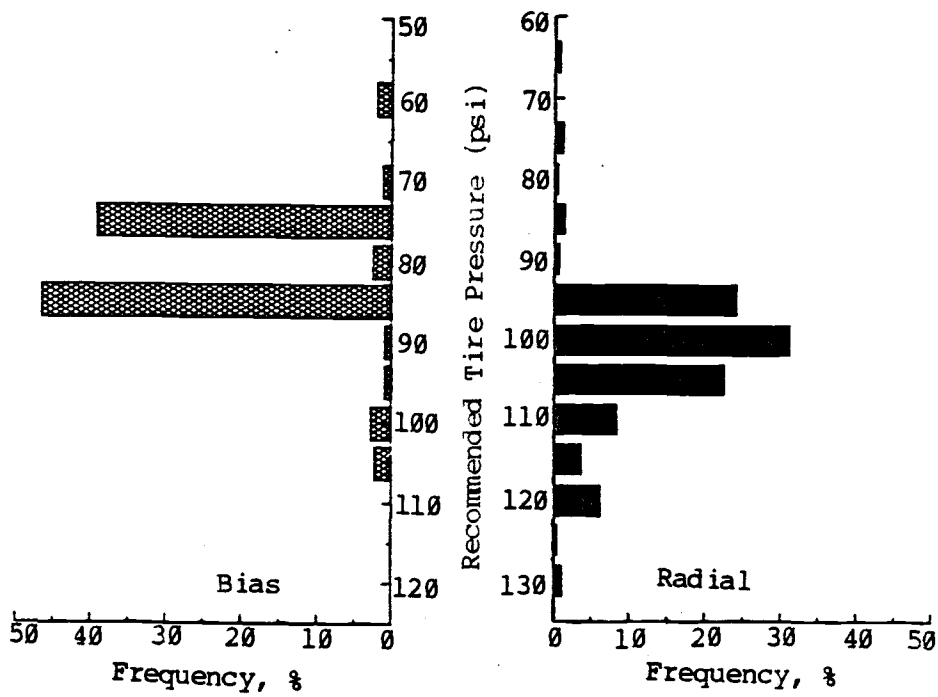


(b) Single Tire, Non-Steering Axle

Figure 4.3 Distribution of the Recommended Tire Pressure.



(c) Dual Tires, Non-Steering Axle



(d) Total Tires

Figure 4.3 Distribution of the Recommended Tire Pressure (Continued).

4.3.5 Tread Depth

Figure 4.5 and Table 4.3 present the results of tread depth survey. The average tread depth for radial tires used for steering axles is 13/32 in. This is the highest tread depth among the groups. The average tread depth for bias dual tires used for non-steering axles is 9/32 in. This is the lowest measured tread depth.

4.3.6 Tire Size

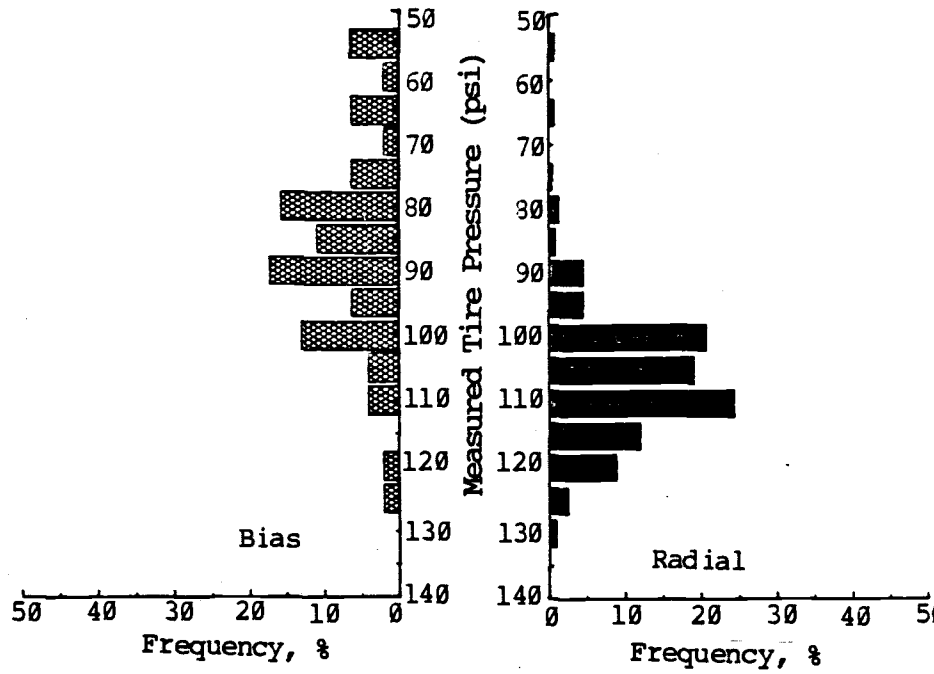
Table 4.4 presents the distribution of tire size for both radial and bias tires. The major tire size for radials is 11/80 R 24.5. However, for the single tire for non-steering axles, the major tire size is 12 R 22.5, which is slightly wider than 11/80 R 24.5. The major tire sizes for bias are 11-24.5 and 10-20.00 as presented in Table 4.4 (b). It should be noted that 13.2% of single radial tires used for non-steering axles are 15 R 22, i.e., 15 in. wide tires, which are wider than the major tire sizes.

Figure 4.6 shows the description of tire dimensional information used in truck tire sizing nomenclature. For example, 11/80 R 24.5 means that the size of this tire is 11 in. wide, aspect ratio of 0.8 (section height/section width, see Figure 4.7), radial, and rim diameter of 24.5 in. Bias ply designated with a hyphen in place of R.

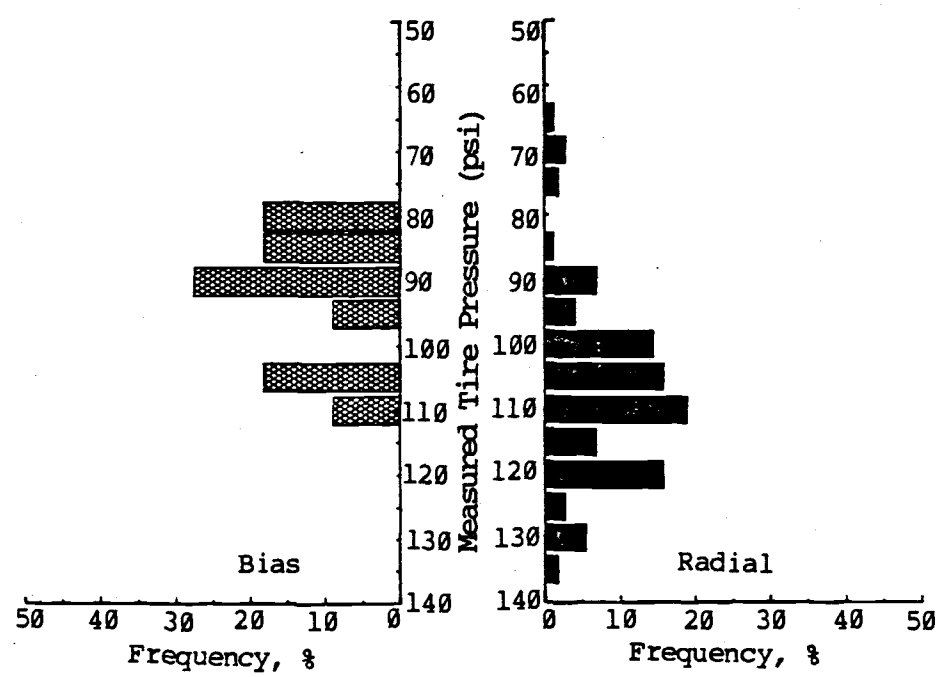
More detailed data on tire size are presented in Refernce 3.

4.3.7 Manufacturer

Table 4.5 presents the distribution of the top eight manufacturers surveyed for both radial and bias tires. It should be noted that one company, which supplies 28% of the radial tires, did not produce any

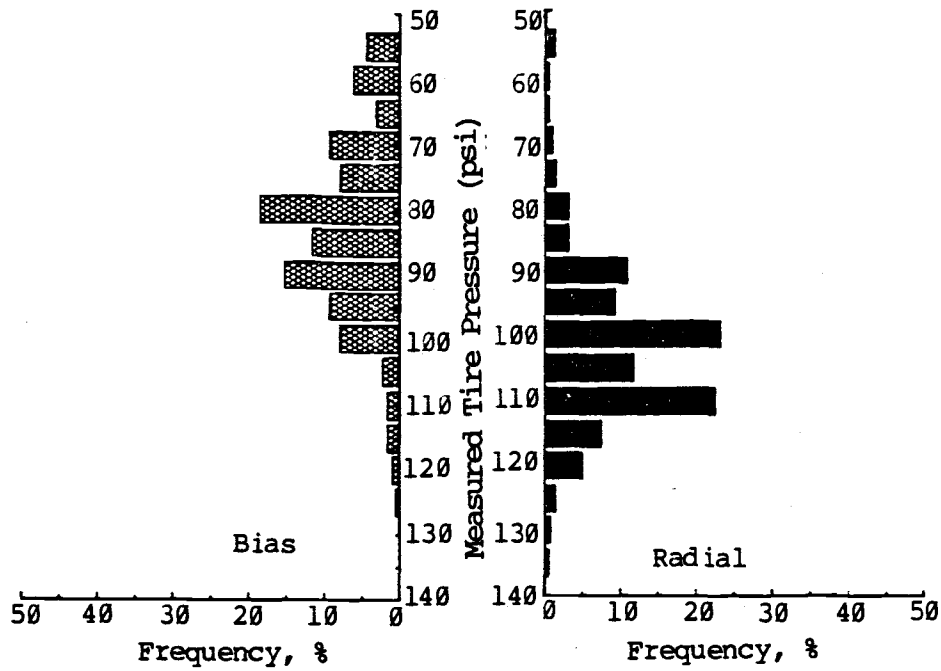


(a) Single Tire, Steering Axle

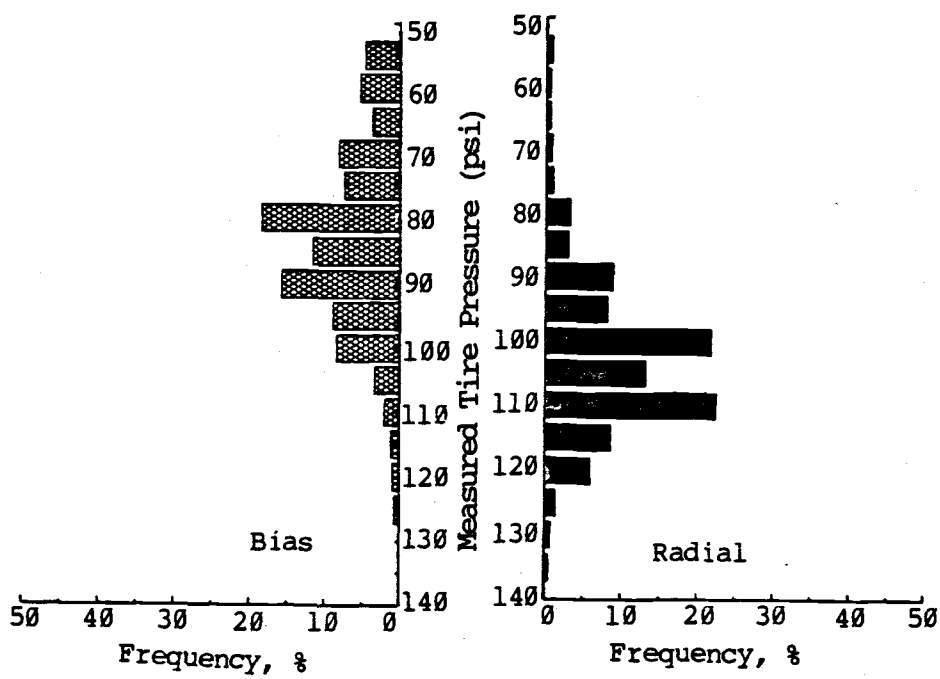


(b) Single Tire, Non-Steering Axle

Figure 4.4 Distribution of the Measured Tire Pressure.

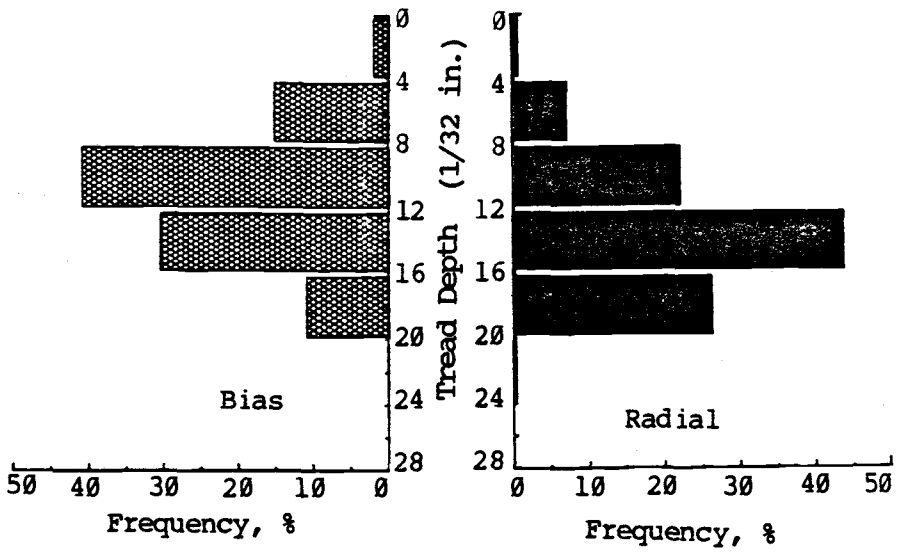


(c) Dual Tires, Non-Steering Axle

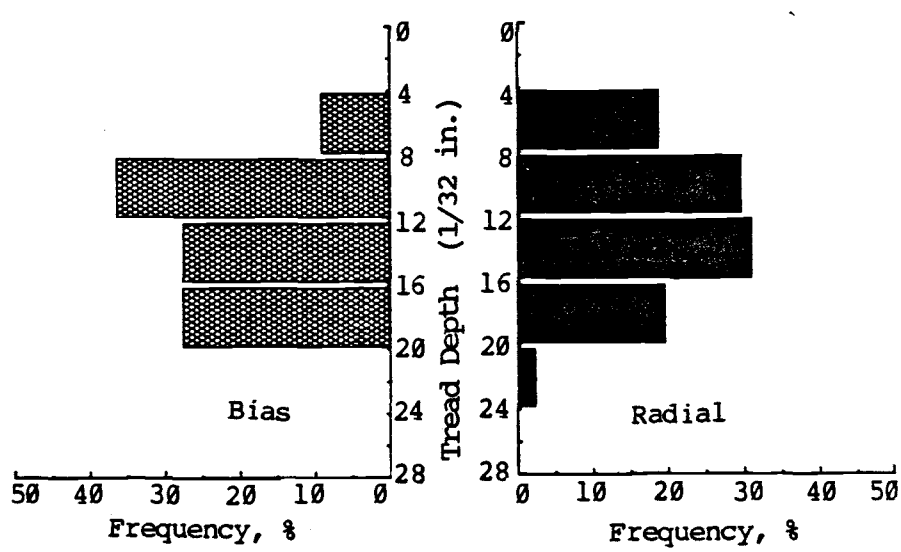


(d) Total Tires

Figure 4.4 Distribution of the Measured Tire Pressure (Continued).

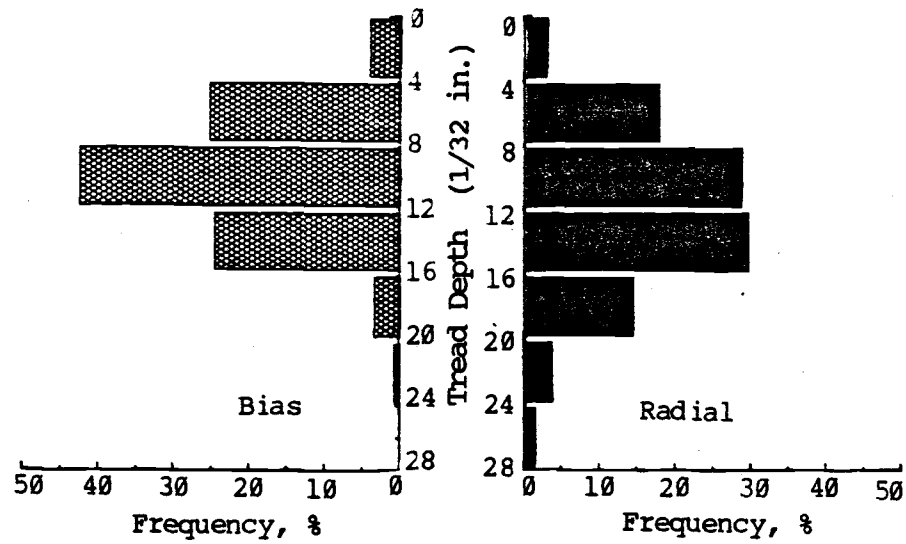


(a) Single Tire, Steering Axle

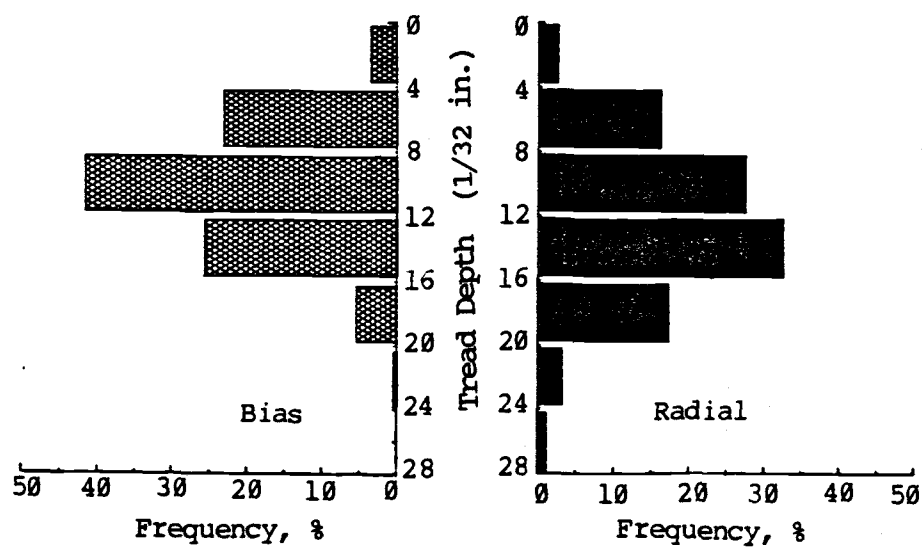


(b) Single Tire, Non-Steering Axle

Figure 4.5 Distribution of Tread Depth.



(c) Dual Tires, Non-Steering Axle



(d) Total Tires

Figure 4.5 Distribution of Tread Depth (Continued).

Table 4.4 Tire Size Distribution (%).

(a) Radial Tire

Tire Size	I*	II	III
11/80 R 24.5	46.5	15.4	49.1
11 R 22.5	22.2	19.8	21.1
285/75 R 24.5	9.6	1.1	7.1
275/80 R 24.5	6.1	3.3	3.9
275/80 R 22.5	3.9	-	4.1
12 R 22.5	2.0	33.0	2.2
10.00 R 22	2.0	-	3.9
15 R 22.5	-	13.2	-
Others	7.7	14.2	8.6
Sample Number	490	91	1737

(b) Bias Tire

Tire Size	I*	II	III
11-24.5	30.8	-	30.8
10.00-20	15.4	36.4	29.8
10.00-22	11.5	18.1	21.2
11-22.5	17.3	-	9.9
9.00-20	3.8	45.5	2.6
Others	21.2	0.0	5.8
Sample Number	52	11	302

I* ; Single Tire, Steering Axle, II; Single Tire, Non-Steering Axle, III; Dual Tires, Non-Steering Axle.

NUMERIC (TWO PART):

7.00	R	15	LT
Approx. Cross Section Width in Inches	Radial Construction *	Rim Diameter In Inches	Light Truck application

ALPHANUMERIC:

H	R	78	—	15	LT
Tire Size/ Load	Radial Const.	Series (Aspect Ratio)		Rim Diameter In Inches	Light Truck application

METRIC:

LT	235	/85	R	16	E
Light Truck	Tire Section Width (MM)	Aspect Ratio	Radial Const. *	Rim Diameter	Load Range

THREE PART (FLOTATION SIZING):

26	x	8.50	R	14	LT
Overall Diam. In Inches		Approx. Cross Section Width In Inches	Radial Const. *	Rim Diam. In Inches	Light Truck application

*Bias ply designated with a hyphen in place of "R".

Figure 4.6 Tire Sizing Designation (After Ref. 7).

bias tires.

More detailed data on tire manufacturer are presented in Reference 3.

4.4 DISCUSSION

As expected, the majority of tires were of radial construction. As presented in Table 4.5, one company which supplies 28% of the radial tires did not produce any bias tires. There may be several reasons that bias tires have replaced with radial tires, as outlined below.

From the 1970s, the trucking industry increased their use of radial truck tires as tire service demands on medium and heavy trucks increased. Testing done on bias and radial tires with similar tread designs from the same manufacturer confirmed that the radial tire generally offered improvements over the bias, as presented in Table 4.6 (8).

As mentioned earlier, the federal goverment permitted 80,000 pounds gross vehicle weight and 34,000 pounds tandem-axle weights on interstate highways in 1982. This allowed a potential 12,000 pound load on the steering axle. Most states invoke a restriction on the load per inch width of tire of 600 pounds, i.e., two 10 in. wide tires could legally support a 12,000 pound axle load. According to Cooper (8), two bias tires in the commonly used sizes and standard 12-ply rating do not have 12,000 pounds capacity, but two standard 14-ply rating radial tires which allow higher inflation pressure necessary for a higher capacity rating do carry over 12,000 pounds. The improved loading capacity and the advantages presented in Table 4.6 are some of the reasons which have led to radial truck tire usage increasing.

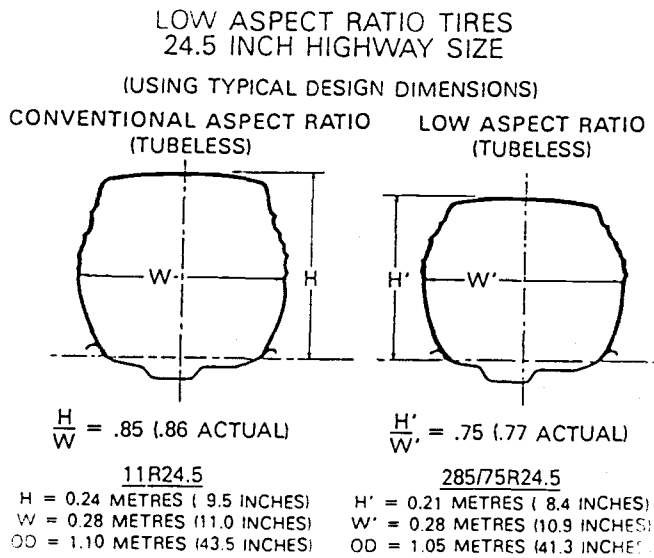


Figure 4.7 Conventional vs. Low Aspect ratio Comparison (After Ref.8).

Table 4.5 Distribution by Tire Manufacturer (%).

(a) Radial Tire

Manufacturer	I*	II	III
1. Michelin	25.0	36.3	28.4
2. Goodyear	22.0	11.0	22.7
3. Bridgestone	15.5	24.2	15.0
4. Toyo	9.7	15.4	9.6
5. Kelly	3.6	2.2	4.0
6. Yokohama	3.8	1.1	3.0
7. Firestone	2.2	1.1	2.8
8. OHTSU	3.6	1.1	1.7
Others	14.6	7.6	12.8
Sample Number	496	91	1755

(b) Bias Tire

Manufacturer	I*	II	III
1. Goodyear	10.9	30.0	23.2
2. Firestone	6.5	-	9.5
3. Goodrich	10.9	20.0	6.7
4. Bridgestone	-	-	7.7
5. General	-	10.0	7.0
6. Multimile	8.7	-	3.5
7. Dunlop	4.3	0.0	3.9
8. OHTSU	8.7	20.0	2.1
Others	50.0	20.0	36.4
Sample Number	46	10	284

I* ; Single Tire, Steering Axle, II ; Single Tire, Non-Steering Axle
III ; Dual Tires, Non-Steering Axle.

Table 4.6 Bias versus Radial Tire Performance Testing (After Ref. 8).

Property	Type Test	Bias Tire	Radial Tire
Wear Rate	Proving Grounds	Par	Better
Wear Regularity	Proving Grounds	Par	More Sensitive
Running Temperature	Laboratory	Par	Better (Lower)
Fuel Economy	Proving Grounds	Par	Better (6% Savings)
Tire Noise	SAE J57A	Par	Better (3 dBA Less)
Puncture Resistance	Commercial Fleet	Par	Better (40% Fewer)

tires in the commonly used sizes and standard 12-ply rating do not have 12,000 pounds capacity, but two standard 14-ply rating radial tires which allow higher inflation pressure necessary for a higher capacity rating do carry over 12,000 pounds. The improved loading capacity and the advantages presented in Table 4.6 are some of the reasons that have led to radial truck tire usage increasing.

Wong (9) indicated that for a radial ply tire on a hard surface, there is a relatively uniform ground pressure over the whole contact area. In contrast, the ground pressure for a bias ply tire varies greatly from point to point as tread elements passing through the contact area undergo complex localized wiping motion. However, the effect of different tire construction on asphalt pavements is still not known well.

As shown in Table 4.3, the average of the recommended tire pressure of single tires (for both steering axles and non-steering axles) is higher than that of dual tires. The same trend appears in the 1st measured tire pressure distribution, as presented in the same Table. Therefore, the data show that truckers tend to use higher tire pressure (i.e., higher rated tires) for a single tire for both steering axle and non-steering axles than for dual tires for non-steering axles.

As indicated in Table 4.3, the 1st measured tire pressure is slightly higher than the 2nd measured one. For radial tires, the difference between the 1st measurement and the 2nd measurement is smaller than that for bias tires except the case of single tire for non-steering axles.

For radial tires, Table 4.3 and Figures 4.4 and 4.5 show that truckers tend to use the manufacturer's maximum recommended tire

pressure. This is due to operation safety and efficiency. For bias single tires with non-steering axle, the average of the measured pressure is higher by about 10 psi than that of the recommended maximum pressure, but the sample size of 11 tires is very small.

As shown in Table 4.7, the difference between recommended pressure and measured pressure for radial tires is almost zero. However, for bias tires, the inflated pressure is greater than the recommended pressure. As presented in Table 4.3, the radial tire pressure is higher by 20 psi than bias tire pressure. The study performed by Middleton et al.(5) indicated that radial tires on the average showed 12 to 21 psi higher pressure than did bias tires.

If government agencies wish to control tire pressures, it would be expedient to control the manufacturer's maximum recommended pressure rather than the inflation pressure used by truckers. This would ensure reasonable control, since the data collected in this study show that measured and recommended tire pressures are nearly equal.

In general, higher inflation pressure tires have deeper tread depth as presented in Table 4.3. This implies that operators may use higher pressures with newer tires.

Recently, the trucking and tire industries have started to promote super single radials and new low profile (or low aspect ratio) tubeless tires. The concept of replacing dual tires with a wide single is not new but has gained popularity recently in the long haul market. As mentioned in the earlier section, 13.2% of single tires used for non-steering axles are 15 R 22 (Table 4.4 (a)). According to the restriction of 600 pounds per inch width of tire, two tires of 15 in. wide can support 18,000 pounds, that is, the equivalent standard single

Table 4.7 Mean Value of the Tire Pressure Difference between Recommended Pressure and 1st Measured Pressure*.

	I**		II		III	
	R#	B	R	B	R	B
Mean (%)	0.3	2.5	-0.2	10.0	1.3	2.2
Standard Deviation (%)	10.7	14.6	8.0	9.6	12.9	19.9
Sample Number	495	44	89	11	1734	285

* ;(Measured Pressure-Recommended Pressure)/(Recommended Pressure)*100.

I** ; Single Tire, Steering Axle, II ; Single Tire, Non-Steering Axle, III ; Dual Tires, Non-Steering Axle.

R# ; Radial Tire, B ; Bias Tire.

axle load used in pavement design by many states.

New super single radial tires are claimed to have 10% or better tread mileage and 8 to 10% better fuel economy than conventional dual radials (10). Also, the lighter weight of the wide-base single tire assembly permits higher payloads. The reduced tire aspect ratio (section height/section width, see Figure 4.7) decreases tire deflection, thereby improving vehicle handling and stability while increasing tread life and fuel economy. However, the effect of the super single tire on the performance of asphalt pavement needs more study.

The pressure data collected indicate that the mean pressure of the whole sample is similar to the Texas study results (5) and is considerably higher than that traditionally used in pavement design (i.e., 80 psi). Since the study described herein and other studies have confirmed the wide varieties of tires and pressures used, it is necessary to refine paving mix design, and pavement structure design methods, to account for the levels of tire pressure prevailing. Also, the remaining life of existing asphalt pavements or maintenance schedules on the section having high truck traffic might be adjusted due to increased truck tire pressure resulting in severe damage. For the consideration of registration fees and fine schedule, the factor of truck tire pressure as well as axle loads could be included.

However, caution is advised when considering the effects of tire pressures on asphalt pavements for say 80 psi (a typical tire pressure in the 1960's) with 100 psi (a typical tire pressure in the 1980's) because the former were almost exclusively bias tires and the latter are predominantly radial tires.

4.5 CONCLUSIONS AND RECOMMENDATIONS

The existing operating characteristics of Oregon's trucks, including levels of tire pressures, were surveyed and analyzed. The major findings and conclusions of this study are:

1. As expected the use of radial tires is dominant. Eighty-seven percent of the tires surveyed were of radial construction. The bias tires used may be replaced with radial tires in future.
2. The average measured pressures of radial and bias tires are 102 and 82 psi, respectively. Therefore, adequate consideration of current levels of tire pressure should be reflected in paving mix design, pavement structure design methods including overlay design, and maintenance schedules.
3. Since the difference between the recommended maximum tire pressure by manufacturer and the measured tire pressure is very small, it can be said that truckers tend to use the recommended maximum pressure (cold) due to operating safety and efficiency.
4. The sizes of most radial and bias tires are 11/80 R 24.5 and 11-24.5, respectively.
5. The average tread depth of radial tires is slightly greater than that of bias tires.

In order to control the effect of increased tire pressure on asphalt concrete pavement, the following recommendations are made:

1. If government agencies wish to control tire pressures, it would be expedient to control the manufacturer's maximum recommended tire pressure (cold) rather than the infalation pressures used by truckers since the data collected in this study show that

measured and recommended tire pressures are nearly equal.

2. For overload permits, fees and fine schedules, the levels of tire pressure might be included in assigning appropriate cost responsibility after the investigation into the effect of higher tire pressures on the asphalt pavements is performed.

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Special appreciation is given to Ken Evert and his staff who collected tire pressure data (total of about 2700 tires) day and night during the summer of 1986.

DISCLAIMER

5.0 STUDY ON MIX DESIGN CRITERIA FOR CONTROLLING THE EFFECT OF INCREASED TIRE PRESSURE ON ASPHALT PAVEMENT

Ok-Kee Kim
Graduate Research Assistant
Department of Civil Engineering
Oregon State University

Chris A. Bell
Associate Professor
Department of Civil Engineering
Oregon State University

James E. Wilson
Retired Engineer of Materials
Oregon Department of Transportation

and
Glenn Boyle
Group Leader - Asphalt Mix Design
Oregon Department of Transportation

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Transportation Research Board Annual Meeting, January 1988

August 1987

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ABSTRACT

As axle loads have increased, higher tire pressures have become more popular in the truck market and radial tires are predominantly used. However, existing mix design procedures may not produce mixtures capable of dealing with higher tire pressures. Similarly, they may not identify potentially highly deformable mixtures.

In order to evaluate the mix design process used by Oregon State Highway Division, aggregate from four different sources was used. One percent lime slurry was added into two aggregates. Six different aggregate gradations, including the Fuller maximum density gradation, were tested. In addition to the routine asphalt mix tests, a simple creep test was run for 3 hours at 40°C and a compression stress of 0.1 MPa (14.5 psi) was applied.

According to the results of creep tests, it is not always true that a mix with high Hveem stability value resists the deformation better than one with low stability. This shows that the current mix design criteria are probably inadequate to produce mixtures capable of dealing with the high tire pressures, and to identify potentially highly deformable mixtures.

In general, the creep stiffness decreases with the increasing percentage passing the #200 sieve size. The effect of percentage passing 1/4-in. or #10 sieve sizes on the creep stiffness is not clear. The results show that adding 1% lime slurry improves the resistance to deformation of asphalt mixes.

5.1 INTRODUCTION

5.1.1 Problem Statement

The economics of truck transportation has tended to cause the average gross weight of trucks to increase such that the majority of trucks are operating close to the legal gross loads or axle loads (1). In 1982, the federal government permitted 80,000 pounds gross vehicle weight, 20,000 pounds single axle weights and 34,000 pounds tandem axle weights on Interstate highways. Tandem axle weights of 34,000 pounds allowed a potential 12,000 pound load on the steering axle. Many states, including Oregon (2), also issue permits for trucks to operate above normal legal load limits.

As axle loads have increased, the use of higher tire pressures has become more popular in the truck market. A recent survey in Texas (3) indicated that trucks typically operate with tire pressures of about 100 psi in that state. Another study in Oregon (4) showed that about 40% of radial tires are inflated more than 110 psi, while the average inflation pressure is 102 psi and 82 psi for radial tires and bias tires, respectively.

Higher tire pressures decrease the contact area between the tire and the pavement, resulting in reduced tire friction or skid resistance and increased potential for pavement damage under the high stress. The higher tire pressures contribute to greater deformation in flexible pavements, manifested as high severity wheel track rutting.

In Oregon, there have been several occurrences of high wheel track rutting which have been associated with high tire pressures prevailing in recent years. The rutting is a function of deformation in all layers of a flexible pavement structure, but with high tire pressures, the

deformation in the asphalt concrete mixture is a major contributor. Existing mix design procedures may not produce mixtures capable of dealing with the high tire pressures. Similarly, they may not identify potentially highly deformable mixtures.

5.1.2 Objectives

A study of procedures for controlling the effect of increased tire pressure on asphalt concrete pavement damage (4) was performed by Oregon Department of Transportation (ODOT) and Oregon State University (OSU).

This paper presents a part of this study, that is, the results of mix design evaluation and the results of creep testing to predict rut depth in asphalt pavement.

The objectives of this paper are:

1) To present and analyze existing asphalt concrete mix design methods in limiting excessive deformation caused by higher loads and tire pressures
2) To present and analyze the results of creep testing to predict deformation in asphalt surface layers.

5.2 BACKGROUND

5.2.1 Mix Design

The Marshall and Hveem methods of mix design have been widely used with satisfactory results. For each of these methods, criteria have been developed by correlating the results of laboratory tests on the compacted paving mixes with the performance of the paving mixes under service conditions.

However, the limitations of such empirically based methods of

pavement mix design have become increasingly apparent in recent years as traffic loads, tire pressures and numbers of trucks have increased. Increasing demands on asphalt pavements from higher traffic volumes and higher truck tire pressures cause highway engineers to examine the foundations of asphalt mix design to see how best to cope with these challenges.

Existing mix design procedures may not produce mixtures capable of dealing with higher tire pressures. Similarly, they may not identify potentially highly deformable mixtures. Such a situation was identified by Finn et al. (5) in designing mixtures for heavy duty airfield pavements, where very high tire pressures occur. They utilized a simple creep test, similar to that developed by Shell researchers (6), to complement Marshall and Hveem mix design procedures and to quantify deformation characteristics of the mix.

Hicks and Bell (7) recently completed a study for Oregon State Highway Division (OSHD) to evaluate their current mix design process, which is based on the Hveem procedure. They indicated that gradation of aggregate can be one of the main contributors in producing tender mixes. Many researchers (8) indicate that the potential of producing tender mixes and pavement deformation increases if gradation values are greater than the following:

Sieve Size	% Passing
#4	55
#10	37
#40	16
#200	3-7

Further, they indicate that gradation curves that cross back and forth over the maximum density curve, especially in the region of the No. 30 to No. 80 sieve, tend to produce tender mixes.

5.2.2 Creep Test

In the major effort towards developing rational procedures for the design of asphalt mixes, an attempt has been made to develop a suitable test method to judge the stability properties of asphalt mixes. Van de Loo (9) defined the stability properties of an asphalt mix as the resistance of a mix to rutting in an actual pavement, i.e., under varying conditions of climate, traffic volume, and traffic load.

Many researchers have used the creep test (static or repeated mode) as a relatively simple test to predict rutting (or permanent deformation) of an asphalt pavement. In 1973, theoretical deformation models of asphalt mixes were formulated by J.F. Hills (10). It was assumed that any deformations in the mix are the result of sliding displacements between adjacent mineral particles, separated by a thin film of asphalt. He interpreted the results in terms of a mix stiffness (S_{mix}) as a function of bitumen stiffness (S_{bit}). Hills stated that, in addition to the effect of the volume concentrations of the mineral aggregate, the gradation, shape, and surface texture of the aggregate play a role and the state of compaction exerts a strong influence on the behavior.

Reference 11 provides the recommendation for the performance of unconfined, static creep test which was standardized during Colloquium 1977 held in Zürich. The recommended sample size is the same as the normal Marshall specimens (i.e., 4-in. diameter and 2.5-in. high) and

should have achieved a steady temperature of 40°C before the test commences. The constant load of 0.1 MPa (14.5 psi) should be applied without any impact and have a duration of one hour. A loading time of one hour is arbitrary.

The deformation of an asphalt specimen is measured as a function of loading time at a fixed test temperature. The general equation of the creep curves is:

$$\log(\epsilon) = c + n\log(t) \qquad [1]$$

where ϵ = creep strain at time t and

c, n = constants.

The constants n and c are related to test conditions such as uniaxial stress and temperature, as well as asphalt cement content and the factors indicated by Hills above. The constant n represents the inclination of the straight line. Relatively small n indicates a less viscous behavior, and, relatively large n a predominant viscous behavior (11). It has been found that the level of the instantaneous response increases with the amount of filler and bitumen (12). Furthermore, the time dependence of the vertical displacement has been associated with the viscosity of the mortar, which is related to the filler-binder ratio.

5.3 EXPERIMENTS DESIGN - TESTS ON ASPHALT MIXTURES

5.3.1 Variables Considered

Aggregate from four different sources was used for the laboratory mixture study. The sources of aggregate were:

1) Morse Brothers Pit (gravel),

2) Cobb Rock (quarry),

3) Hilroy Source (gravel), and

4) Blue Mountain Asphalt Pit (gravel).

For the mix with the aggregate from Cobb Rock and Blue Mountain Asphalt Pit, 1% lime slurry was added.

The variables considered in the laboratory mixture preparation for the creep test were:

1) Asphalt cement content:

   A) 4, 5, 6%,

   B) 4.5, 5.5, 6.5%, and

   C) 5, 6, 7%.

2) Aggregate gradation: A through F (Table 5.1)

   A) 65% passing 1/4-in., 32% passing #10, 5% passing #200,

   B) 60% passing 1/4-in., 29% passing #10, 5% passing #200,

   C) Fuller curve - 60% passing 1/4-in., 36% passing #10, 8% passing #200,

   D) Same as B except 35% passing #10,

   E) 60% passing 1/4-in., 33% passing #10, 5% passing #200, and

   F) Same as E except 8% passing #200.

Table 5.2 presents the aggregate gradations considered for each aggregate source. The properties of asphalt cements used are presented in Table 5.3.

5.3.2 Specimen Preparation and Test Program

Following the standard ODOT procedure (13) using a kneading compactor, 4-in. (100 mm) diameter by 2.5-in. (63 mm) high specimens were fabricated from four different aggregate sources.

A flow chart of the test program followed in this study is given in

Table 5.1 Aggregate Gradations (A through F).

Aggregate Gradation	Morse Brothers Pit			Cobb Rock			
	A	B	C	A	B	C	D
1-in.	—	—	—	—	—	—	—
3/4-in.	100	100	100	100	100	100	100
1/2-in.	98	97	82	99	99	86	82
3/8-in.	86	83	72	82	78	73	72
1/4-in.	65	60	60	66	60	60	60
No. 10	32	30	37	32	29	37	37
No. 40	13	11	18	13	11	19	19
No. 200	4.7	4.3	6.8	6.7	6	9	6.9

Aggregate Gradation	Hilroy Source						Blue Mountain Asphalt Pit				
	A	B	C	D	E	F	A	B	C	D	E
1-in.	100	100	100	100	100	100	—	—	—	—	—
3/4-in.	99	98	99	99	98	98	100	100	100	100	100
1/2-in.	86	85	82	85	85	85	87	87	86	87	87
3/8-in.	76	72	72	72	72	72	77	74	73	73	73
1/4-in.	65	60	60	60	60	60	65	60	60	60	60
No. 10	33	31	37	37	34	34	32	29	36	36	34
No. 40	14	13	19	19	14	14	14	13	16	16	15
No. 200	4.5	4.2	5.9	4.3	5	6.9	5	4.5	7	5	5.2

Table 5.2 Aggregate Gradations Considered for Each Aggregate Source.

Aggregate Source	Aggregate Gradation					
	A	B	C	D	E	F
1. Morse Brothers Pit	X	X	X			
2. Cobb Rock (with 1% lime slurry)	X	X	X	X		
3. Hilroy Source	X	X	X	X	X	X
4. Blue Mountain Asphalt Pit (with 1% lime slurry)	X	X	X	X	X	

Figure 5.1. The Oregon Department of Transportation (ODOT) testing program included the conventional mix tests such as the Hveem stability test (AASHTO T-246), Rice maximum specific gravity test (AASHTO T-209), bulk specific gravity test (AASHTO T-166), and repeated load diametral test for resilient modulus (as compacted and after moisture conditioning). Oregon State University (OSU) performed the creep test with 54 laboratory-fabricated specimens as described in the following section.

5.3.3 Test Methods

After preparing laboratory mixes, repeated load diametral tests and creep tests were performed. The procedures are outlined below.

5.3.3.1 Resilient Modulus: The resilient modulus test was performed using the repeated load diametral test apparatus. The maximum load applied and the horizontal elastic tensile deformation were recorded to determine the resilient modulus using the following equation:

$$M_R = P(0.2692 + 0.9974\nu)/(\Delta H \times t) \qquad [2]$$

where M_R = resilient modulus, psi;

ΔH = horizontal elastic tensile deformation, inches;

P = dynamic load, lbs;

t = specimen thickness, inches; and

ν = Poisson's ratio.

Poisson's ratio was assumed constant and equal to 0.35, which simplified Eq. [2] to:

$$M_R = 0.6183P/(\Delta H \times t) \qquad [3]$$

Table 5.3 Physical Properties of Asphalt Cement.

	I*	II	III	IV
Grade	AR 4000	AR 4000	AR 4000	AR 4000
Original				
- Penetration, 77 F	68	68	68	61
- Absolute Viscosity, 140 F, Poises	1339	1349	1349	2111
- Kinematic Viscosity, 275 F, C.S.	261	248	248	352
- Flash Point, Open Cup, F	600	605	605	580
After Rolling Thin Film Oven Test				
- Penetration	41	40	40	32
- Absolute Viscosity, 140 F, Poises	3033	3139	3139	5860
- Kinematic Viscosity, 275 F, C.S.	367	365	365	562
- Loss on Heating, %	0.45	0.52	0.52	0.65

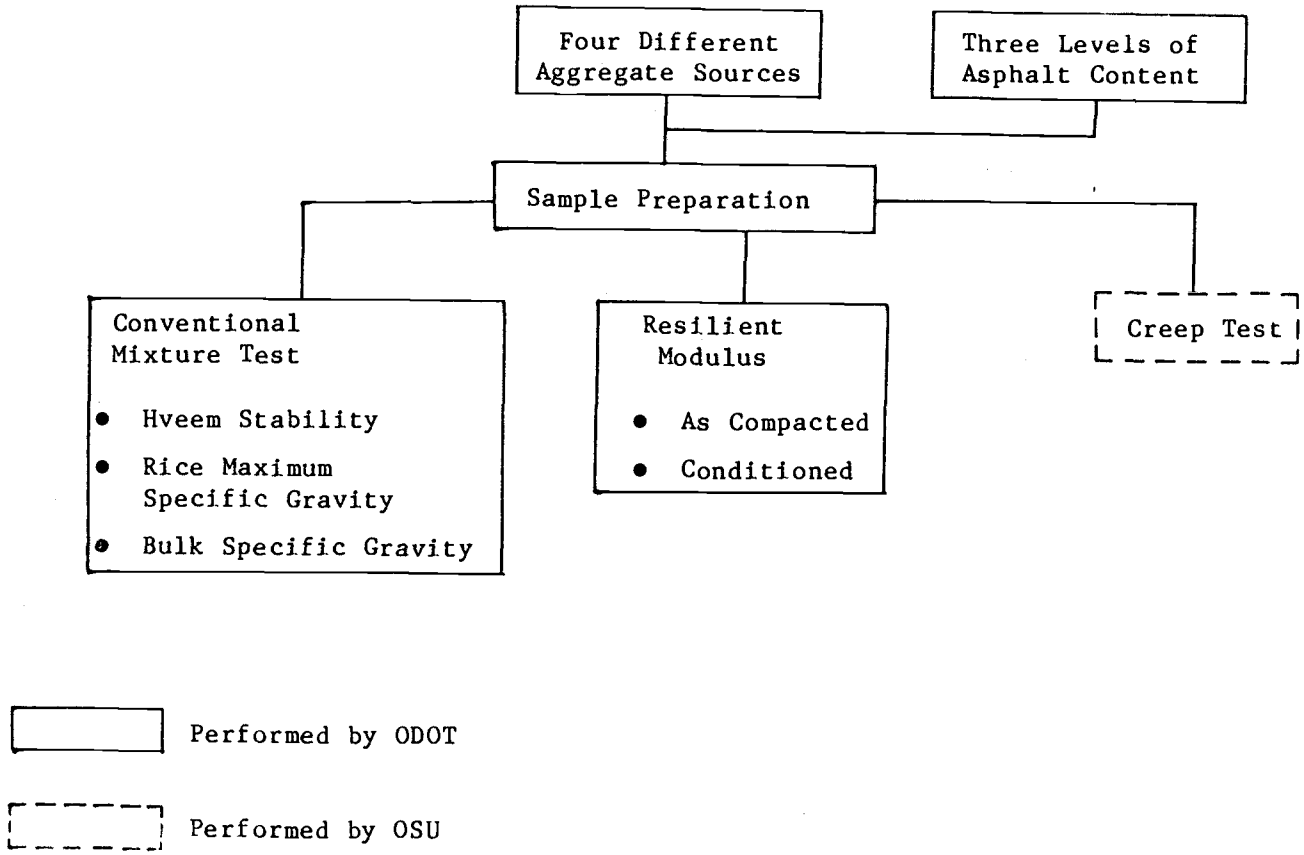


Figure 5.1 Flow Chart for Test Program.

During the test, the dynamic load duration was fixed at 0.1 sec and the load frequency at 60 cycles per minute. A static load of 10 pounds (4.5 kg) was applied to hold the specimen in place. The test was carried out at 77°F (25°C).

Each specimen was tested both before and after conditioning for each aggregate source. The specimen conditioning procedures was based on the moisture damage test defined by Lottman (14).

5.3.3.2 Creep Test: OSU was responsible for developing a simple creep test and running the test. For the creep test, a loading device for soil consolidation and data acquisition/control unit with a personal computer were used. The creep test was run for 3 hours at 40° C and a compression stress of 0.1 MPa (14.5 psi) was applied. The creep test procedure is as follows:

1) Put a loading device for the soil consolidation in an environmental cabinet and connect to the repeated load test control cabinet. Put the specimens and a dummy specimen with a thermistor into the environmental cabinet. Set the regulator at 0.1 MPa and control the air pressure by repeated load test control cabinet.
2) Warm the inside of the environmental cabinet to 40°C and check the temperature of the dummy specimen using the data acquisition system and thermistor.
3) After the temperature of the dummy specimen core reaches 40°C, put a specimen on a load plate. Put an LVDT on the bottom plate and attach a thermistor to the specimen. Check the level of the bottom plate before running the test.

4) Wait for 5 to 10 minutes after closing the environmental cabinet door to keep the temperature at 40°C.

5) Apply a pressure of 10 kPa as a preload for 2 minutes.

6) Apply a pressure of 0.1 MPa and run the computer program.

Reference 4 describes the apparatus and the procedure for sample preparation in detail. Also in Reference 4 are the computer programs to monitor the temperature and measure the deformation of a specimen.

5.4 RESULTS

5.4.1 Mix Design

The summary of the mix design for each of the aggregate sources with different aggregate gradations is presented in Table 5.4. Table 5.4 includes the resilient modulus (both as-compacted and after conditioning) and the minimum asphalt content for the retained modulus ratio of 0.7. The retained modulus ratio is defined by Eq. [4]:

$$\text{Retained Modulus Ratio} = \frac{M_R \text{ after conditioning}}{M_R \text{ before conditioning}} \qquad [4]$$

5.4.2 Creep Test

Table 5.5 presents the creep test results, including the intercept (I) and slope (S) after regression analysis and creep stiffness at 60 minutes. The coefficients of determination (R^2) also are presented. The regression analysis was performed in the range from 1 min. to 90 min. Figure 5.2 presents a typical relationship between creep strain and time.

The intercept and the slope of each sample are obtained by the following equations:

$$\log(\text{strain},\%) = \log(I) + S*\log(\text{time},\text{sec}) \qquad [5]$$

Table 5.4 Summary of Mix Design Data.

(a) Morse Brothers Pit, Gravel, Chevron AR-4000

Sample ID*	Max. Sp.Gr.	Bulk Sp.Gr.	Air Voids (%)	A/C Contents (%)	VMA (%)	Stability[1]	M_r (ksi) As-Comp.[2]	M_r (ksi) Cond.[3]	M_r Ratio[4]	Min. A/C to .7 MRRT[5] (%)	Optimum A/C (%)
A32	2.484	2.26	9.0	5.0		33	258	146	0.56		
A33	2.455	2.30	6.3	6.0		35	227	197	0.87	5.5	6.6
A34	2.408	2.32	3.6	7.0		31	224	189	0.84		
B29	2.463	2.28	7.4	5.0		35	186	102	0.55		
B30	2.446	2.30	6.0	6.0		32	187	139	0.75	5.8	6.6
B31	2.423	2.33	3.8	7.0		33	194	133	0.69		
C26	2.489	2.34	6.0	4.5		36	492	161	0.33		
C27	2.466	2.37	3.9	5.5		37	447	349	0.78	5.3	5.1
C28	2.440	2.40	1.6	6.5		19	303	237	0.78		

*A, B, C = aggregate gradation type.

[1]Stability = stability at first compaction

[2]M_r As-Comp. = resilient modulus at 25°C, as compacted

[3]M_r Cond. = resilient modulus at 25°C, after conditioning

[4]M_r Ratio = $\frac{\text{resilient modulus after conditioning}}{\text{resilient modulus before conditioning}}$

[5]Min A/C to .7 MRRT = minimum asphalt content for the retained modulus ratio (M_r Ratio) of 0.7

Table 5.4 Summary of Mix Design Data (Continued).

(b) Cobb Rock, Quarry, 1% Lime, Chevron AR-4000

Sample ID*	Max. Sp.Gr.	Bulk Sp.Gr.	Air Voids (%)	A/C Contents (%)	VMA (%)	Stability[1]	M_r (ksi) As-Comp.[2]	M_r (ksi) Cond.[3]	M_r Ratio[4]	Min. A/C to .7 MRRT[5] (%)	Optimum A/C (%)
A11	2.514	2.25	10.5	4.5	15.1	41	361	172	0.48		
A12	2.476	2.29	7.5	5.5	14.5	37	320	346	1.08	4.9	6.3
A13	2.433	2.33	4.2	6.5	13.9	37	320	312	0.97		
B09	2.506	2.26	9.8	4.5	14.7	33	312	127	0.41		
B10	2.471	2.30	6.9	5.5	14.1	30	240	120	0.50	6.5	6.2
B11	2.433	2.34	4.2	6.5	13.5	37	266	187	0.70		
C09	2.512	2.33	7.2	4.5	12.0	39	465	301	0.65		
C10	2.471	2.37	4.1	5.5	11.5	31	392	501	1.28	4.6	5.3
C11	2.428	2.41	0.1	6.5	10.9	5	282	374	1.33		
D29	2.541	2.31	9.1	4.0	12.3	45	205	76	0.37		
D30	2.497	2.35	5.9	5.0	11.8	38	404	242	0.60	5.2	5.3
D31	2.459	2.39	2.8	6.0	11.2	33	232	302	1.30		

*A, B, C, D = aggregate gradation type

[1]Stability = stability at first compaction

[2]M_r As-Comp. = resilient modulus at 25°C, as compacted

[3]M_r Cond. = resilient modulus at 25°C, after conditioning

[4]M_r Ratio = $\frac{\text{resilient modulus after conditioning}}{\text{resilient modulus before conditioning}}$

[5]Min A/C to .7 MRRT = minimum asphalt content for the retained modulus ratio (M_r Ratio) of 0.7

Table 5.4 Summary of Mix Design Data (Continued).

(c) Hilroy Source, Gravel, Chevron AR-4000

Sample ID*	Max. Sp.Gr.	Bulk Sp.Gr.	Air Voids (%)	A/C Contents (%)	VMA (%)	Stability[1]	M_r (ksi) As-Comp.[2]	M_r (ksi) Cond.[3]	M_r Ratio[4]	Min. A/C to .7 MRRT[5] (%)	Optimum A/C (%)
A30	2.501	2.27	9.2	4.5	15.3	38	362	94	0.26		
A31	2.465	2.31	6.3	5.5	14.7	38	252	115	0.46	6.4	6.4
A32	2.429	2.34	3.7	6.5	14.5	36	239	180	0.75		
B21	2.493	2.27	8.9	4.5	15.3	36	364	93	0.26		
B22	2.459	2.29	6.9	5.5	15.5	35	280	150	0.54		6.2
B23	2.422	2.33	3.8	6.5	14.9	34	265	176	0.66		
C24	2.523	2.33	7.7	4.0	12.6	39	541	66	0.12		
C25	2.477	2.37	4.3	5.0	12.1	44	438	159	0.36	5.8	5.2
C26	2.437	2.41	1.1	6.0	11.5	35	384	302	0.79		
D27	2.474	2.33	5.8	5.0	13.5	40	391	142	0.36		
D28	2.431	2.37	2.5	6.0	13.0	41	403	260	0.65	6.3	5.6
D29	2.414	2.40	0.6	7.0	12.8	18	329	284	0.87		
E29	2.519	2.29	9.1	4.0	14.1	40	752	175	0.23		
E30	2.482	2.34	5.7	5.0	13.2	37	401	199	0.50	7.0	5.9
E31	2.443	2.35	3.8	6.0	13.7	40	396	239	0.60		
F09	2.519	2.30	8.7	4.0	13.8	37	420	89	0.21		
F10	2.482	2.38	4.1	5.0	11.7	39	429	293	0.68	5.3	5.3
F11	2.452	2.40	2.1	6.0	11.9	36	374	272	0.74		

*A, B, C, D, E, F = aggregate gradation type

[1]Stability = stability at first compaction

[2]M_r As-Comp. = resilient modulus at 25°C, as compacted

[3]M_r Cond. = resilient modulus at 25°C, after conditioning

[4]M_r Ratio = $\frac{\text{resilient modulus after conditioning}}{\text{resilient modulus before conditioning}}$

[5]Min A/C to .7 MRRT = minimum asphalt content for the retained modulus ratio (M_r Ratio) of 0.7

Table 5.4 Summary of Mix Design Data (Continued).

(d) Blue Mountain Asphalt Pit, Gravel, 1% Lime, Chevron AC-20

Sample ID*	Max. Sp.Gr.	Bulk Sp.Gr.	Air Voids (%)	A/C Contents (%)	VMA (%)	Stability[1]	M_r (ksi) As-Comp.[2]	M_r (ksi) Cond.[3]	M_r Ratio[4]	Min. A/C to .7 MRRT[5] (%)	Optimum A/C (%)
A38	2.583	2.33	9.8	4.5	17.9	29	437	214	0.49		
A39	2.545	2.37	6.9	5.5	17.4	30	404	291	0.72	5.4	5.6
A40	2.504	2.41	3.8	6.5	16.9	30	371	289	0.78		
B32	2.590	2.36	8.9	4.5	16.8	37	465	294	0.63		
B33	2.548	2.40	5.8	5.5	16.3	37	425	346	0.81	4.9	5.9
B34	2.510	2.44	2.8	6.5	15.8	38	374	346	0.92		
C29	2.607	2.37	9.1	4.0	16.0	39	679	339	0.5		
C30	2.565	2.41	6.0	5.0	15.5	38	630	353	0.56	5.4	5.3
C31	2.517	2.45	2.7	6.0	15.0	27	601	536	0.89		
D35	2.617	2.36	9.8	4.0	16.4	40	650	317	0.49		
D36	2.568	2.40	6.5	5.0	15.9	38	592	292	0.49	6.0	5.5
D37	2.530	2.44	3.6	6.0	15.4	33	523	372	0.71		
E37	2.607	2.32	11.0	4.0	17.8	37	836	496	0.59		
E36	2.574	2.39	7.1	5.0	16.2	35	728	737	1.01	4.3	5.7
E35	2.528	2.44	3.5	6.0	15.4	33	753	499	0.66		

*A, B, C, D, E, F = aggregate gradation type

[1]Stability = stability at first compaction

[2]M_r As-Comp. = resilient modulus at 25°C, as compacted

[3]M_r Cond. = resilient modulus at 25°C, after conditioning

[4]M_r Ratio = $\frac{\text{resilient modulus after conditioning}}{\text{resilient modulus before conditioning}}$

[5]Min A/C to .7 MRRT = minimum asphalt content for the retained modulus ratio (M_r Ratio) of 0.7

Table 5.5 Creep Test Results.

(a) Morse Brothers Pit, Gravel, Chevron AR-4000

Sample ID *	S_{mix}[1] (ksi)	I[2]	S[3]	R*R[4]
A32	3.47	0.098	0.177	0.961
A33	3.93	0.132	0.126	0.957
A34	3.14	0.116	0.169	0.996
B29	4.14	0.126	0.124	0.929
B30	6.37	0.084	0.122	0.930
B31	2.83	0.146	0.153	0.983
C26	3.57	0.142	0.129	0.951
C27	4.85	0.117	0.114	0.977
C28	5.24	0.069	0.170	0.973

* A, B, C = aggregate gradation type.

[1] S_{mix} = predicted creep stiffness at 60 min after regression.

[2] I = interception; strain, % at 1 sec.

[3] S = slope; strain, % = I*(time,sec)**S.

[4] R*R= coefficient of determination.

Table 5.5 Creep Test Results (Continued).

(b) Cobb Rock, Quarry, 1% Lime, Chevron AR-4000

Sample ID *	S_{mix}[1] (ksi)	I[2]	S[3]	R*R[4]
A11	4.76	0.135	0.099	0.940
A12	3.68	0.171	0.102	0.929
A13	5.40	0.105	0.115	0.997
B09	5.15	0.096	0.134	0.940
B10	3.33	0.206	0.091	0.931
B11	7.33	0.069	0.128	0.948
C09	3.95	0.075	0.194	0.998
C10	2.80	0.114	0.185	0.985
C11	1.47	0.307	0.143	0.962
D29	5.03	0.107	0.121	0.942
D30	3.81	0.093	0.172	0.964
D31	3.73	0.113	0.151	0.985

* A, B, C, D = aggregate gradation type.

[1]S_{mix} = predicted creep stiffness at 60 min after regression.

[2]I = interception; strain, % at 1 sec.

[3]S = slope; strain, % = I*(time,sec)**S.

[4]R*R= coefficient of determination.

Table 5.5 Creep Test Results (Continued).

(c) Hilroy Source, Gravel, Chevron AR-4000

Sample ID *	S_{mix}[1] (ksi)	I[2]	S[3]	R*R[4]
A30	5.06	0.127	0.099	0.898
A31	3.50	0.128	0.143	0.929
A32	2.05	0.073	0.227	0.983
B21	6.07	0.058	0.173	0.889
B22	4.85	0.064	0.188	0.944
B23	3.75	0.051	0.247	0.938
C24	4.05	0.101	0.155	0.960
C25	4.62	0.056	0.210	0.979
C26	3.59	0.091	0.182	0.984
D27	5.72	0.058	0.180	0.990
D28	8.06	0.046	0.167	0.945
D29	2.70	0.135	0.169	0.973
E29	5.90	0.027	0.271	0.977
E30	7.56	0.018	0.292	0.964
E31	7.77	0.018	0.283	0.976
F09	4.87	0.025	0.303	0.971
F10	4.70	0.020	0.336	0.980
F11	4.58	0.130	0.109	0.803

* A, B, C, D, E, F = aggregate gradation type.

[1] S_{mix} = predicted creep stiffness at 60 min after regression.

[2] I = interception; strain, % at 1 sec.

[3] S = slope; strain, % = I*(time,sec)**S.

[4] R*R= coefficient of determination.

Table 5.5 Creep Test Results (Continued).

(d) Blue Mountain Asphalt Pit, Gravel, 1% Lime, Chevron AC-20

Sample ID *	S_{mix}[1] (ksi)	I[2]	S[3]	R*R[4]
A38	5.34	0.137	0.084	0.939
A39	4.91	0.182	0.059	0.922
A40	2.31	0.148	0.176	0.991
B32	2.24	0.270	0.107	0.942
B33	2.99	0.188	0.116	0.945
B34	2.57	0.175	0.143	0.984
C29	2.61	0.182	0.137	0.965
C30	2.42	0.243	0.110	0.984
C31	1.48	0.358	0.123	0.970
D35	3.90	0.094	0.169	0.956
D36	2.17	0.206	0.143	0.968
D37	2.88	0.190	0.119	0.967
E38	5.01	0.031	0.273	0.943
E39	5.86	0.027	0.269	0.941
E40	4.25	0.012	0.409	0.952

* A, B, C, D, E = aggregate gradation type.

[1] S_{mix} = predicted creep stiffness at 60 min after regression.

[2] I = interception; strain, % at 1 sec.

[3] S = slope; strain, % = I*(time,sec)**S.

[4] R*R= coefficient of determination.

Creep strain and creep stiffness can be determined by the following equations:

$$\epsilon_c = \Delta h/H \qquad [6]$$

where ϵ_c = creep strain,

Δh = deformation at time t, and

H = thickness of specimen.

$$S_{mix}(T,t) = \sigma/\epsilon(T,t) \qquad [7]$$

where $S_{mix}(T,t)$ = creep stiffness at temperature T and time t,

σ = compressive stress, and

$\epsilon(T,t)$ = creep strain at temperature T and time t.

The creep stiffness of each sample presented in Table 5.5 is the predicted value after regression analysis using the measured stiffness. Figure 5.3 shows a mix stiffness (S_{mix}) as a function of bitumen stiffness (S_{bit}). Bitumen stiffness was obtained by using the Van der Poel bitumen stiffness nomograph with the asphalt properties (P.I. and Softening Point) and a range of loading time.

5.4.3 Rut Depth

In order to predict the rut depth due to the increased tire pressure, the relationships between S_{mix} and S_{bit} resulting from the creep test were used. Physical properties of the asphalt cement, and vertical compressive stress (shown in Figure 5.4) for a typical asphalt pavement structure in Oregon (SN = 3.0, Figure 5.5) were used. The Shell method (6) was employed to predict the rut depth of the asphalt layer in the given pavement structure. An 18 kip single axle with dual tires, and tire pressure of 80 psi (i.e., assumed tire pressure in previous pavement design) and 125 psi (possible tire pressure for future pavement

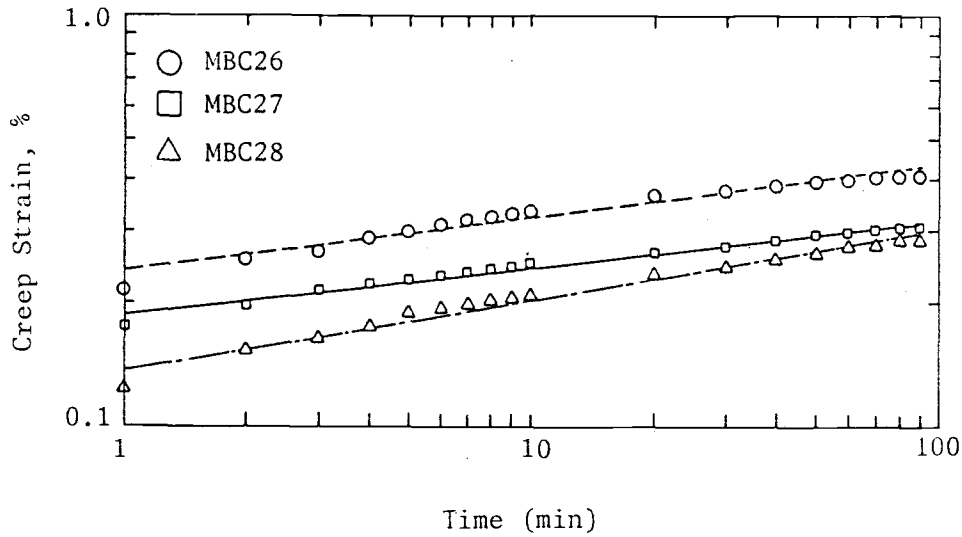


Figure 5.2 Creep Strain vs. Time.

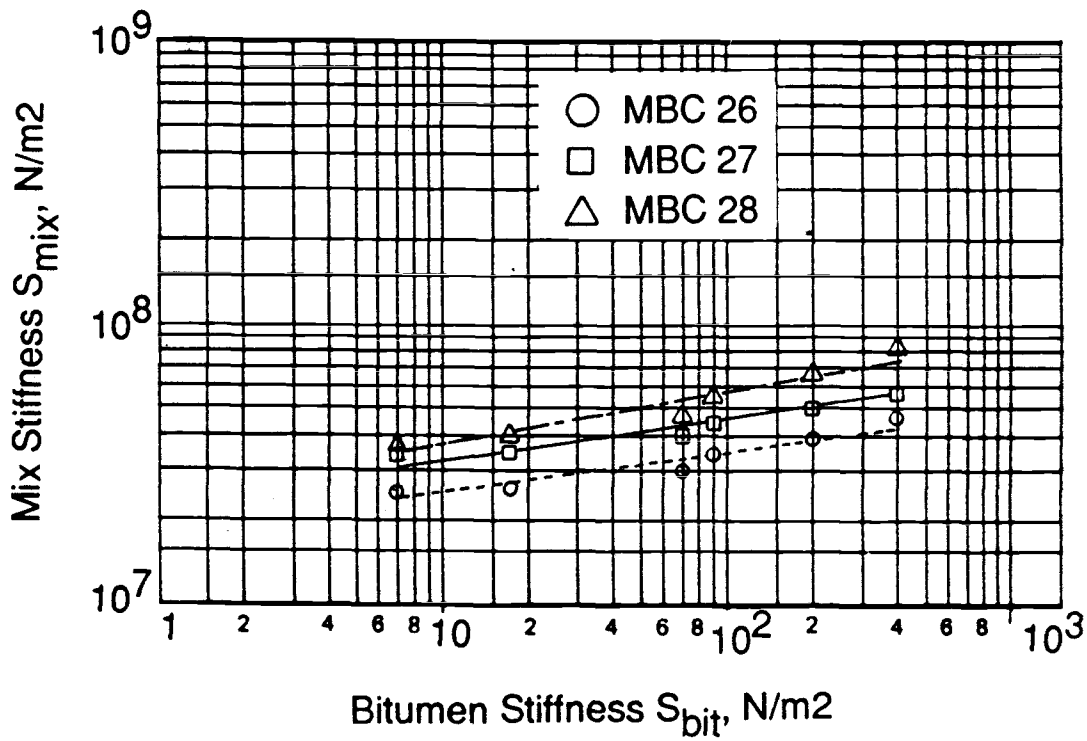


Figure 5.3 S_{mix} vs. S_{bit}.

design) were used.

According to Van de Loo (15), the permanent deformation in the asphalt layer can be calculated by the following equation;

$$\delta = C_M H_o \sigma_{avg} / S_{mix} \qquad [8]$$

where δ = reduction in layer thickness,

C_M = correction factor for the so-called dynamic effect, which takes account of differences between static (creep) and dynamic (rutting) behavior [this factor depends on the type of mix and must be determined empirically],

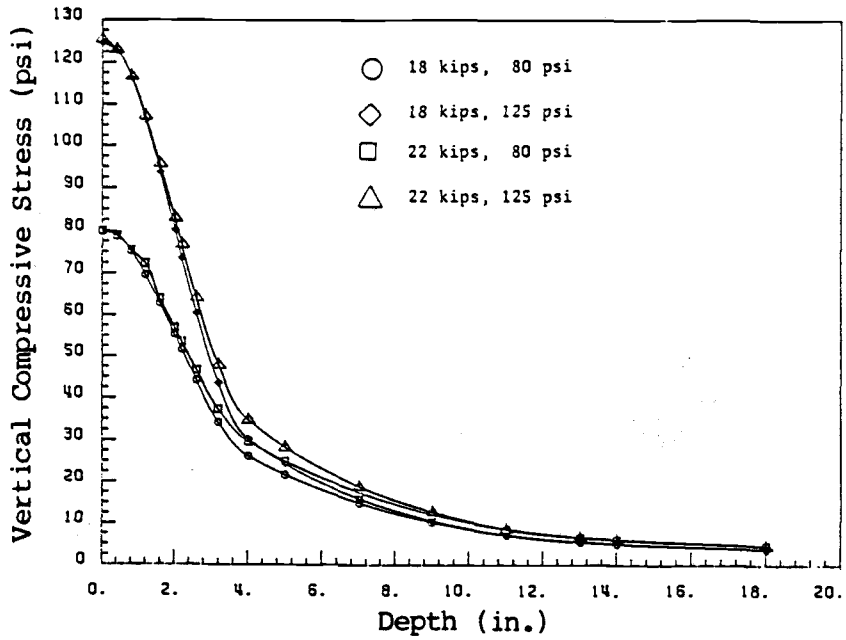
H_o = design thickness of the asphalt layer,

σ_{avg} = average stress in the pavement under the moving wheel, and

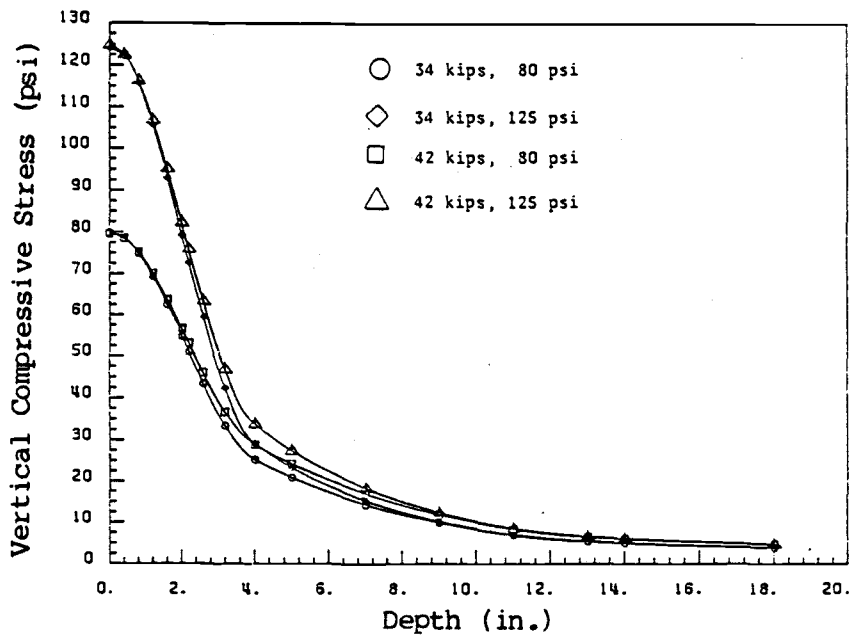
S_{mix} = value of stiffness of the mix at $S_{bit} = S_{bit.visc}$.

In order to determine the vertical compressive stress ELSYM5 (16) was used. Values of the input parameters (modulus, thickness, and Poisson's ratio) of each layer were selected to represent Oregon pavements designed for medium traffic levels.

Table 5.6 presents the average vertical compressive stresses calculated from the output of ELSYM5, and Table 5.7 presents the predicted rut depth for the asphalt surface layer (thickness is 2 inches). The Penetration Index is -1.4 (for an AR-4000 grade asphalt cement) and the loading time is 0.0125 sec. (corresponding to a speed of 50 mph). The number of load repetitions was one million and the correction factor C_M was 1.2. Using Eq. [8], the rut depth for a tire pressure of 80 psi is 0.022 inch, while that for 125 psi is 0.034 inch after one million load repetitions.



(a) Single Axle Dual Tires



(b) Tandem Axle Dual Tires

Figure 5.4 Vertical Compressive Stress.

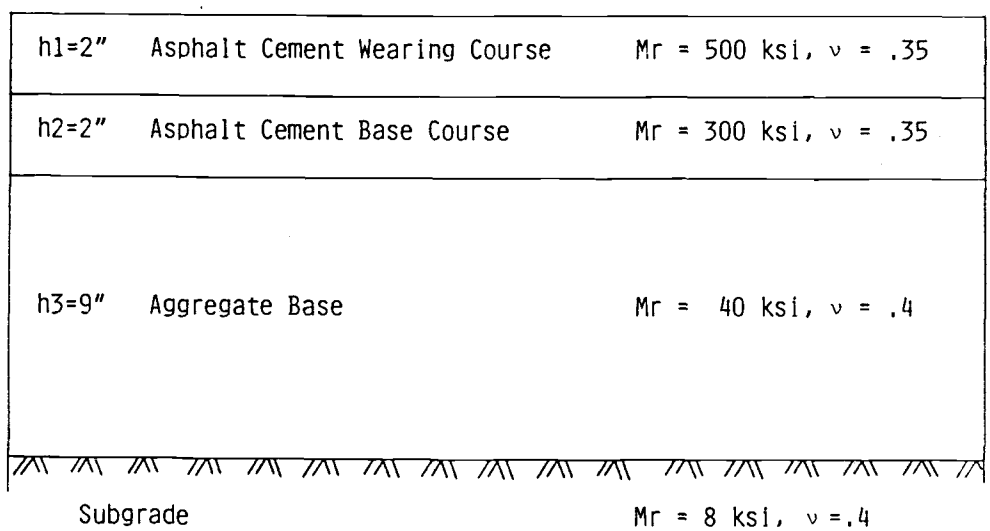


Figure 5.5 Typical Asphalt Pavement in Oregon (SN=3.0).

For this paper only one set of calculations for the "C" gradation mixes of Morse Brothers Pit was presented for the purpose of demonstration. Since the resilient modulus of asphalt layer is varied with different mixtures, the modulus value for ELSYM5 should correspond to the resilient modulus test results.

More detailed data on the rut depth calculation are presented in Reference 4.

5.5 DISCUSSION

5.5.1 Mix Design

Table 5.4 summarizes the mix design results of laboratory compacted mixes. The stability is considered to be most significant in relation to this study. ODOT requires a minimum Hveem stability of 30. The stability is varied with traffic as recommended by the Asphalt Institute for the Hveem method (17).

As presented in Table 5.8 (a), the correlation between log(Hveem Stability) and log(Creep Stiffness) is not strong except for the Cobb Rock mixes. According to the results of creep tests, it is not always true that a mix with high stability value resists the deformation better than one with low stability. This shows that the current mix design criteria are probably inadequate to produce mixtures capable of dealing with the high tire pressures, and to identify potentially highly deformable mixtures.

It is noted that gradation "C" mix (the Fuller maximum density gradation) requires the least amount of optimum asphalt content for each aggregate source on the basis of the existing mix design method. Also, gradation "C" has the smallest voids in the mineral aggregate (VMA).

Table 5.6 Average Vertical Compressive Stress (psi).

(a) Single Axle, Dual Tires

18 kips		22 kips	
80 psi	125 psi	80 psi	125 psi
70.7	108.2	71.8	109.4

(b) Tandem Axle, Dual Tires

34 kips		42 kips	
80 psi	125 psi	80 psi	125 psi
70.4	107.6	71.1	108.8

Table 5.7 Predicted Rut Depth under Given Conditions.

Tire Pressure (psi)	Rut Depth (in.)
80	0.022
125	0.034

Conditions: AR 4000 (P.I. = -1.4),
Asphalt Pavement (SN = 3.0) in Figure 5.5,
H_o = 2.0 in.,
Number of Repetitions = 10^6,
MAAT = 20 C.

Table 5.8 Correlation Analysis.

(a) Correlations with log(Creep Stiffness, ksi)

Variables	Morse Brothers Pit	Cobb Rock	Hilroy Source	Blue Mountain Asphalt Pit
log(Stability)	-0.3141	0.8176	0.4878	-0.0482
log(M_R; As-Comp., ksi)	0.0636	-0.0859	0.5004	-0.2771
log(M_R; Cond., ksi)	0.2664	-0.4886	-0.1981	-0.7012
log(M_R Ratio)	0.2428	-0.5353	-0.3665	-0.3592
log(A/C, %)	-0.0906	-0.2440	-0.4839	-0.3310
log(Max. Sp.Gr.)	0.1638	0.2542	0.4825	0.3015
log(Air Voids, %)	-0.1736	0.7529	0.3890	0.5625
log(VMA)	N/A	0.5805	0.0615	0.7465
log(Pass 1/4-in., %)	-0.4197	0.2196	-0.4026	0.5609
log(Pass #10, %)	0.1970	-0.5034	0.0897	-0.1731
log(Pass #200, %)	-0.3141	-0.6766	-0.1416	-0.3799
log(Intercept)	-0.6955	-0.7780	-0.3532	-0.7038
log(Slope)	-0.4395	-0.2410	-0.3908	-0.5329

Table 5.8 Correlation Analysis (Continued).

(b) Correlations with log(Slope)

Variables	Morse Brothers Pit	Cobb Rock	Hilroy Source	Blue Mountain Asphalt Pit
log(Stability)	-0.5737	-0.1073	-0.0163	0.4056
log(Creep Stff., ksi)	-0.4395	-0.2410	-0.3908	-0.5329
log(M_R; As-Comp., ksi)	-0.1814	0.5060	-0.3602	0.2600
log(M_R; Cond., ksi)	-0.0878	0.4838	0.3963	0.2604
log(M_R Ratio)	0.0993	0.3077	0.4671	-0.0078
log(A/C, %)	0.3817	-0.0476	0.5256	0.0436
log(Max. Sp.Gr.)	-0.3476	0.0459	-0.4687	0.0079
log(Air Voids, %)	-0.3252	-0.2107	-0.2363	-0.2589
log(VMA)	N/A	-0.7506	-0.0819	-0.4468
log(Pass 1/4-in., %)	0.4420	-0.5647	-0.2625	-0.4437
log(Pass #10, %)	-0.0317	0.6751	-0.0743	0.2183
log(Pass #200, %)	-0.5737	0.6777	-0.0215	0.0439
log(Intercept)	-0.3388	-0.4176	-0.5332	-0.2156

Table 5.8 Correlation Analysis (Continued).

(c) Correlations with log(Intercept)

Variables	Morse Brothers Pit	Cobb Rock	Hilroy Source	Blue Mountain Asphalt Pit
log(Stability)	0.7761	-0.7241	-0.3974	-0.2351
log(Creep Stff., ksi)	-0.6955	-0.7780	-0.3532	-0.7038
log(M_R; As-Comp., ksi)	0.0714	-0.2807	-0.1320	0.1409
log(M_R; Cond., ksi)	-0.2211	0.1370	-0.4243	0.5916
log(M_R Ratio)	-0.3393	0.3109	-0.2871	0.3855
log(A/C, %)	-0.2007	0.2766	-0.1660	0.2983
log(Max. Sp.Gr.)	0.0961	-0.2882	0.0705	-0.2930
log(Air Voids, %)	0.4335	-0.6008	0.1323	-0.4049
log(VMA)	N/A	-0.0696	0.2844	-0.5163
log(Pass 1/4-in., %)	0.0795	0.1491	0.5427	-0.3455
log(Pass #10, %)	-0.1846	0.0341	-0.1375	0.0135
log(Pass #200, %)	0.7761	0.1812	-0.1531	0.4049
log(Slope)	-0.3388	-0.4176	-0.5332	-0.2156

Table 5.8 Correlation Analysis (Continued).

(d) Correlations with log(Stability)

Variables	Morse Brothers Pit	Cobb Rock	Hilroy Source	Blue Mountain Asphalt Pit
log(Creep Stff., ksi)	-0.3141	0.8176	0.4878	-0.0482
log(M_R; As-Comp., ksi)	0.0471	0.1153	0.3026	0.0361
log(M_R; Cond., ksi)	-0.2101	-0.3435	-0.2735	0.0017
log(M_R Ratio)	-0.2987	-0.4685	-0.3810	-0.3332
log(A/C, %)	-0.4433	-0.4636	-0.4805	-0.4824
log(Max. Sp.Gr.)	0.3579	0.5197	0.4139	0.5657
log(Air Voids, %)	0.7820	0.9546	0.6501	0.3984
log(VMA)	N/A	0.4529	0.0179	-0.0909
log(Pass 1/4-in., %)	0.1302	0.2283	0.0664	-0.6330
log(Pass #10, %)	-0.2928	-0.2220	0.0104	-0.0017
log(Pass #200, %)	1.0000	-0.4696	0.2500	-0.1198
log(Slope)	-0.5737	-0.1073	-0.0163	0.4056
log(Intercept)	0.7761	-0.7241	-0.3974	-0.2351

In general, the optimum asphalt content from the existing mix design method is higher than that required to achieve the retained modulus ratio (MMRT) of 0.7 except for the mixes from Hilroy Source aggregate.

It seems to be necessary to study further which mix design criteria including creep stiffness should be considered and how to determine the optimum asphalt content of a mix in order to resist the rutting and show good durability.

5.5.2 Creep Behavior of Mixes

The creep behavior of an asphalt mixture can be interpreted by the slope obtained after regression analysis and creep strain or creep stiffness. To analyze the effect of some mix variables, including aggregate gradation on creep behavior, the correlation analysis among the variables indicated in Table 5.8 was made. In general, the creep stiffness decreases with the increasing percentage of passing the #200 sieve size, as presented in Table 5.8.

With the limited data, the effect of percentage of passing 1/4-in. or #10 sieve size on the creep stiffness is not clear. With regard to the passing 1/4-in. or #10 sieve size, however, the results concerning the creep stiffness of the aggregates from the Morse Brothers Pit and the Hilroy Source show one similar trend (i.e., negative correlation with the percentage of passing 1/4-in. sieve sizes and positive correlation with the percentage of passing #10 sieve size). And, the results from the Cobb Rock and the Blue Mountain Asphalt Pit which were mixed with slurry lime of one percent indicate another similar trend (i.e., positive correlation with the percentage of passing 1/4-in. sieve size

and negative correlation with the percentage of passing #10 sieve size).

For four aggregate sources, the creep stiffness has negative correlation with the intercept (which shows the deformation characteristics at the initial stage) or slope (which shows the resistance to the deformation).

The slope decreases with the increasing of the percentage of passing 1/4-in., except for the Morse Brothers Pit.

Mixes with the Morse Brothers Pit aggregate show a similar trend to those with the Hilroy source (i.e., the slope has negative correlations with both percentages passing the #10 and #200 sieves) and mixes with the Cobb Rock aggregate have a similar trend with the Blue Mountain Asphalt Pit (i.e., the slope has positive correlations with both percentages passing the #10 and #200 sieve size particles).

For the Cobb Rock aggregate and the Blue Mountain Asphalt Pit aggregate mixed with 1% lime slurry, the slope increases with the increasing of passing #10 and #200.

From the results of the mix design, it can be noted that adding 1% lime slurry improves not only the durability of the asphalt mix, as presented by the retained modulus ratio in Table 5.4, but also the resistance to deformation. This may be due in part to the increased strength of mix from the addition of the lime. But, there still needs to be investigation into the effect of lime slurry on the permanent deformation of asphalt mixes.

It should be noted that the creep stiffness of Gradation "C" mix (Fuller maximum density gradation) is not the highest in the range of asphalt content tested in this study as shown in Figure 5.6, even though

the mix with gradation "C" has the smallest VMA (Table 5.4).

For the Hveem stability, the mix with the Cobb Rock aggregate has high correlation between log(Stability) and log(Creep Stiffness).

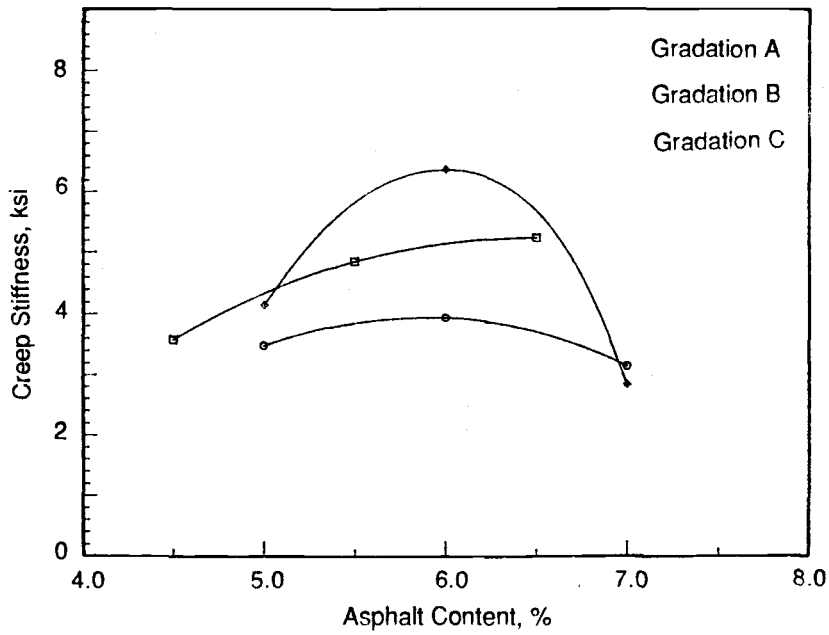
As can be seen in Figure 5.6, the relationship between asphalt content and creep stiffness (at 60 minutes) is not clear. The stiffness of a mix from different aggregate sources and/or different gradations is unique.

5.5.3 Rut Depth

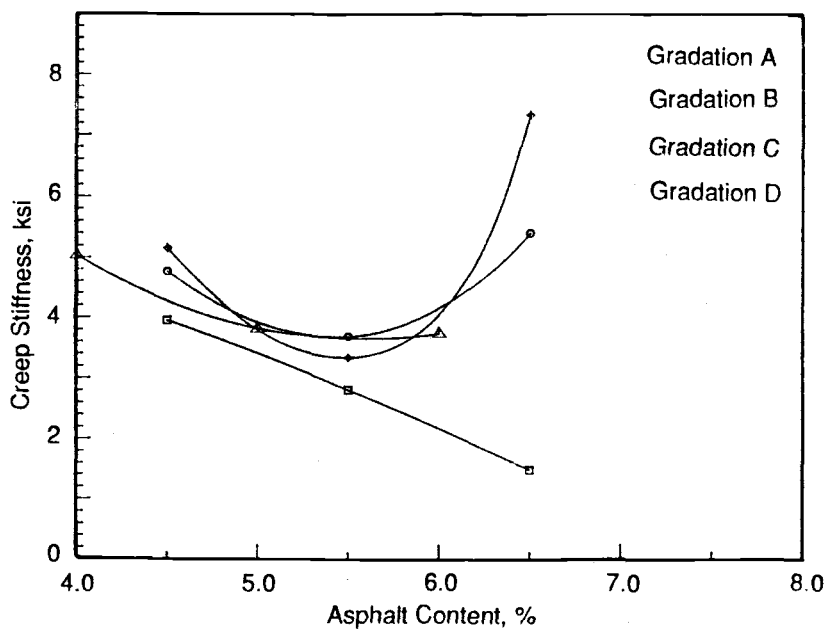
In order to predict the rut depth in asphalt surface layer, the Shell method was employed. For the rut depth calculation, the creep test results of "C" gradation mixes of Morse Brother Pit were used.

The average vertical compressive stress in asphalt surface layer illustrated in Figure 5.5 is about 90 percent of the inflation tire pressure as presented in Table 5.6. As presented in Table 5.7, the rut depth in the asphalt surface layer increases by 52 percent as the tire inflation pressure increases by 56 percent. Therefore, it can be said that the increase in rut depth for asphalt layer is approximately proportional to the increase in tire inflation pressure.

As indicated by Van de Loo (18), it is essential that the creep curve which is used as input in the calculation procedure is representative of the mix as it will be present in the pavement. Since the creep behavior (i.e., slope of the curve) of laboratory prepared specimens may be quite different from that obtained on cores from pavements due to different compaction effort and different heating process, core samples should be obtained shortly after construction and used for the creep test. Because of this, the prediction of rut depth with laboratory

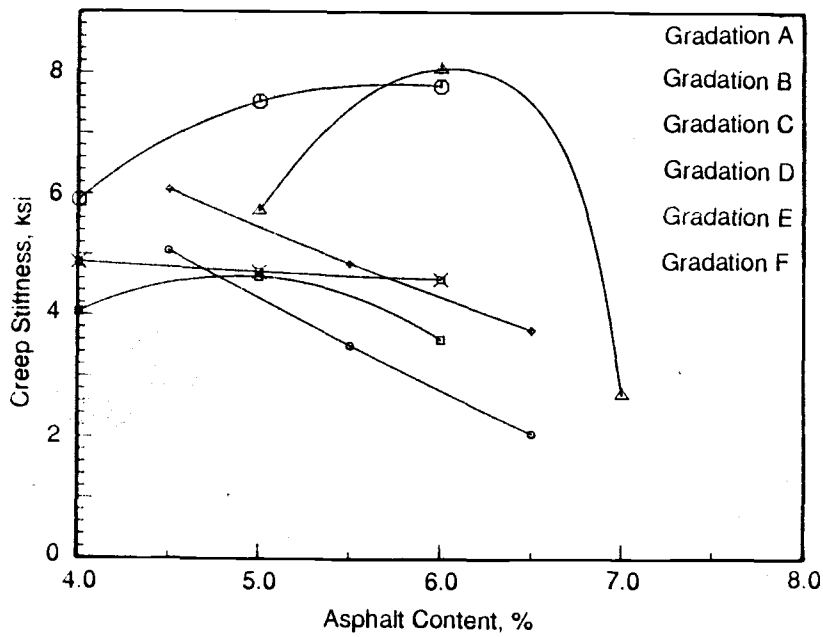


(a) Morse Brothers Pit

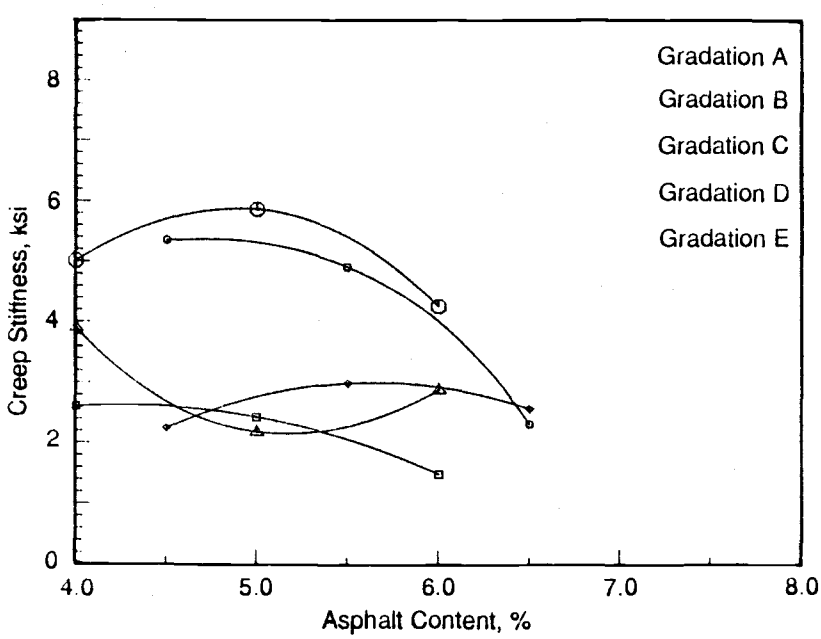


(b) Cobb Rock

Figure 5.6 Effect of Asphalt Contents on Creep Stiffness.



(c) Hilroy Source



(d) Blue Mountain Asphalt Pit

Figure 5.6 Effect of Asphalt Contents on Creep Stiffness (Continued).

specimens is meaningless. However, laboratory prepared specimens can beused to determine the ranking of different mixes.

In this paper, the evaluation was performed mainly on the stability of mixes. For the overall performance of asphalt pavement, however, durability, fatigue characteristic as well as stability of asphalt mixes should be considered in the mix design process.

5.6 CONCLUSIONS AND RECOMMENDATIONS

5.6.1 Conclusions

The mix design process used by Oregon State Highway Division was investigated to evaluate its ability to minimize damage from higher tire pressure. For this study aggregate from four different sources was used. Six different aggregate gradations, including the Fuller maximum density gradation, were tested.

A simple method of creep test to predict deformation of an asphalt mixture which used a loading device for soil consolidation and data acquisition system with a microcomputer was performed.

The major findings and conclusions of this study are:

1. Gradation "C" (the Fuller maximum density gradation) requires the least amount of optimum asphalt for each aggregate source.
2. Hveem stability has little relationship with the creep stiffness. The results of creep tests show that it is not always true that a mix with high Hveem stability value resists creep deformation better than one with low stability. Therefore, for projects where deformation is a major concern, addition of creep tests in the mix design process should be of benefit.

3. The creep stiffness decreases with the increasing percentage passing the #200 sieve size. However, the effect of percentage passing 1/4-in. or #10 sieve size on the creep stiffness is not clear. Control of the passing #200 material clearly contributes to deformation resistance, and, should be given more emphasis in the mix design and construction process.
4. Using 1% lime slurry shows some improvement in creep stiffness.

5.6.2 Recommendations

In order to control the effect of increased tire pressure on asphalt concrete pavement, the following recommendations are made:

1. Include the creep behavior of a mix in mix design criteria, such as creep stiffness, to predict the rut depth due to the increased tire pressure, and/or to rank candidate mixes. As Shell manual indicated, it is essential that the creep curve which is used as an input in the calculation procedure is representative of the behavior of the mix in the pavement. However, a study to correlate laboratory mixture stability (i.e., Hveem Stability, Marshall Stability and creep stiffness) with field deformation is recommended.
2. More investigation is needed into the effect of lime slurry on the permanent deformation of the asphalt mix. The results of this study show some improvement in creep stiffness in those mixtures using lime slurry.
3. The use of other additives should be considered to increase creep stiffness of mixtures.
4. Further study on mix design process dealing with higher tire

pressure is necessary in laboratory and field.

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DISCLAIMER

6.0 EFFECT OF INCREASED TRUCK TIRE PRESSURE ON ASPHALT CONCRETE PAVEMENT

Ok-Kee Kim
Graduate Research Assistant
Department of Civil Engineering
Oregon State University

Chris A. Bell
Associate Professor
Department of Civil Engineering
Oregon State University

and
James E. Wilson
Retired Engineer of Materials
Oregon Department of Transportation

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ABSTRACT

As axle loads have increased, higher tire pressures have become more popular in the truck market. A survey to collect data on current levels of tire pressure was carried out at a weigh station located on Interstate 5 in Oregon during the summer of 1986. The data show that 87% of the tires surveyed are of radial construction. The average measured tire pressures of radial and bias tires are 102 and 82 psi, respectively.

This paper presents the results of the investigation into the influence of increased tire pressures on the fatigue and rutting performance and on the vertical compressive stress of asphalt surfaced pavements. Results of elastic layer analyses (ELSYM5) are presented for two typical Oregon highways (SN=3.0 and 3.4) to determine levels of stress and strain in the pavements. These parameters are used to develop equivalency factors and to calculate the deformation in the asphalt layer using the results of creep tests.

The analyses show that the effect of increased tire pressure on vertical compressive stress is significant in the asphalt surfacing layer. As the pressure increases, the maximum tensile strain at the bottom of the asphalt layer increases.

The theoretical equivalency factors take into account tire pressures (80, 100, 125, and 150 psi) and number of tires per axle (2, 4, and 8 tires). Two outputs of ELSYM5, namely the maximum tensile strain at the bottom of the asphalt base layer and the maximum compressive strain at the top of the subgrade, are considered. An 18-kip single axle with dual tires and a tire pressure of 80 psi were used as a standard axle load and tire pressure. The results indicate

that a 25% increase in tire pressure could result in a 40 to 60% increase in equivalency for a dual tired single axle of 18 kips and a tandem axle of 34 kips.

6.1 INTRODUCTION

6.1.1 Problem Statement

The economics of truck transportation has tended to cause the average gross weight of trucks to increase such that the majority of trucks are operating close to the legal gross loads or axle loads (1). Many states, including Oregon (2), also issue permits for trucks to operate above normal legal load limits.

In 1982, the federal government permitted 80,000 pounds gross vehicle weight, 20,000 pounds single axle weights and 34,000 pounds tandem axle weights on interstate highways. As axle loads have increased, the use of higher tire pressures has become more popular in the truck market. Higher tire pressures decrease the contact area, resulting in reduced tire friction or skid resistance and increased potential for pavement damage due to the higher stresses that are transmitted to the pavement. The higher tire pressures contribute to greater deformation in flexible pavements, manifested as high severity wheel track rutting. Rutting results in hazardous pavements, since ruts cause an uneven pavement and can accumulate water and ice in harsh weather. The higher tire pressures also tend to be accompanied by higher loads, and these will tend to increase the severity of fatigue cracking.

AASHTO equivalency factors (3) used currently by most state and local agencies for pavement design are probably inadequate due to the large range tire pressures that can occur within the truck fleet. Also, the estimates of rut depth due to higher tire pressures are desirable as an element of mix and pavement design.

6.1.2 Objectives

A study of procedures for controlling the effect of increased truck tire pressure on asphalt concrete pavement damage (4) was performed by Oregon Department of Transportation (ODOT) and Oregon State University (OSU).

This paper presents a part of this study, that is, the results of the tire pressure survey that was carried out at a weigh station located on the Interstate 5 during the summer of 1986, and, the results of asphalt pavement analyses which evaluated the range of tire pressures observed.

The objectives of this paper are:

1) To present existing operating characteristics of Oregon's trucks, particularly levels of truck tire pressures.
2) To present and analyze the results of an investigation into the influence of increased truck tire pressures on the fatigue and rutting damage on asphalt pavements making use of ELSYM5 (5), a microcomputer program for elastic layer (6).
3) To develop theoretical equivalency factors for typical Oregon pavements taking into account tire pressures (80, 100, 125, and 150 psi) and number of tires per axle (2, 4, and 8 tires).

6.1.3 Scope

This paper is intended to demonstrate that tire pressure is a significant factor to be considered when assigning equivalency factors, particularly since a wide range of tire pressures are in use. The assessment of pavement damage due to higher tire pressures herein is a theoretical study. The analysis used cannot take account of any differ-

ences between radial and bias tires. However, Wong (7) indicated that radial tires on a hard surface have a more uniform contact pressure than bias tires. Also, Roberts and Rosson (8) indicated that for bias tires, contact pressure can be very non-uniform and localized stress concentration can be very high. This implies that radial tires may be less damaging to pavements than bias tires at the same inflation pressure.

Throughout the paper, the authors have frequently referred to "increased tire pressures". This may lead some to believe that this is a dramatic or sudden upward trend. However, the authors believe that the trend has been very slow and associated with the gradual shift from the use of bias tires to radials. Pavement engineers should be concerned with the effect of tire pressures because there is a significant variation in inflation pressures used. Undoubtedly very high pressures (combined with high load) are extremely damaging to pavements and should be controlled if possible. However, caution is advised when considering the results of analysis such as presented herein for say 80 psi (a typical tire pressure in the 1960's) with 100 psi (a typical tire pressure in the 1980's) because the former were almost exclusively bias tires and the latter are predominantly radial tires.

6.2 BACKGROUND

A recent survey in Texas (9) indicates that trucks typically operate with tire pressures of about 100 psi in that state. The study performed by Roberts and Rosson (8) indicated that the resulting contact pressure between the tire-pavement could be as high as 200 psi. The Texas study shows that for legal axle loads, increasing tire pressure

from 75 to 125 psi for a bias ply tire (10.00-20) can cut the life of the typical thin asphalt concrete pavements of Texas by amounts ranging from 30 to 80%. In addition to the decreased fatigue life of these pavements, a significant increase in the permanent deformation within the surface should occur.

One method of assessing the destructive effect of increased axle loads and tire pressures is through the use of the concept of load equivalency factors (3,10,11,12,13). The load equivalency factor of a given axle loading may be defined as the number of applications of a standard load that is equivalent in destructive effect to one application of the load under consideration; a 80 kN (18-kip) single axle load on dual tires is normally used as the standard load.

Most published equivalency factors included only various axle loads without considering the variables of tire pressure and number of tires per axle. However, one recent paper by Southgate and Deen (14) did present equivalencies which consider tire pressure and axle configuration. The equivalency factors presented in the 1985 AASHTO pavement design guide (3) are based mainly on the data resulting from the AASHTO road test in which tire pressures of 70-80 psi were used. Therefore, it is necessary to investigate the effect of higher tire pressures that are now in use, nearly 30 years after the road test.

From the point of view of asphalt pavement design and rehabilitation strategies including overlay design, the increased tire inflation pressures associated with the axle load configuration is a very important design variable. Adequate consideration of current levels of these factors could result in the refinement of paving mix design, and pavement structure design methods. Also, it is necessary to review the

maintenance schedules and the remaining life of the existing pavements constructed on the basis of truck tire pressures of 80 psi.

6.3 CALCULATION FOR EQUIVALENCY FACTORS

The following two equations referring to the tensile strain at the bottom of the asphalt base layer and the compressive strain at the top of the subgrade are used to calculate the equivalency factors developed in this study:

$$\text{Equivalency Factor (E.F.)} = (\epsilon_{tij}/\epsilon_{tss})^m \qquad [1]$$

and

$$\text{Equivalency Factor (E.F.)} = (\epsilon_{cij}/\epsilon_{css})^b \qquad [2]$$

where ϵ_{tij} = the tensile strain for the i axle load and j tire inflation pressure,

ϵ_{tss} = the tensile strain for the standard axle load (18 kips, single axle, dual tires) and the assumed standard tire pressure (80 psi),

ϵ_{cij} = the compressive strain for the i axle load and j tire inflation pressure,

ϵ_{css} = the compressive strain for the standard axle load (18 kips, single axle, dual tire) and the assumed standard tire pressure (80 psi),

m = 4.5, and

b = 4.48.

The tensile strain and compressive strain are obtained from the output of ELSYM5.

The following sections explain the background of Eqs. [1] and [2].

6.3.1 Fatigue Criteria

The relationship between fatigue failure and tensile strain is represented by the following equation:

$$N_f = K(1/\epsilon_t)^m \qquad [3]$$

where N_f = number of repetitions to failure,

ϵ_t = tensile strain at bottom of asphalt layer, and

K,m = coefficients.

The equivalency factor on the basis of fatigue failure with the same material is the ratio of N_{fS} to N_{fL}, i.e.:

$$\begin{aligned} E.\ F. &= N_{fS}/N_{fL} \\ &= [K(1/\epsilon_{tss})^m]/[K(1/\epsilon_{tij})^m] \\ &= (\epsilon_{tij}/\epsilon_{tss})^m \qquad [4] \end{aligned}$$

where N_{fS} = number of repetitions to failure of standard axle load and tire pressure, and

N_{fL} = number of repetitions to failure of arbitrary axle load and tire pressure.

Since the value of m ranged from 2 to 5 and the average was close to 4.5 in previous studies performed by OSU (15, 16), an m value of 4.5 was chosen for this study.

6.3.2 Rutting Criteria

For a given stress state and material properties, there is a linear relationship between log(ϵc) and log(N_c) for soils. So,

$$N_c = a(1/\epsilon_c)^b \qquad [5]$$

where N_c = number of load applications,

ϵ_c = vertical compressive strain at the top of subgrade, and

a, b = coefficients.

Since the equivalency factor is the ratio of the number of standard load applications (N_{cS}), to the number of arbitrary load applications (N_{cL}),

$$\begin{aligned} E.F. &= N_{cS}/N_{cL} \\ &= [a(1/\epsilon_{css})^b]/[a(1/\epsilon_{cij})^b] \\ &= (\epsilon_{cij}/\epsilon_{css})^b \end{aligned} \qquad [6]$$

In order to determine b, Eq. [5] is rewritten as the following,

$$\epsilon_c = x(1/N_c)^y \qquad [7]$$

So,

$$\log(N_c) = (1/y)\log(x/\epsilon_c) \qquad [8]$$

Finally,

$$N_c = x^{(1/y)} * (1/\epsilon_c)^{(1/y)} \qquad [9]$$

From Eqs. [5] and [9],

$$b=1/y \qquad [10]$$

Shook et al.(17) list the values for x and y for three well known variations of equation [5]:

Methodology	x	y
Shell	2.8	0.25
Chevron	1.05	0.223
Nottingham	2.16	0.28

Since the Chevron method is more conservative, a y value of 0.223 was chosen in this study. Therefore, b is equal to 1/0.223 or 4.48.

6.4 PREDICTION OF RUT DEPTH

The Shell method (18) was employed in order to predict the rut depth due to a range of inflation pressures. The information required for the Shell method is the results from creep tests, physical properties of asphalt cement used, average of the vertical compressive stress

in the asphalt layers and the thickness of asphalt layers used.

According to the Shell method, the permanent deformation in the asphalt layer can be calculated by the following equation;

$$\delta = C_M H_o \sigma_{avg} / S_{mix} \qquad [11]$$

where δ = reduction in layer thickness, inches,

C_M = correction factor for the so-called dynamic effect, which takes account of differences between static (creep) and dynamic (rutting) behavior [this factor depends on the type of mix and must be determined empirically],

H_o = design thickness of the asphalt layer, inches,

σ_{avg} = average stress in the pavement under the moving wheel, psi, and

S_{mix} = value of stiffness of the mix at $S_{bit}=S_{bit.visc.}$, psi.

The rut depth increases with the increasing average vertical compressive stress or the decreasing $S^m{}_{ix}$ according to Eq. [11].

6.5 RESULTS

6.5.1 Operating Characteristics of Oregon's Trucks

A survey to evaluate tire inflation pressures and types of tires in use was carried out at a weigh station located near Woodburn, Oregon on Interstate 5 from July 28 to July 30 and from August 25 to August 31 in 1986. A tire pressure data collection sheet is shown in Figure 6.1. One data collection form represents one truck. The data collection form consists of four parts, as follows:

1) Basic data: date, time, Public Utility Commission (PUC) safety inspection number, inspector, PUC plate number, and commodity.
2) Weather information, including air temperature and pavement temperature.
3) Truck classification used in Oregon's Weigh-In-Motion study.
4) Tire data: axle number, dual/single tire, manufacturer, tire construction (radial/bias), tire size, tread depth, and tire pressure (recommended by manufacturer and measured).

The tires surveyed were divided into three groups for the purpose of analysis:

1) single tires used for steering axles,
2) single tires for non-steering axles, and
3) dual tires for non-steering axles.

Figures 6.2 and 6.3 show the distribution of the recommended maximum tire pressure (cold) by manufacturers and the measured tire pressure for three groups of radial and bias tires. Table 6.1 presents the mean value and one standard deviation of the recommended tire pressure and the measured tire pressure with the distribution of the tire construction. The data collected show that the majority of tires are radials (87% of 2704 tires). The average measured pressures of radial and bias tires are 102 and 82 psi, respectively. The average recommended inflation pressure for radial tires was 102 psi and that for bias tires was 81 psi. About 40% of all radial tires are operated with inflation pressures above 110 psi. The sample included measurements on a total of 270 trucks, of which 56% were 18-wheelers (3-S2) as presented in Table 6.2.

Reference 4 includes more detailed data on the tire pressure survey.

Chris A. Bell
Oregon State University
Dept. of Civil Engineering
Corvallis, OR 97331
503/754-4273

Research Section: 503/378-2318
Permits and Weighmasters: 503/378-2568

TIRE PRESSURE DATA COLLECTION SHEET

BASIC DATA: Test No. (no entry required): _____ Date: _____ Start Time: _____

PUC Safety Inspection No.: _____ Place of Inspection: _____ Inspector: _____

PUC Plate No.: _____ Commodity: _____ Comments: _____

WEATHER: (tick one)

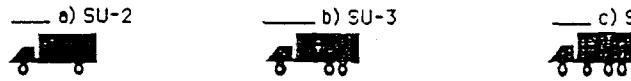
a) Hot & Sunny ___; b) Cool & Sunny ___; c) Hot & Cloudy ___; d) Cool & Cloudy ___; e) Intermittent Showers ___; f) Frequent Showers ___; g) Persistent Rain ___

*Air Temperature ____°F *Pavement Temperature ____°F *Record immediately after start time

TRUCK CLASSIFICATION: (tick one)

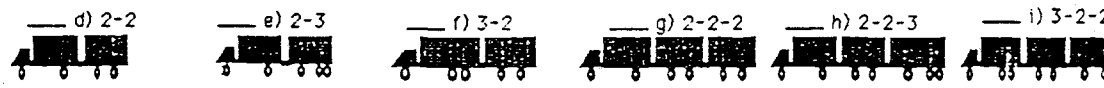
A. Single Units:

___ a) SU-2 ___ b) SU-3 ___ c) SU-4



B. Trucks & Trailers:

___ d) 2-2 ___ e) 2-3 ___ f) 3-2 ___ g) 2-2-2 ___ h) 2-2-3 ___ i) 3-2-2

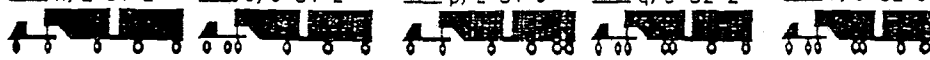


C. Tractors & Semitrailers

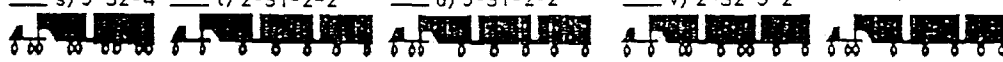
___ j) 2-S1 ___ k) 3-S1 ___ l) 2-S2 ___ m) 3-S2

D. Tractors, Semitrailers & Trailers:

___ n) 2-S1-2 ___ o) 3-S1-2 ___ p) 2-S1-3 ___ q) 3-S2-2 ___ r) 3-S2-3



___ s) 3-S2-4 ___ t) 2-S1-2-2 ___ u) 3-S1-2-2 ___ v) 2-S2-3-2 ___ w) 3-S1-2-3



TIRE DATA:

A. Left Side - Outer Tires

Axle #	Twin/ Single Tire	Mfr. Name	Rec/Max Pressure (psi)	Rad/ Bias (R/B)	Size	Pressure (psi) 1st*/2nd**	Tread Depth†
(1) (strg)							
(2)							
(3)							
(4)							
(5)							
(6)							
(7)							
(8)							
(9)							

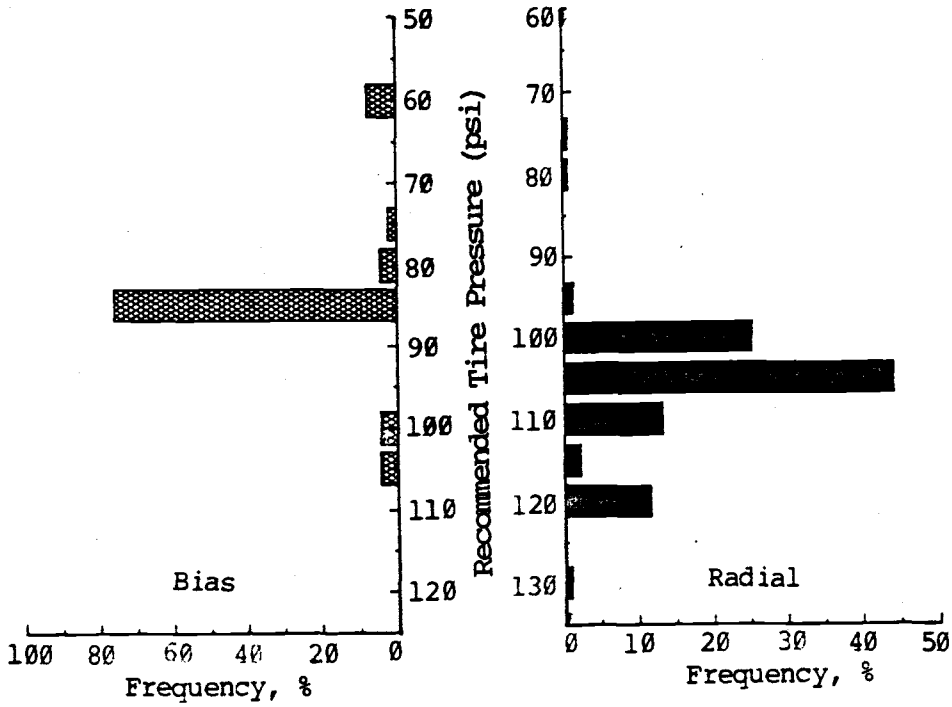
B. Right Side - Outer Tires

Axle #	Twin/ Single Tire	Mfr. Name	Rec/Max Pressure (psi)	Rad/ Bias (R/B)	Size	Pressure (psi) 1st*/2nd**	Tread Depth†
(1) (strg)							
(2)							
(3)							
(4)							
(5)							
(6)							
(7)							
(8)							
(9)							

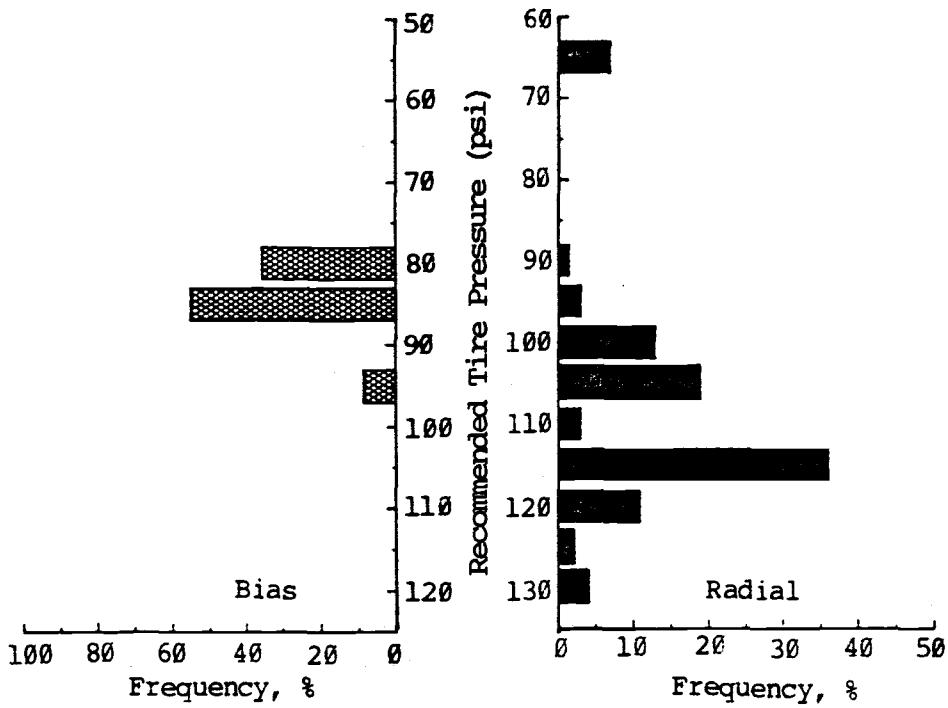
*measured at beginning of inspection; ** measured at end of inspection; †1/32nd in.

Finish time: _____

Figure 6.1 Tire Pressure Data Collection Sheet.

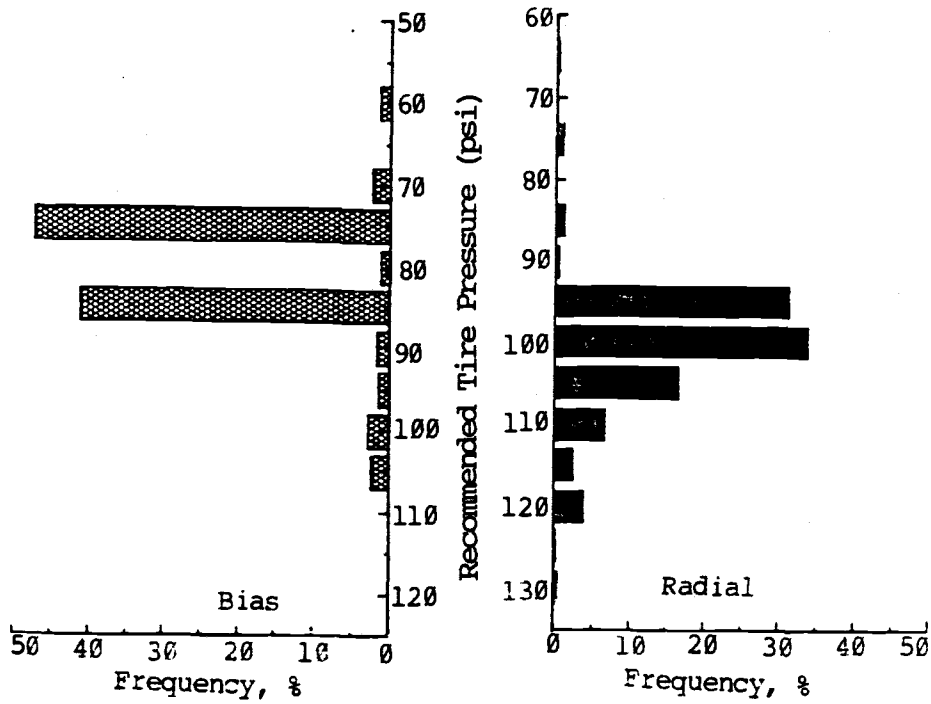


(a) Single Tire, Steering Axle

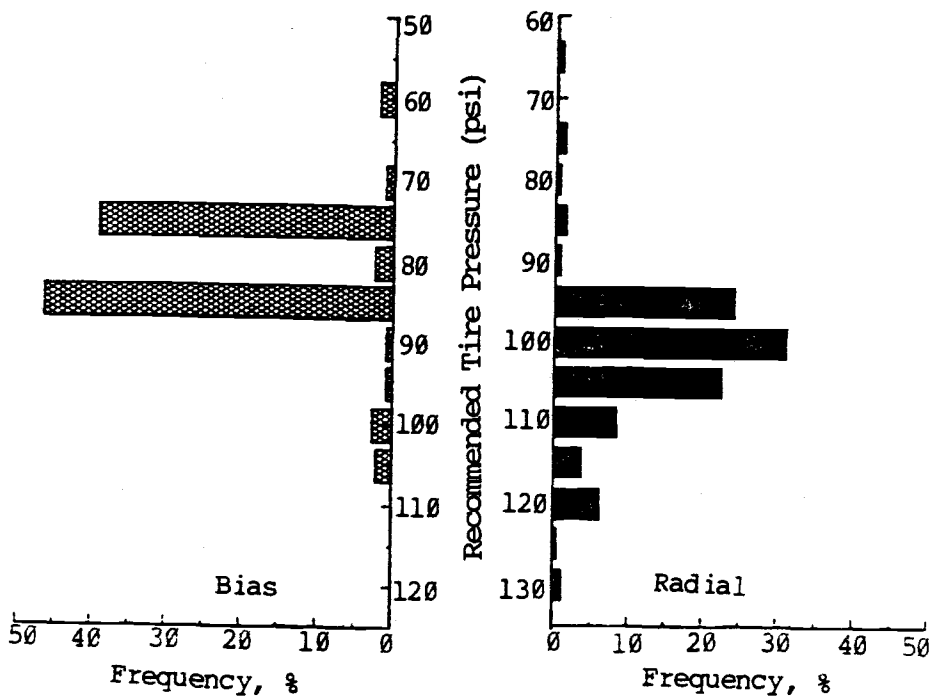


(b) Single Tire, Non-Steering Axle

Figure 6.2 The Distribution of the Recommended Tire Pressure.

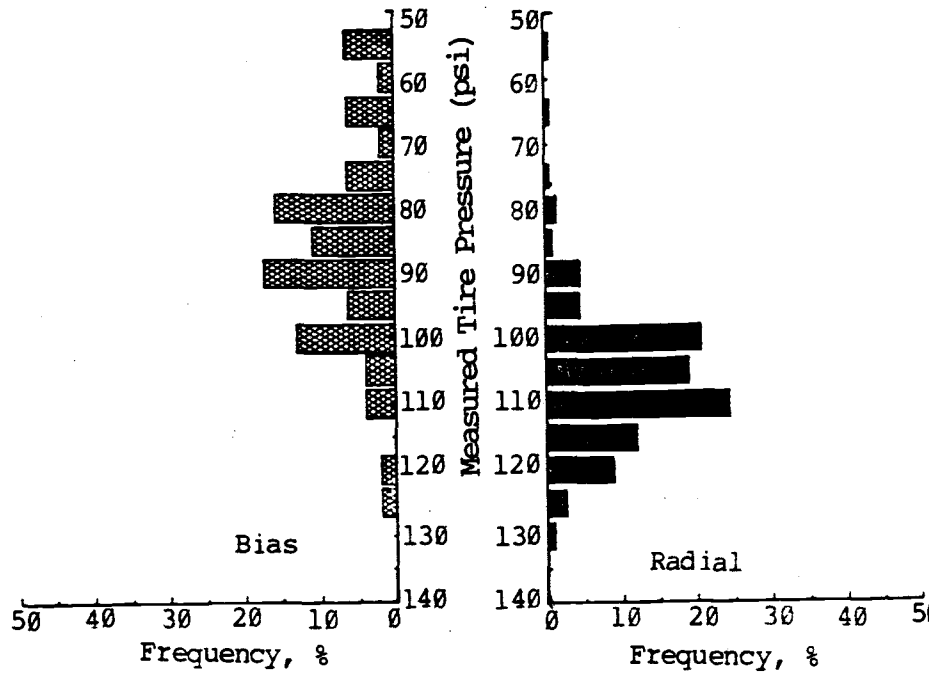


(c) Dual Tires, Non-Steering Axle

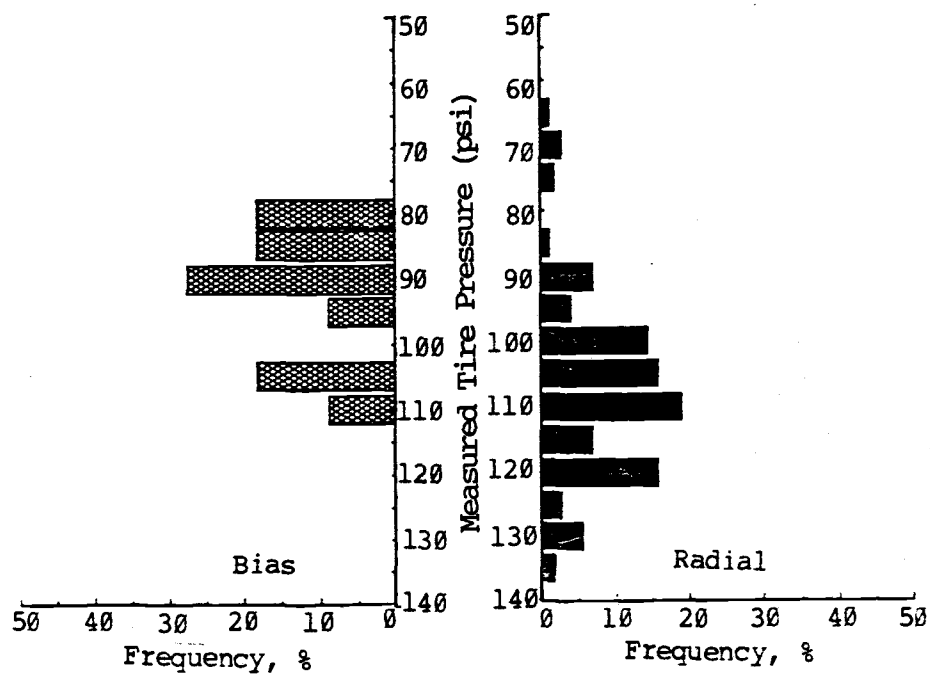


(d) Total Tires

Figure 6.2 Distribution of Recommended Tire Pressure (Continued).

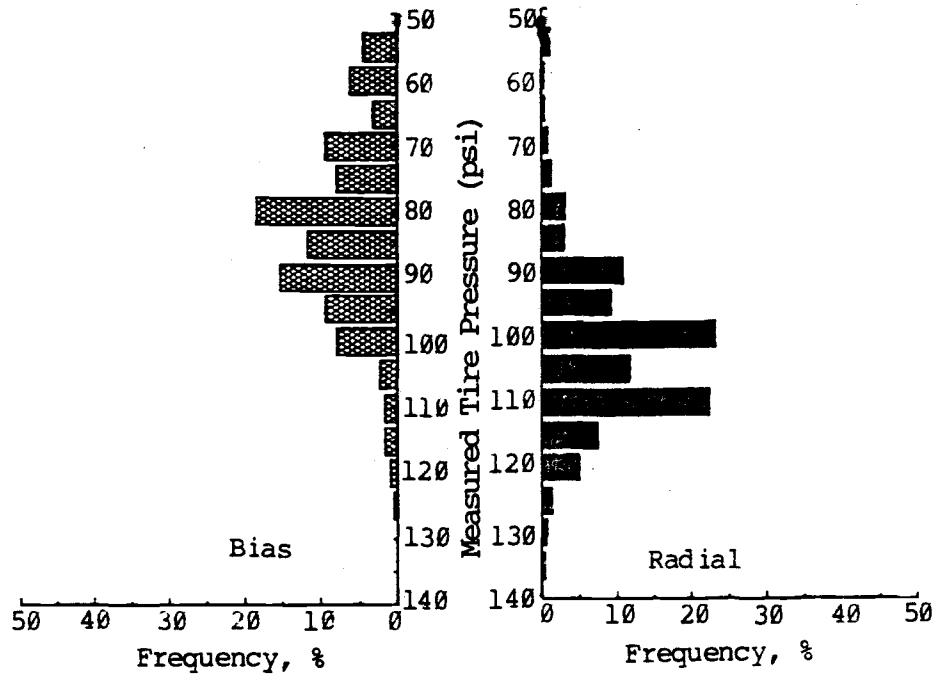


(a) Single Tire, Steering Axle

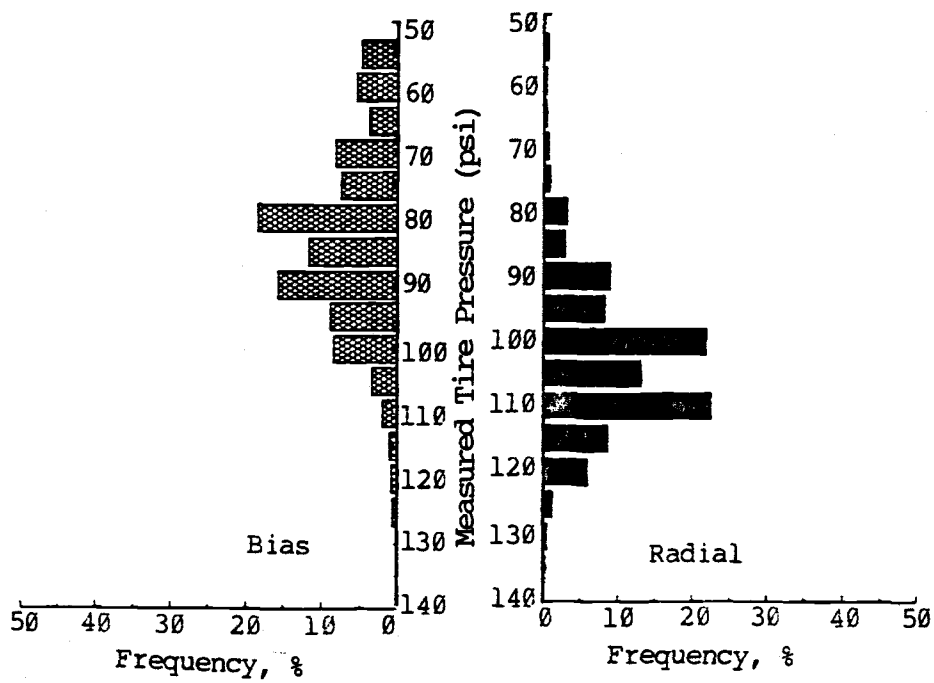


(b) Single Tire, Non-Steering Axle

Figure 6.3 The Distribution of the Measured Tire Pressure.



(c) Dual Tires, Non-Steering Axle



(d) Total Tires

Figure 6.3 Distribution of Measured Tire Pressure (Continued).

Table 6.1 Results of Truck Tire Pressure Survey.

	I*		II		III	
	R**	B	R	B	R	B
A. Tire Construction						
Sample Number	498	46	91	11	1755	292
Sample Frequency (%)	91.5	8.5	89.2	10.8	85.7	14.3
B. Recommended Tire Pressure						
Mean (psi)	106	84	108	84	101	81
One Standard Deviation (psi)	7	9	14	4	8	8
Sample Number	495	46	89	11	1735	285
C. Measured Tire Pressure						
Mean (psi)	106	86	107	93	102	82
One Standard deviation (psi)	10	17	15	10	12	15
Sample Number	498	46	91	11	1755	292
D. Tread Depth (1/32-in.)						
Mean	13	11	12	12	11	9
One Standard Deviation	3.4	3.7	4.3	3.7	4.9	3.4
Sample Number	496	46	88	11	1746	287

Table 6.2 Number of Trucks in the Sample.

Truck Type		Frequency (Number)	Frequency (%)
Single Units	SU-2	11	4.1
	SU-3	9	3.3
Trucks and Trailers	2-3	2	0.7
	3-2	16	5.9
	3-3	4	1.5
	3-4	3	1.1
	4-4	1	0.4
Tractors and Semi-Trailers	2-S1	12	4.4
	3-S1	3	1.1
	2-S2	11	4.1
	3-S2	151	55.9
	4-S2	1	0.4
	2-S3	1	0.4
	3-S3	1	0.4
Tractors, Semi-Trailers and Trailers	2-S1-2	10	3.7
	3-S1-2	11	4.1
	3-S2-2	3	1.1
	3-S2-3	3	1.1
	3-S2-4	1	0.4
	2-S1-2-2	4	1.5
	3-S1-2-2	2	0.7
	2-S1-2-1	1	0.4
Unknown		9	3.3
TOTAL		270	100.0

6.5.2 Analysis of Pavement Structures

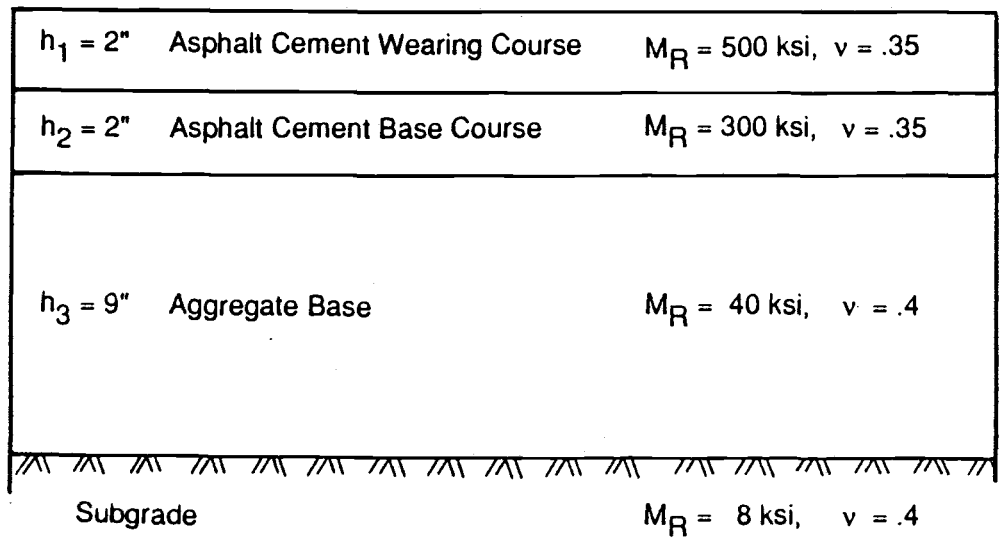
In order to investigate the effects of the increased tire pressures with axle loads on asphalt pavements, ELSYM5 (6) was used with two typical asphalt pavement structures from state highways in Oregon (Figure 6.4).

ELSYM5 determines the various component stresses, strains and displacements along with principal values in a three-dimensional ideal elastic layered system, the layered system being loaded with one or more identical uniform circular loads normal to the surface of the system. The assumptions for ELSYM5 are that the top surface of the system is free of shear, and each layer is of uniform thickness and extends infinitely in the horizontal direction. The contact pressure between tire and pavement surface is assumed to be the same as the tire inflation pressure.

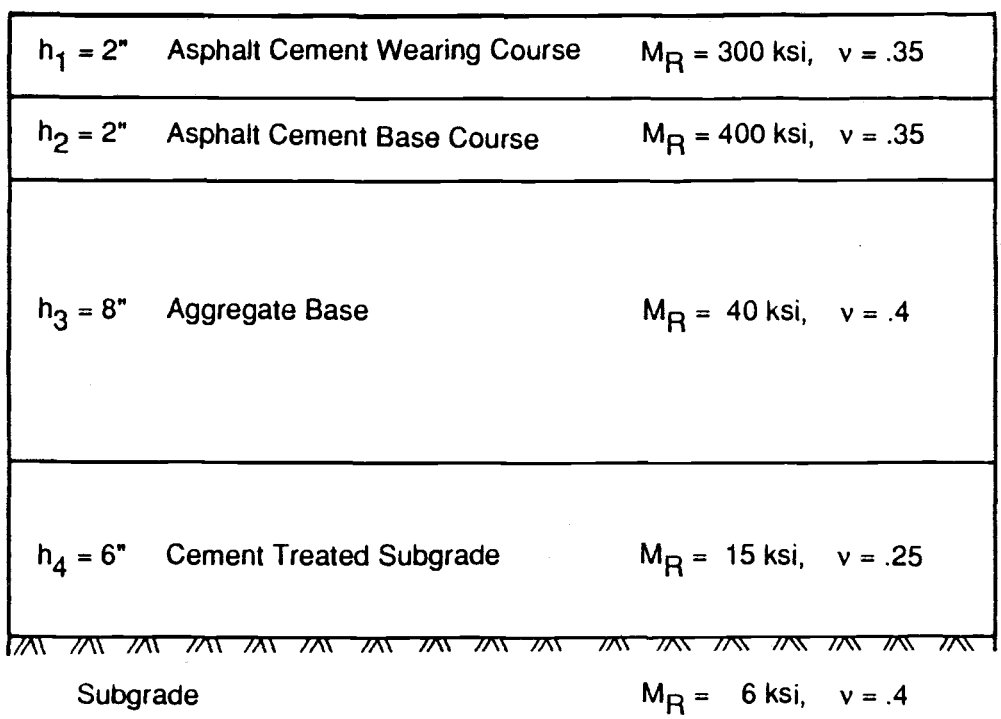
Values of the input parameters (modulus, thickness, and Poisson's ratio) of each layer were selected to represent Oregon pavements designed for medium traffic levels. The structural numbers (SN) for each pavement shown in Figure 6.4 are 3.0 and 3.4, respectively.

Figure 6.5 demonstrates the dimensions for axle and tire configuration used for this analysis.

For the analysis of asphalt pavements shown in Figure 6.4, 18 kips and 22 kips single axles with dual tires, 34 kips and 42 kips tandem axles with dual tires, and tire pressures of 80 psi (i.e., assumed tire pressure in previous pavement design) and 125 psi (possible tire pressure for future pavement design) were used. For the evaluation parameters, the vertical compressive stress through the pavement structure, the horizontal strain in the asphalt concrete wearing course



(a) Asphalt Concrete Pavement A (SN=3.0)



(b) Asphalt Concrete Pavement B (SN=3.4)

Figure 6.4 Typical Asphalt Pavements in Oregon.

structure, the horizontal strain in the asphalt concrete wearing course and base course, and, the vertical compressive strain through the pavement structure at the point below the wheel load, were used and are shown in Figures 6.6, 6.7, and 6.8.

6.5.3 Equivalency Factors

As a standard tire pressure and axle load, 80 psi and 18 kips for single axle dual tires was used. The equivalency factors with reference to this tire pressure and axle load can be computed easily with using the Eqs. [1] and [2] and the maximum tensile strain or maximum vertical compressive strain from the results of ELSYM5.

Table 6.3 presents the equivalency factor for the two asphalt concrete pavements shown in Figure 6.4. The greater equivalency factor between that from the maximum tensile strain and that from the maximum compressive strain was chosen.

6.5.4 Rut Depths

Table 6.5 presents the average vertical compressive stresses calculated from the output of ELSYM5, and Table 6.6 presents the predicted rut depth for the asphalt surface layer (thickness is 2 inches) in Pavement A (Figure 6.4) under the given traffic conditions and asphalt properties. The Penetration Index is -1.4 (for an AR 4000 grade asphalt cement) and the loading time is 0.0125 sec. (corresponding to a speed of 50 mph). The number of load repetitions was one million and the correction factor C_M was 1.2. Using Eq [11], the rut depth for a tire pressure of 80 psi is 0.022 inch, while that for 125 psi is 0.034 inch after one million load repetitions.

Table 6.3 Equivalency Factor.

(a) Pavement Type A in Figure 6.4

Axle Load (kips)	Single Axle Single tire 80	100	125	150	Single Axle Dual Tire 80	100	125	150	Tandem Axle Single tire 80	100	125	150	Tandem Axle Dual Tire 80	100	125	150
2	0.0058	0.0070	0.0082	0.0092	0.0006	0.0007	0.0007	0.0008	--	--	--	--	--	--	--	--
6	0.1803	0.2677	0.3787	0.4868	0.0382	0.0481	0.0585	0.0673	--	--	--	--	--	--	--	--
10	0.6039	1.0033	1.5878	2.2274	0.1960	0.2689	0.3545	0.4327	0.1105	0.1579	0.2153	0.2683	0.0188	0.0231	0.0276	0.0312
14	1.1610	2.0805	3.5437	5.2825	0.5147	0.7484	1.0470	1.3375	0.2683	0.4124	0.6028	0.7942	0.0589	0.0763	0.0953	0.1117
18	2.4988	3.3323	6.0048	9.3609	1	1.5181	2.2182	2.9354	0.4839	0.7846	1.2139	1.6698	0.1302	0.1760	0.2292	0.2767
22	5.4404	6.2211	8.7384	14.138	2.3808	2.5762	3.8966	5.3127	0.7405	1.2567	2.0285	2.8936	0.2361	0.3315	0.4475	0.5542
26	10.252	11.955	13.514	19.322	4.8766	5.0798	6.0851	8.4917	1.0233	1.8016	3.0202	4.4371	0.3779	0.5470	0.7603	0.9657
30	17.370	20.645	23.759	26.176	8.9658	9.3908	9.7625	12.471	1.3210	2.3985	4.1500	6.2619	0.5563	0.8265	1.1778	1.5295
34	27.301	32.971	38.609	42.981	15.227	16.068	16.787	17.263	2.1042	3.0359	5.3918	8.3188	0.8301	1.1685	1.7045	2.2522
38	40.438	49.641	58.993	66.406	24.242	25.736	27.073	27.917	3.2655	3.6917	6.7214	10.574	1.3454	1.5713	2.3436	3.1430
42	--	--	--	--	--	39.313	41.483	42.981	4.8168	5.4622	8.1065	12.984	2.0741	2.1419	3.0911	4.2103
46	--	--	--	--	--	57.619	61.103	63.565	6.8487	7.8086	9.5313	15.522	3.0709	3.1814	3.9444	5.4532
50	--	--	--	--	--	81.589	87.082	90.706	9.3908	10.834	12.198	18.145	4.3959	4.5645	4.9020	6.8679

Due to Fatigue

Due to Deformation

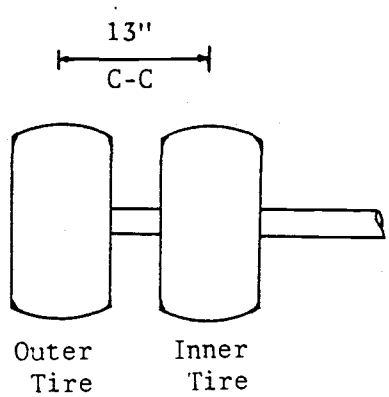
Table 6.3 Equivalency Factor (Continued).

(b) Pavement Type B in Figure 6.4

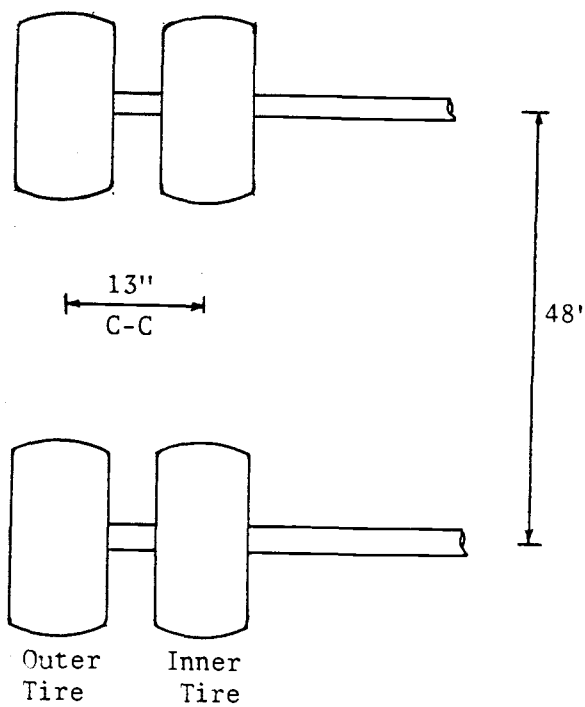
Axle Load	Single Axle Single tire				Single Axle Dual Tire				Tandem Axle Single tire				Tandem Axle Dual Tire			
(kips)	80	100	125	150	80	100	125	150	80	100	125	150	80	100	125	150
2	0.0080	0.0099	0.0118	0.0133	0.0008	0.0009	0.0010	0.0011	--	--	--	--	--	--	--	--
6	0.2128	0.3289	0.4830	0.6356	0.0467	0.0600	0.0756	0.0882	--	--	--	--	--	--	--	--
10	0.6433	1.1192	1.8498	2.6853	0.2211	0.3147	0.4290	0.5366	0.1329	0.1970	0.2781	0.3556	0.0240	0.0302	0.0368	0.0423
14	1.1512	2.1638	3.8657	5.9752	0.5433	0.8249	1.2005	1.5828	0.3023	0.4851	0.7383	1.0018	0.0715	0.0956	0.1226	0.1462
18	1.8023	3.2814	6.2040	10.062	1.0000	1.5906	2.4298	3.3319	0.5168	0.8809	1.4214	2.0223	0.1520	0.2128	0.2862	0.3532
22	4.1373	4.3774	8.6522	14.557	2.3944	2.5929	4.1045	5.8026	0.7571	1.3491	2.2852	3.3784	0.2656	0.3884	0.5422	0.6895
26	8.1576	8.9283	11.070	19.202	4.9332	5.0883	6.1890	8.9670	1.0090	1.8674	3.2814	5.0096	0.4133	0.6230	0.8991	1.1756
30	14.408	16.035	17.448	23.797	9.1383	9.4969	9.7916	12.800	1.2600	2.4082	4.3774	6.8664	0.5899	0.9159	1.3583	1.8179
34	23.498	26.506	29.191	31.064	15.602	16.255	16.872	17.273	1.6551	2.9570	5.5304	8.8875	0.9220	1.2622	1.9210	2.6234
38	36.107	41.304	45.941	49.380	25.006	26.265	27.324	27.992	2.6408	3.5062	6.7217	11.023	1.4991	1.6620	2.5853	3.5935
42	--	--	--	--	--	40.288	41.993	43.278	4.0016	4.2784	7.9300	13.240	2.3179	2.3794	3.3504	4.7184
46	--	--	--	--	--	59.233	62.154	64.216	5.8209	6.2824	9.1276	15.496	3.4432	3.5402	4.1976	6.0116
50	--	--	--	--	--	84.466	88.720	91.860	8.1576	8.8937	10.305	17.784	4.9332	5.0883	5.2241	7.4521

Due to Fatigue

Due to Deformation

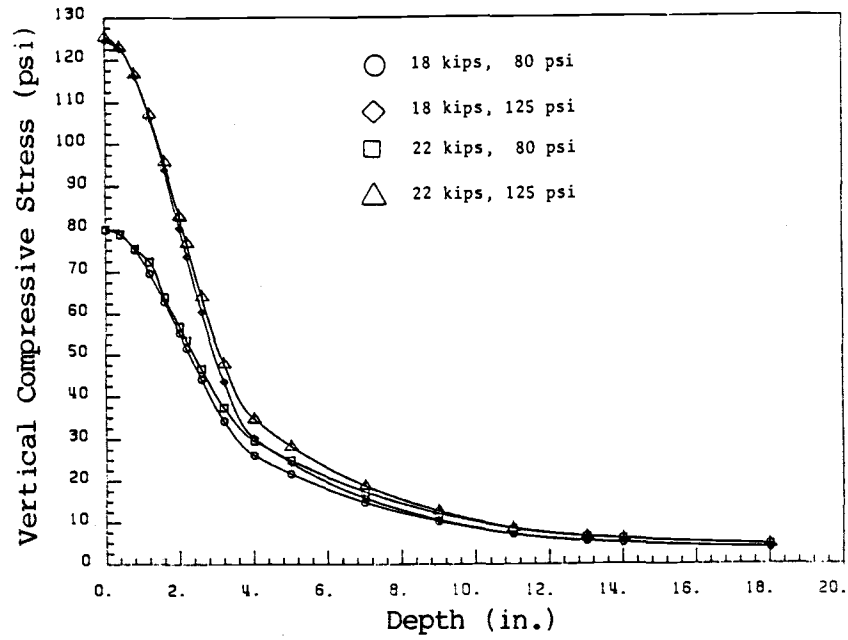


(a) Single Axle Dual Tires

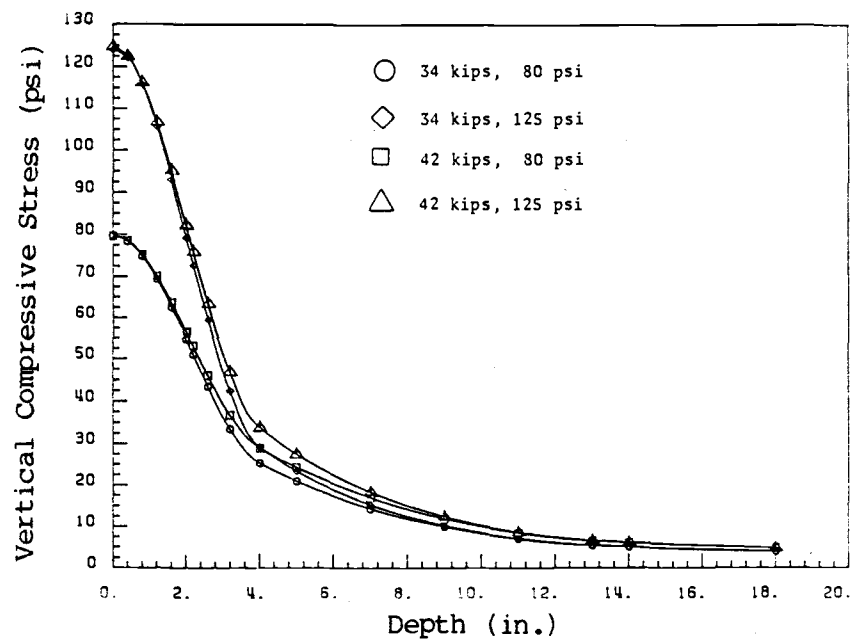


(b) Tandem Axle Dual Tires

Figure 6.5 Axle and Tire Configurations for ELSYM5 Analysis.

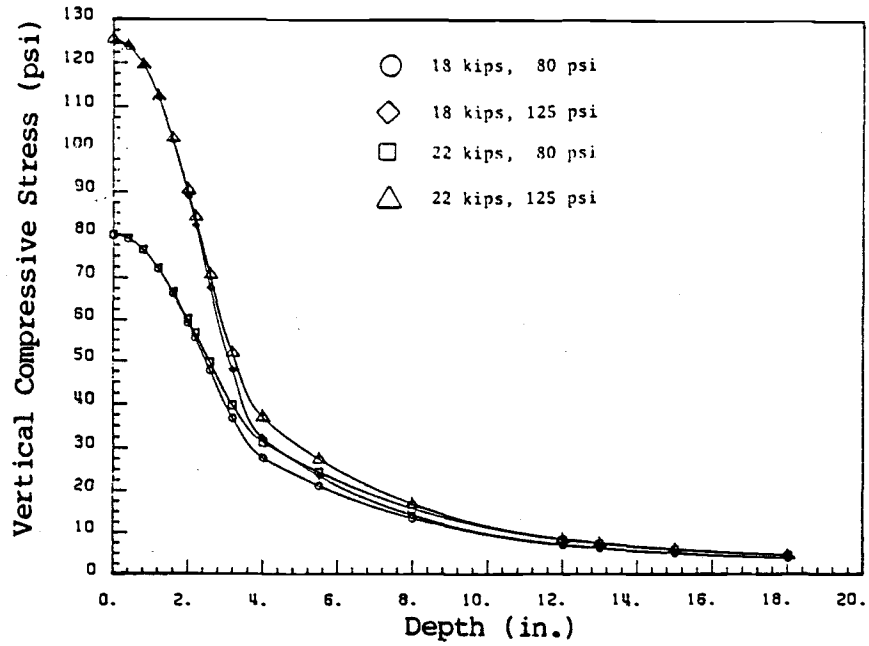


(a) Single Axle Dual Tires - Pavement A in Figure 6.4

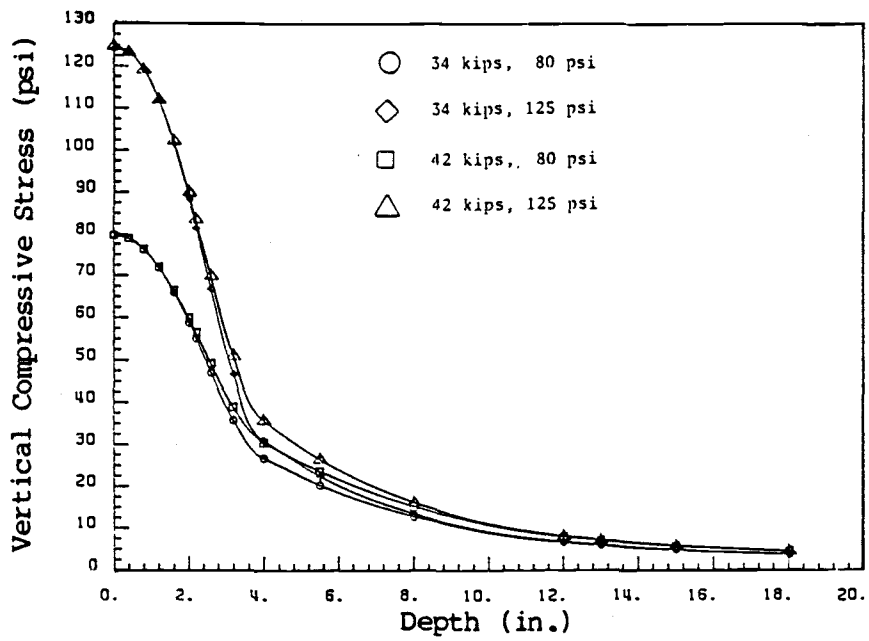


(b) Tandem Axle Dual Tires - Pavement A in Figure 6.4

Figure 6.6 Vertical Compressive Stress.

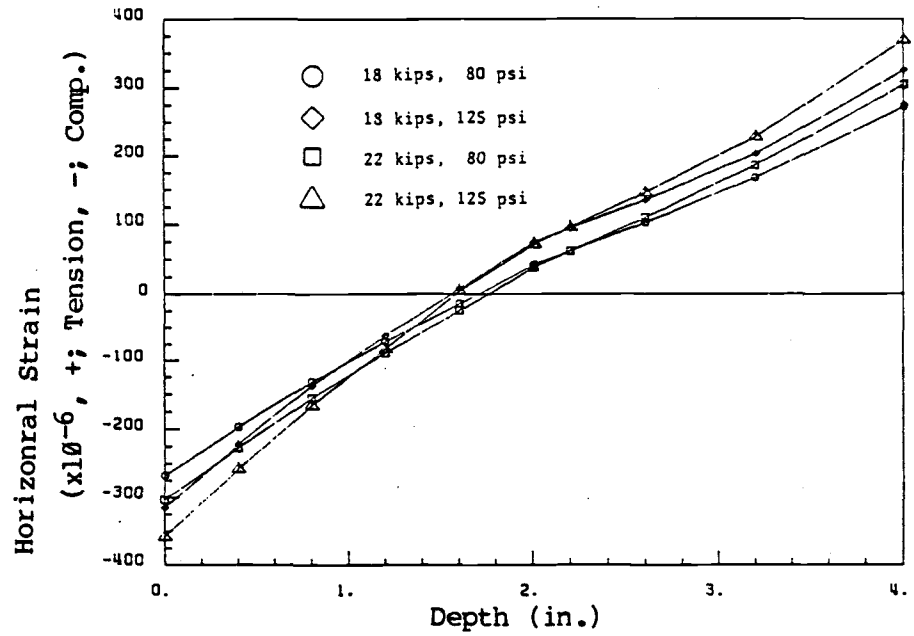


(c) Single Axle Dual Tires - Pavement B in Figure 6.4

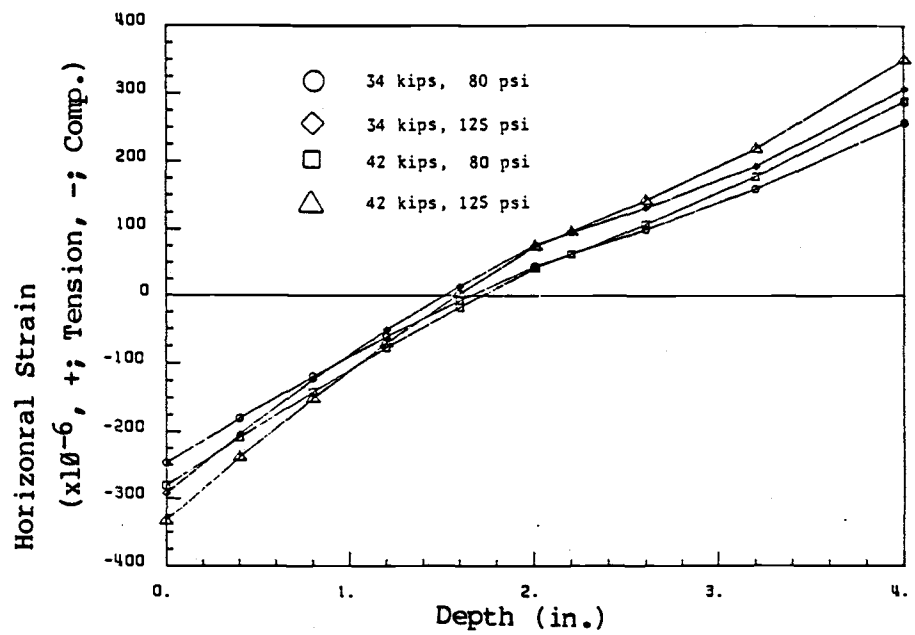


(d) Tandem Axle Dual Tires - Pavement B in Figure 6.4

Figure 6.6 Vertical Compressive Stress (Continued).

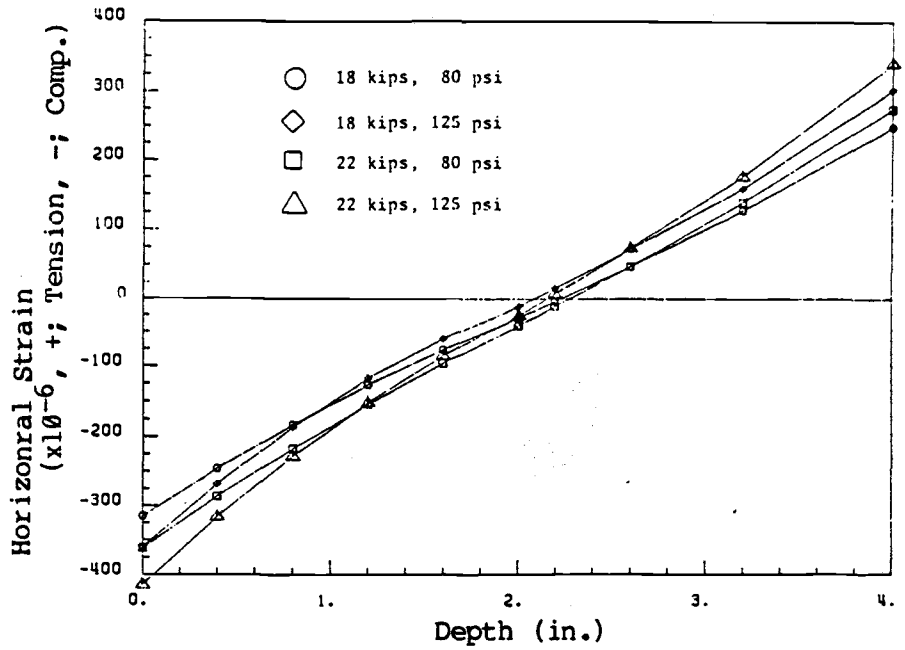


(a) Single Axle Dual Tires - Pavement A in Figure 6.4

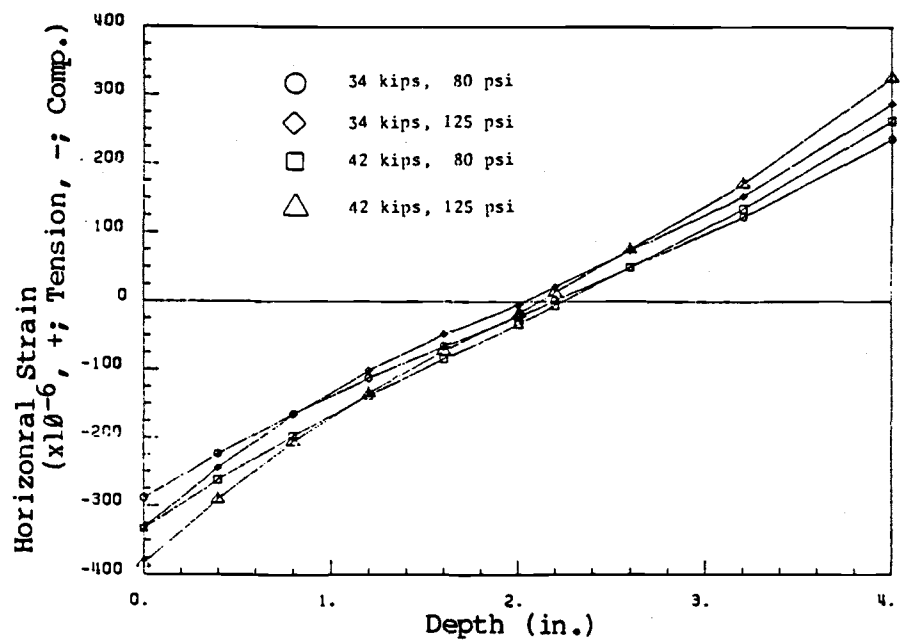


(b) Tandem Axle Dual Tires - Pavement A in Figure 6.4

Figure 6.7 Horizontal Strain in Asphalt Layers.

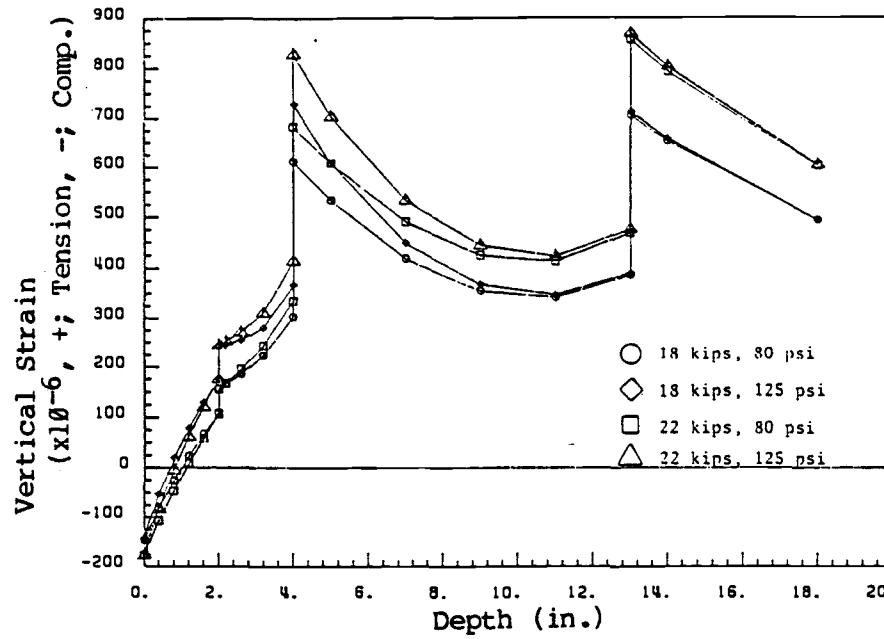


(c) Single Axle Dual Tires - Pavement B in Figure 6.4

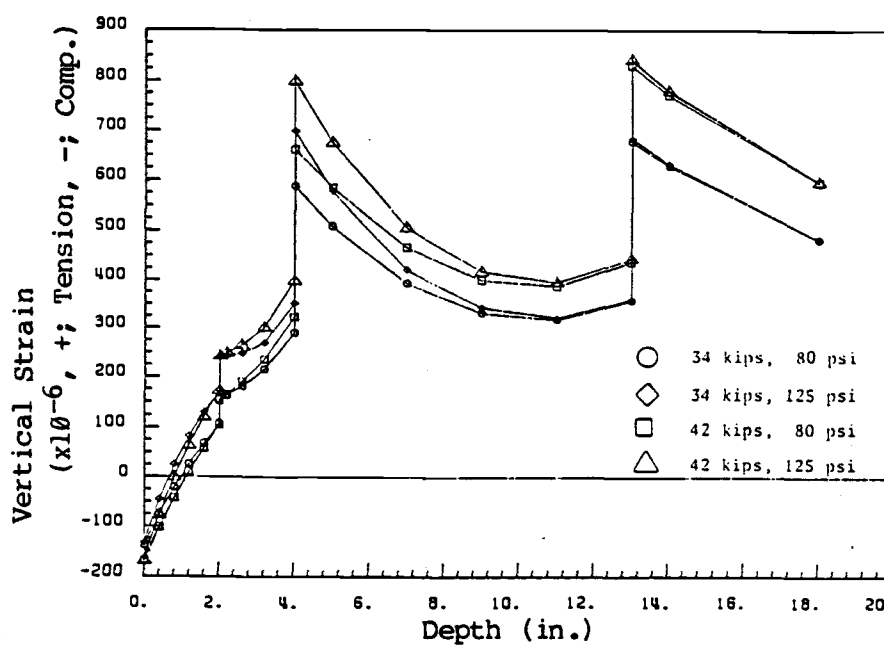


(d) Tandem Axle Dual Tires - Pavement B in Figure 6.4

Figure 6.7 Horizontal Strain in Asphalt Layers (Continued).

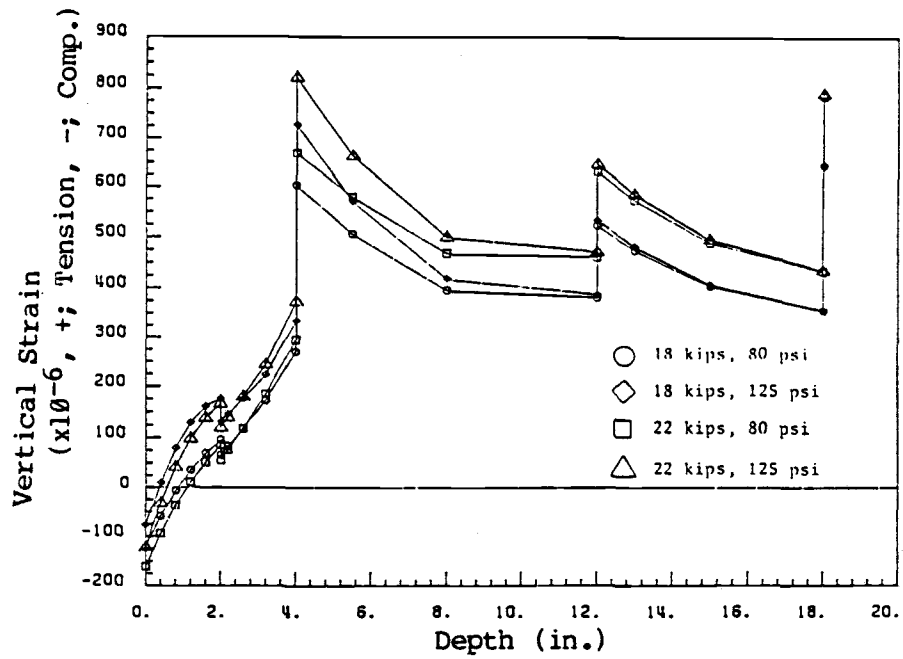


(a) Single Axle Dual Tires - Pavement A in Figure 6.4

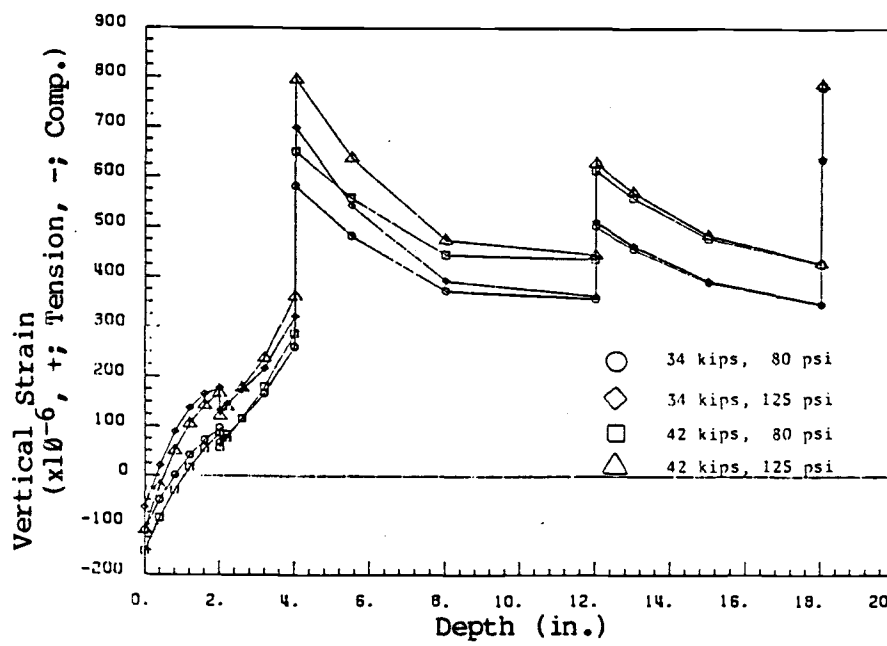


(b) Tandem Axle Dual Tires - Pavement A in Figure 6.4

Figure 6.8 Vertical Compressive Strain.



(c) Single Axle Dual Tires - Pavement B in Figure 6.4



(d) Tandem Axle Dual Tires - Pavement B in Figure 6.4

Figure 6.8 Vertical Compressive Strain (Continued).

More detailed data on the creep test and rut depth calculations are presented in Reference 4.

6.6 DISCUSSION

6.6.1 Tire Pressures

As shown in Table 6.1, the average of the recommended pressure of single tires (for both steering axles and non-steering axles) is higher than that of dual tires. The same trend appears in the measured tire pressure distribution, as presented in Table 6.1. Therefore, the data show that truckers tend to use higher tire pressure for a single tire than for dual tires.

For radial tires, Table 6.1 and Figures 6.2 and 6.3 show that truckers tend to use the manufacturer's recommended tire pressure, due in part to operation safety and efficiency. For bias single tires with non-steering axle, the average of the measured pressure is higher by 10 psi than that of the recommended pressure, but the sample size of 11 tires is very small.

As shown in Table 6.4, the difference between recommended pressure and measured pressure for radial tires is almost zero. However, for bias tires, the inflated tire pressure is greater than the recommended pressure. As presented in Table 6.1, the radial tire pressure is higher by 20 psi than bias tire pressure. The survey performed by Middleton et al. (7) indicated that the radial tires on the average showed 12 to 21 psi higher pressure than did bias tires.

Wong (18) indicated that for a radial tires, there is a relatively uniform contact pressure over the whole contact area. In contrast, the contact pressure for a bias tire varies greatly from point to point as

Table 6.4 Mean Value of the Tire Pressure Difference between Recommended and Measured Tire Pressure*.

	I**		II		III	
	R#	B	R	B	R	B
Mean (%)	0.3	2.5	-0.2	10.0	1.3	2.2
Standard Deviation (%)	10.7	14.6	8.0	9.6	12.9	19.9
Sample Number	495	44	89	11	1734	285

* ; (Measured Pressure-Recommended Pressure)/(Recommended Pressure)*100.

I**; Single Tire, Steering Axle, II ; Single Tire, Non-Steering Axle, III; Dual Tires, Non-Steering Axle.

R# ; Radial Tire, B ; Bias Tire

tread elements passing through the contact area undergo complex localized wiping motion. However, the effect of different tire construction and tread type on asphalt pavement is still not known well.

If government agencies wish to control tire pressures, it would be expedient to control the manufacturer's maximum recommended pressure rather than the inflation pressures used by truckers. This would ensure reasonable control, since the data collected in this study show that measured and recommended tire pressures are similar.

In summary, the pressure data collected indicate that the mean pressure of the whole sample (100 psi) is considerably higher than that traditionally used in pavement design (i.e., 80 psi), and the difference between the recommended maximum tire pressure by manufacturer and the measured tire pressure is very small, particularly for radial tires.

6.6.2 Pavement Analyses

In order to investigate the effect of increased axle loads and tire pressures on the asphalt concrete pavements shown in Figure 6.4, 18 kips and 22 kips single axles with dual tires, 34 kips and 42 kips tandem axles with dual tires, and tire pressures of 80 and 125 psi were used. For the evaluation parameters, the vertical compressive stress through the pavement structures, the horizontal strain in the asphalt concrete wearing and base courses, and the vertical compressive strain through the pavement structure at the point below the wheel load were used and are shown in Figures 6.6, 6.7, and 6.8.

The data from the vertical compressive stress are used for the calculation of the rut depth in the asphalt concrete surface layer. As presented in Figure 6.6, the effect of the higher tire pressures is sig-

nificant in the asphalt wearing layer for both structures and for both axles. However, at a depth of about 15 in. the vertical compressive stresses are about equal for both structures and for both pressures.

The point of change from compressive strain to tensile strain of horizontal strain in the asphalt concrete layer is shifted to the upper part as the tire pressure increases and/or axle load decreases. The magnitude of the maximum tensile strain, however, is the greatest for the heaviest axle load and highest tire pressure.

The maximum tensile strain at the bottom of the asphalt base layer of Pavement B (SN = 3.4, the base layer is stiffer than the surface layer) is less than that of Pavement A (SN = 3.0, the base layer is softer then the surface layer). Tandem axle dual tires (34 kips) result in less tensile strain than single axle dual tires (18 kips).

Figure 6.8 shows the vertical compressive strain profile through the pavement structures in Figure 6.4. For the same load, the effect of different tire pressures on the vertical compressive strain at the top of subgrade is negligible. However, the magnitude of the vertical compressive strain at the top of subgrade in Pavement B, which has softer subgrade but deeper structure depth than Pavement A, is greater than that in Pavement A.

In summary, the effect of increased tire pressures is significant in the asphalt surface layer. The magnitude of the maximum tensile strain increases with the increased tire pressure. Greater compressive strain occurs in the softer subgrade regardless of the structural number (SN) and the structural depth.

6.6.3 Equivalency Factors

One method of assessing the destructive effect of increased tire pressures is through the use of the concept of load equivalency factors.

As mentioned earlier, in most previous studies (3,9,10,11,12), only the effect of the axle loads was investigated. In this study, tire pressure variation was considered as well as axle loads.

As a standard tire pressure and axle load, 80 psi and 18 kips for single axle dual tires was used. The equivalency factors with reference to this tire pressure and axle load can be computed easily from the maximum tensile strain or maximum vertical compressive strain, from the results of ELSYM5. The greater equivalency factor between that from the maximum tensile strain and that from the maximum compressive strain was chosen. Table 6.3 presents the equivalency factor for the two asphalt pavements shown in Figure 6.4.

For the single axle, the change of equivalency factor from 80 psi to 100 psi is relatively small in the range of axle load up to 26 kips. However, the change becomes bigger as the pressure increases to 125 psi or 150 psi (which corresponds to the maximum observed in this study). A similar trend occurs for the tandem axle in the range of axle load from 34 kips to 50 kips.

The results indicated that a 25% increase in tire pressure (80 psi to 100 psi) could result in a 40 to 60% increase in equivalency for dual tire single axle of 18 kips and tandem axle of 34 kips.

In general, the equivalency factor for a tandem axle with dual tires is the smallest. That is, tandem axle and/or dual tire does less damage to the asphalt pavement.

A comparison of equivalency factors developed in this study and

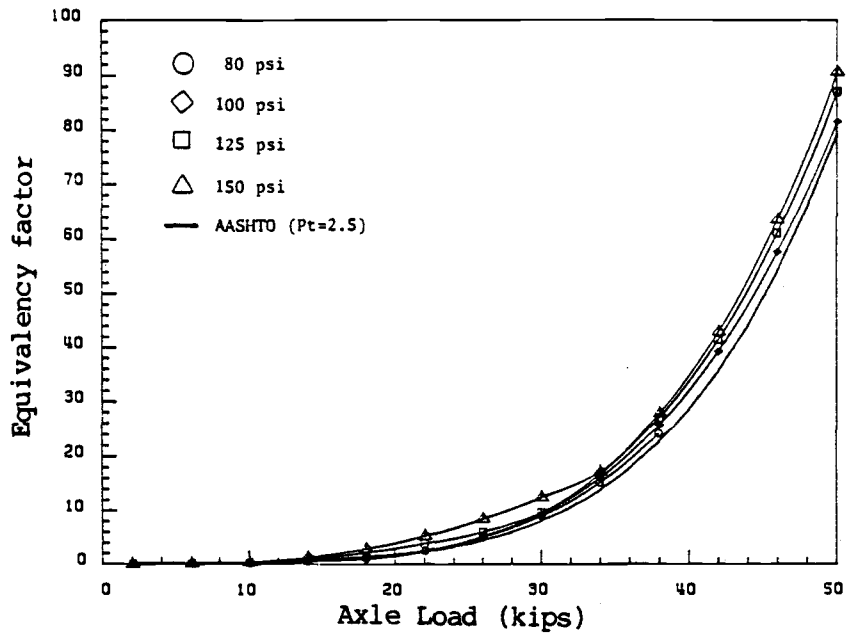
AASHTO factors (3) is illustrated in Figure 6.9. For single axle dual tires, the theoretical equivalency factors developed in this study are significantly greater than AASHTO factors for the given axle load and tire pressure ranges. For tandem axle dual tires, however, the AASHTO equivalency factors are greater for a tire pressure of 80 psi and 100 psi (above an axle load of 34 kips). The equivalencies developed in this study for tandem axle dual tires and pressure of 125 and 150 psi are much larger than AASHTO equivalencies.

It can be concluded that the effect of increased tire pressure on asphalt pavement is significant. For the asphalt pavements considered herein fatigue failure of the asphalt layer seems to be the main distress type due to the increased tire pressure as indicated in Table 6.3. This means that the equivalency factor based on the tensile strain at the bottom of the asphalt base layer is greater than that based on the compressive strain at the top of the subgrade except for the range of extremely heavy axle loads under the given pavement structures in this study. However, it should be noted that the analysis used here does not account for the increased asphalt layer deformation. This problem is addressed in the following section.

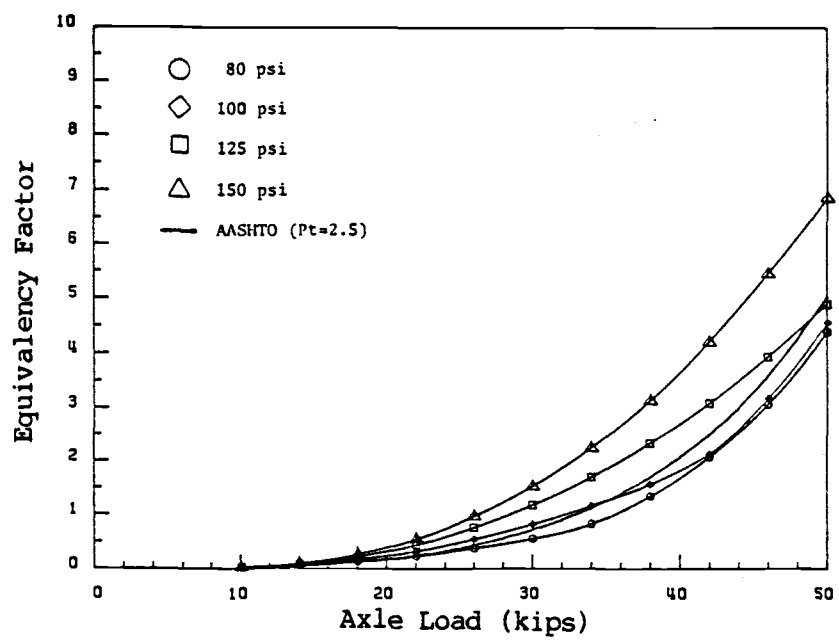
6.6.4 Rut Depths

In order to predict the rut depth in asphalt surface layer due to increased tire pressures, the Shell method (17) was employed.

The average vertical compressive stress in asphalt surface layers illustrated in Figure 6.4 is about 90 percent of the inflation tire pressure as presented in Table 6.5. As presented in Table 6.6, the rut depth in the asphalt surface layer for pavement A in Figure 6.4



(a) Single Axle Dual Tires - Pavement A in Figure 6.4



(b) Tandem Axle Dual Tires - Pavement A in Figure 6.4

Figure 6.9 Comparison of Equivalency Factor from ELSYM5 and AASHTO.

increases by 52 percent as the tire inflation pressure increases by 56 percent. Therefore, it can be said that the increase in rut depth for asphalt layers is approximately proportional to the increase in tire inflation pressure.

6.7 CONCLUSIONS AND RECOMMENDATIONS

6.7.1 Conclusions

The existing operating characteristics of Oregon's trucks, including levels of tire pressures, were surveyed. In order to investigate the effect of increased truck tire pressure on asphalt pavements ELSYM5 was run. The equivalency factors taking into account tire pressures (80, 100, 125, 150 psi) with typical asphalt pavements in Oregon were developed.

The major findings and conclusions of this study are:

1. As expected the use of radial tires is dominant. Eighty-seven percent of the tires surveyed are of radial construction. The bias tires used may be replaced with radial tires in future.
2. The average measured pressures of radial and bias tires are 102 and 82 psi, respectively. Adequate consideration of current levels of tire pressure should be reflected in paving mix design, pavement structure design methods including overlay design, and maintenance schedules.
3. Since the difference between the recommended maximum tire pressure by manufacturer and the measured tire pressure is very small, it can be said that truckers tend to use the recommended maximum tire pressure (cold) due to operating safety and efficiency.

Table 6.5 Average Vertical Compressive Stresses (psi).

(a) Single Axle Dual Tires

	18 kips		22 kips	
	80 psi	125 psi	80 psi	125 psi
Pavement A in Figure 6.4	70.7	108.2	71.8	109.4
Pavement B in Figure 6.4	72.6	112.8	72.9	113.2

(b) Tandem Axle Dual Tires

	34 kips		42 kips	
	80 psi	125 psi	80 psi	125 psi
Pavement A in Figure 6.4	70.4	107.6	71.2	108.8
Pavement B in Figure 6.4	72.4	112.4	72.7	112.8

Table 6.6 Predicted Rut Depth under Given Conditions.

Tire Pressure(psi)	Rut Depth (in.)
80	0.022
125	0.034

Conditions: AR 4000 (P.I.= -1.4),
Pavement A (SN = 3.0) in Figure 6.4,
H_o = 2.0 in.,
Number of Repetitions = 10^6,
MAAT = 20°C.

4. The effect of the increased tire pressure is significant in the asphalt surface layer in terms of vertical compressive stresses. And, the magnitude of the maximum tensile strain at the bottom of asphalt base layer increases with the increased tire pressures.
5. Fatigue failure of asphalt layer considered in this study is the main distress type on asphalt pavement as tire pressure increases.
6. Theoretical equivalency factors relative to an 18 kip single axle dual tires and an 80 psi tire pressure, were developed to take into account increased tire pressures and number of tires per axle. These indicate that a 25% increase in tire pressure (from 80 psi to 100 psi) could result in a 40 to 60% increase in equivalency for a single axle of 18 kips with dual tires and a tandem axle of 34 kips with dual tires.
7. Using the Shell method, the rut depth of the asphalt concrete layer increases as the tire pressure increases. The percent increase in deformation of asphalt layer is approximately the same as percent increase in truck tire inflation pressure.

6.7.2 Recommendations

In order to control the effect of increased tire pressure on asphalt concrete pavement, the following recommendations are made:

1. In order to reduce the damage from increased tire pressure and axle load, the use of tandem axle dual tires rather than single axle single tires is recommended. This is in keeping with the concept of spreading the load as much as possible.

2. Use a stiff asphalt surface layer designed to minimize deformation to reduce the fatigue failure and rutting. This should only be considered if the supporting layers are also stiff.
3. More comprehensive study on the tire-asphalt pavement system is needed. Particularly, the effect of bias tires versus radial tires is poorly defined, and the effect of different tire tread types.

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DISCLAIMER

7.0 CONCLUSIONS

7.1 SUMMARY

Three major factors affecting the performance of asphalt mixtures were investigated. Three major factors are mixing moisture, oxidative aging, and higher tire pressures and axle loads.

The repeated load diametral test was used for the evaluation of the effects of mixing moisture, additives (lime and Pavebond Special) and oxidative aging on asphalt mixtures. A modified Pressure Oxidation Bomb (POB) laboratory accelerated aging method with pure oxygen for both asphalt cements and asphalt mixtures and the Fraass brittle test for asphalt cements were used.

The existing operating characteristics of Oregon's trucks, including levels of tire pressures, were surveyed and analyzed.

Mix design criteria with aggregate gradation used by Oregon State Highway Division were investigated to minimize damage from higher tire pressures. A simple method of creep test to predict deformation of an asphalt mixture which used a loading device for soil consolidation, an LVDT and a microcomputerized data acquisition system was developed.

In order to investigate the effect of higher truck tire pressure on asphalt pavements, ELSYM5 was run. The theoretical equivalency factors were developed taking into account tire pressures with typical asphalt pavements in Oregon.

7.2 CONCLUSIONS

The general conclusions are drawn from the findings of three independent researches presented herein:

1. The performance of asphaltic mixtures are best explained by

resilient modulus. In general, mixtures with higher modulus shows better performance, that is, longer fatigue life and less deformation. Also, modulus ratio (after conditioning/ before conditioning) is a good indicator to identify moisture and/or aging susceptible mixtures.

2. The level of compaction or air voids of mixture is the contolling factor for the better performance of asphalt mixtures.
3. Lime shows a substantial benefit to modulus and resistance to deformation (stability), in particular with the existence of moisture.
4. Current empirical mix design may not be adequate to produce mixtures having desirable durability and stability.

7.3 RECOMMENDATIONS

Recommendations drawn from these studies are;

1. It is necessary to improve the procedures or test methods for selecting optimum asphalt content in order to achieve desirable stiffness and durability of mixtures. Mechanistic mix design approach using the diametral test performed herein is recommended. The diametral test can evaluate the fundamental properties of asphalt mixtures in terms of modulus, fatigue, permanent deformation at different levels of mix variables.
2. More comprehensive study on the interactive effects of environmental conditions, traffic characteristics and pavement structure is needed. Simple creep tests or diametral tests combining oxygen conditioning with a Lottman conditioning are recommended.

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APPENDIX

EFFECT OF MIX CONDITIONING ON PROPERTIES OF ASPHALT MIXTURES

by

Ok-Kee Kim
Chris A. Bell
and
R.G. Hicks

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ABSTRACT

The serviceability of asphalt pavements is controlled by many factors, such as expected load, mixture, and environmental variables. In order to provide good serviceability, an asphalt mixtures must have several characteristics--stiffness, tensile strength, resistance to fatigue, permanent deformation, and resistance to water damage. Recently, water-induced damage of asphalt mixtures has caused serious distress, reduced performance, and increased maintenance for pavements in Oregon. The information from tests performed at Oregon State University concerning three projects built during 1978-1980 was used to determine relationships between asphalt concrete pavement performance as indicated by resilient modulus, indirect tensile strength, fatigue life, and mix level of compaction for both as-compacted and conditioned samples. It was found that the rate of water-induced damage of asphalt mixtures was strongly related to aggregate quality and air void content of the mixture--the higher the air void content and the poorer the aggregate, the larger loss of strength.

NOTATION

The following symbols are used in this paper:

M_R - resilient modulus, psi

ΔH - horizontal elastic tensile deformation, inches

P - dynamic load, pounds

h - sample thickness, inches

ν - Poisson's ratio

N_f - number of load repetitions to failure

K, m - regression constants

ϵ_t - initial elastic tensile strain

RCL_{mod} - retained resilient modulus ratio at same compaction level

$$= \frac{\text{Modulus of conditioned sample}}{\text{Modulus of as-compacted sample}}$$

$R100_{mod}$ - retained resilient modulus ratio compared to 100% of as-compacted samples

$$= \frac{\text{Modulus of conditioned sample}}{\text{Modulus of as-compacted sample at 100\% compaction}}$$

RCL_{ts} - retained indirect tensile strength ratio at same compaction level

$$= \frac{\text{Tensile strength of conditioned sample}}{\text{Tensile strength of as-compacted sample}}$$

$R100_{ts}$ - retained indirect tensile strength ratio compared to 100% of as-compacted samples

$$= \frac{\text{Tensile strength of conditioned sample}}{\text{Tensile strength of as-compacted sample at 100\% compaction}}$$

CEF - conditioning effectiveness factor at 90% compaction and 50 microstrain

$$= \frac{RCL_{mod}}{\dfrac{N_f,\ \text{conditioned}}{N_f,\ \text{as-compacted}}}$$

r^2 - coefficient of determination

INTRODUCTION

The performance of asphalt pavement materials is affected by many factors including type of mixtures, degree of compaction, stress level, rate of loading, and environmental factors. Currently, the effects of the environment under which pavements serve, including both climatic and loading, are receiving increased notice. For example, several studies have recently been performed to investigate the effect of water on asphalt pavement performance including effect of additives to reduce the moisture damage [1,2,3,4,5]. The loss of adhesion between the asphalt cement and aggregate surface, which affects the asphalt mixture properties, is primarily due to the action of water. Decreases in strength and modulus due to moisture reduce the performance of the asphalt mixtures and, consequently, should be considered in pavement and mixture design practice.

The purpose of this paper is to: 1) summarize the test results for three recent projects in Oregon; 2) obtain a better understanding of the causes of the pavement problems with respect to moisture; and 3) develop relationships between mixture performance (resilient modulus, fatigue life, and indirect tensile strength) and the different mix variables for as-compacted and conditioned samples.

PROJECTS EVALUATED

The projects studied were North Oakland-Sutherlin (NO-S), Castle Rock-Cedar Creek (CR-CC), and Warren-Scappoose (W-S). These three projects were constructed between 1978 and 1980 in Oregon, and each project utilized an Oregon Class B mix (Table 1). The construction reports of top lift and base lift of the pavement indicated that

several mix variables were ranging within a very wide band, indicating quality control problems during mixing (asphalt content, gradation) and during compaction (air void content) [6,7,8]. The pavement cross sections are illustrated in Figure 1.

North Oakland-Sutherlin Project (NO-S)

This project is a section of Interstate 5 located approximately 12 miles (19 km) north of Roseburg. Its overall length is 3.21 miles (5.14 km). The recommended asphalt content was 6.9% of an AR 8000 asphalt cement treated with 0.85% "pavebond" (an antistrip agent). The asphalt concrete base on this project was paved in October through December 1978, and showed problems of ravelling and potholing shortly thereafter. An investigation performed by the Oregon Department of Transportation (ODOT) showed that the reduced quality of the paving was basically the result of using varying amounts of unsound and nondurable aggregate in the mix. The aggregate used in this project was a submarine basalt containing seams of sulfate compounds of calcium, sodium and magnesium. Soundness test results for produced aggregate used in the paving ranged from 4.16% to 38.94% loss for coarse aggregate and 11.56% to 48.23% loss for fine aggregate [6].

Castle Rock-Cedar Creek Project (CR-CC)

This project, built in 1979, is a section of the Hebo-Valley Junction Highway, located in Tillamook and Yamhill counties. The overall length is 11.7 miles (18.7 km). Asphalt contents of 6.1% for wearing surface and 6.7% for the base course were recommended. The average for the as-constructed thickness is 2.0 inches (5.1 cm) for the

base and 1.7 inches (4.3 cm) for the wearing surface. The asphalt grade recommended was an AR 4000. Progressive pavement ravelling and potholing was noticed during the months following construction of this project. In this case the ODOT investigation showed that the reduction in pavement life results from excess variability in aggregate gradation, inadequate asphalt coating of aggregate, and high air void content [7].

Warren-Scappoose Project (W-S)

This project is a section of the Columbia River Highway, located in Columbia county. The overall length is 5.05 miles (8.13 km). The base course was constructed in 1979 and the wearing surface in 1980. The recommended asphalt content was 5.1% for the wearing surface and 5.7% for the base course. The asphalt grade recommended was an AR 4000. Progressive pavement ravelling and potholing were noticed in the base course during the months following construction. The core data obtained for this project showed that the reduction in pavement life resulted from high air void content and variability in aggregate gradation [8].

SAMPLE PREPARATION AND TEST METHODS

Laboratory samples were prepared at Oregon State University in order to determine the resilient modulus, tensile strength, fatigue life, and permanent deformation of the asphalt mixtures. For each project the percent passing the No. 200 sieve material was 6% and the asphalt content were as follows:

North Oakland-Sutherlin: 6.0%,

Castle Rock-Cedar Creek: 6.0%, and

Warren-Scappoose: 5.5%.

For each project, samples were prepared at the range of compaction levels shown in Table 2. All tests were run on standard laboratory samples using the repeated load indirect tensile test.

Sample Preparation

Following the standard ODOT procedure [9], 4-inch (100 mm) diameter by 2.5-inch (63 mm) high specimens were fabricated for each project using the same materials (asphalt and aggregate) employed during construction. Sixteen samples were prepared for each mix conditions. Eight samples were tested as compacted and eight samples tested after moisture and freeze-thaw conditioning (Figure 2). All samples were tested in the diametral mode for elastic modulus, fatigue life and permanent deformation. MTS equipment was used for measurement of indirect tensile strength. In this paper, the results of permanent deformation tests are not included.

Test Method

Dynamic diametral tests were run to obtain the data of modulus, fatigue life and permanent deformation. The dynamic load duration was fixed at 0.1 seconds and the load frequency at 60 cycles per minute. All tests were carried out at room temperature (22 ±2°C, 71.6 ±3.6°F). The Lottman conditioning procedure [3] was used to evaluate the influence of moisture and freeze-thaw. The main steps of this conditioning procedure are:

1. Determine the resilient modulus of the as-compacted samples,

2. Vacuum saturate (26 in., 66 cm Hg) the samples for two hours,
3. Place the saturated samples in a freezer at -18°C (0°F) for 15 hours.
4. Place the frozen, saturated specimen in a warm water (60 ± 2°C, 140 ± 3.6°F) bath for 24 hours,
5. Place the specimen in a water bath at room temperature (22.8 ± 1°C, 73 ± 1.8°F) for three hours, and
6. Return the modulus test along the same sample axis as the as-compacted modulus was measured (step 1), (22.8 ± 1°C, 73 ± 1.8°F).

RESULTS

All tests were run at horizontal tensile strains ranging between 50 and 150 microstrain. The bulk specific gravity and air void content corresponding to each level of compaction for the three projects are shown in Table 3.

Resilient Modulus

The following equation was used to determine the modulus [10]:

$$M_R = P \times (0.2692 + 0.9974\nu)/(\Delta H \times h) \quad (1)$$

Poisson's ratio was assumed constant and equal to 0.35, which simplifies equation (1) to:

$$M_R = P \times 0.6183/(\Delta H \times h) \quad (2)$$

Moduli values of as-compacted and conditioned samples from all projects are presented in Table 4. In order to determine the effect of conditioning on mixture performance, two parameters, RCL and $R100_{mod}$, were computed for each level of compaction, and are also presented in Table 4. These are defined as follows: RCL_{mod} is the ratio of retained stiffness at the same compaction level and $R100_{mod}$ is the ratio of retained stiffness compared to the modulus at 100% compaction of as-compacted. The moduli of as-compacted and conditioned and the values of RCL_{mod} for the North Oakland-Sutherlin project were the lowest, while those for the Warren-Scappoose project for each compaction level are the highest. The Warren-Scappoose project also exhibited higher bulk specific gravities or lower air void content than the others (Table 3) and had lower asphalt content. Figure 3 shows the variation of resilient modulus with air void content. As indicated for both as-compacted (Figure 3a) and conditioned samples (Figure 3b), the diametral modulus has a very strong linear relationship to the air void content. The coefficients of determination of each project are around 1.0. In general, as the air void content decreased from 10% to 4%, the moduli increased about twofold for both as-compacted and conditioned samples.

Values for $R100_{mod}$ increases as the air void content decreases (Figure 4). As would be expected, $R100_{mod}$ has a very strong linear relationship with air void content. Values for RCL_{mod} are given in Figure 5. As indicated, most of the RCL_{mod} for the North Oakland-Sutherlin project are below 70%, while those for the others are above 70%. The RCL_{mod} below 70% for the North Oakland-Sutherlin project is a result of using poor quality aggregate. Hence, the parameter RCL_{mod}

demonstrates clearly that with poor quality aggregate there is a very rapid loss of performance if good compaction is not maintained. The parameter $R100_{mod}$ shows the importance of mixture compaction more strongly than RCL_{mod}, but does not indicate the influence of aggregate quality. In summary, the effect of moisture conditioning on the stiffness of three projects is very significantly affected by the quality of aggregate used and has linear relationship to air void content.

Indirect Tensile Strength

Values for indirect tensile strength of as-compacted and conditioned samples of each project are presented in Table 5 together with their ratios of retained indirect tensile strength, i.e., $R100_{ts}$ and RCL_{ts}. For the North Oakland-Sutherlin project, the indirect tensile strength of as-compacted and conditioned samples are generally lower than the other two projects. Also RCL_{ts} and $R100_{ts}$ for the North Oakland-Sutherlin project are lower at all compaction levels. The Warren-Scappoose project again exhibits the highest strength and $R100_{ts}$ value, particularly at high levels of compaction. Figure 6 shows that indirect tensile strength of both as-compacted and conditioned samples have strong linear relationships with air void content. Like $R100_{mod}$, there is very strong linear relationship between $R100_{ts}$ and the air void content (Figure 7). In general, as the air void content decreases from 12 to 4%, the indirect tensile strength increases about twofold in both as-compacted and conditioned samples. Figure 8 shows RCL_{ts} of each project at all compaction levels tested. It is again noticed that values of RCL_{ts} for the North Oakland-Sutherlin project remain below

70%, while those of the other projects rise above 100%; that is, the retained indirect tensile strengths of conditioned samples are greater than those of as-compacted samples in the test of the Castle Rock-Cedar Creek and Warren-Scappoose project for each compaction level.

Figure 9 indicates that indirect tensile strengths of as-compacted and conditioned samples also have fairly linear relationships with resilient modulus at the corresponding air void content. In summary, the results of the indirect tensile strength of each project are similar to those of resilient modulus; that is, the effect of aggregate quality on RCL_{ts} is similar to RCL_{mod}. For poor quality aggregate, RCL_{ts} is consistently below 70% while for good quality aggregate, RCL_{ts} is above 70%.

Fatigue Life

Fatigue life is characterized by the number of load applications required to cause failure of the sample. Attempts to relate the number of load applications to the state of stress or strain have shown that the best correlation exists between the tensile strain and the number of load applications, as follows:

$$N_f = K\ (1/\epsilon_t)^m \tag{3}$$

The fatigue life of a specific mix is, therefore, defined by the constant K and m. Both K and m are affected by the mix variables. The horizontal tensile strain for the diametral test specimen is calculated from the following equation [10]:

$$\epsilon_t = H \times (0.03896 + 0.1185\nu)/(0.0673 + 0.2494\nu) \quad (4)$$

Assuming that the Poisson's ratio is constant and equal to 0.35, equation (4) becomes:

$$\epsilon_t = \Delta H \times 0.5203 \quad (5)$$

Horizontal tensile strain versus number of load repetitions to failure of each project are shown in Figures 10, 11, and 12, together with the level of compaction. The results for both as-compacted and conditioned specimens show a substantial decrease in fatigue life when the level of compaction drops. For the North Oakland-Sutherlin project, the fatigue relationship is affected significantly at a low level of compaction and not affected greatly at a high level of compaction. This may be due in part to the low quality aggregate employed on this project. For the Castle Rock-Cedar Creek and Warren-Scappoose projects good quality aggregates were employed. The fatigue life after conditioning generally increased compared with that of the as-compacted. This is due in part to the fact that the load applied for conditioned samples in order to maintain the initial strain was lower than that for the as-compacted samples. The samples from the Warren-Scappoose project had the highest moduli and generally the shortest fatigue life for both compaction levels. Although the North Oakland-Sutherlin project gave lowest moduli at a compaction level of 96%, the fatigue life was shorter than that for the Castle Rock-Cedar Creek project. In addition, the fatigue life for the North Oakland-Sutherlin project at 50 microstrain and compaction level of 96% decreased slightly after conditioning but at a low compaction level

decreased drastically at all strain levels even though the conditioned samples had lower modulus than as-compacted ones. This result is due principally to the quality of aggregate used.

The initial tensile strain, air void content, and aggregate quality are predominant factors to the fatigue life. The result indicates that the durability of asphalt pavement is dependent on the quality of aggregate, load applied and the level of compaction, or air void content.

DISCUSSION

The results of tests on mixtures from the three projects show that damage to the pavement is increased with the low values of tensile strength and resilient modulus, or with relatively large drops in strength after conditioning the sample. The data from the tests show that values of modulus and indirect tensile strength for the North Oakland-Sutherlin project are considerably lower than those for the Castle Rock-Cedar Creek and Warren-Scappoose project. The fatigue life for each project generally increased after conditioning for each compaction level and initial tensile strain. The exception was the North Oakland-Sutherlin project for which the fatigue resistance decreased especially at a low compaction level.

Fatigue life is expressed as a function of modulus, initial tensile strain, and air void content. Thus fatigue life can be used as a valuable mix characteristic to evaluate the effect of conditioning. One possible parameter is the Conditioning Effectiveness Factor (CEF). CEF represents the effects of material used and conditioning in reducing the modulus and prolong the mixture life. When good quality

aggregate was used (Castle Rock-Cedar Creek and Warren-Scappoose projects) the modulus after conditioning decreased and the fatigue life increased at the same initial tensile strain used for measuring modulus before conditioning. When poor quality aggregate was used the fatigue life as well as the modulus of conditioning samples decreased, compared to as-compacted samples, as occurred in the North Oakland-Sutherlin project. A high value of CEF, therefore, represents poor materials and a low value represents good materials, and are less susceptible to conditioning. From the results of the test, CEF for the North Oakland-Sutherlin project is 1.61, while CEF for the Castle Rock-Cedar Creek and Warren-Scappoose project is 0.37 and 0.43, respectively (Table 6). CEF clearly shows the effect of the quality of aggregate with conditioning.

CONCLUSIONS

Performance of as-compacted and conditioned mixtures used in the construction of three Oregon State projects was evaluated using dynamic testing of laboratory-compacted samples. Mix resilient modulus, indirect tensile strength, and fatigue life of as-compacted and conditioned samples were determined for samples prepared within the following range of variables:

1. Mix level of compaction: 100%, 97%, 92% and 90%,
2. Asphalt content: 6%, and
3. Percent passing No. 200: 6%.

The following major conclusions are drawn from the findings of this study:

1. There is a very strong linear relationship between air void

content and the properties of the conditioned as well as the as-compacted samples,

2. At low air void content, as-compacted and conditioned samples of each project have high values of resilient modulus, indirect tensile strength, and fatigue life,
3. The resilient modulus, indirect tensile strength, and fatigue life of conditioned samples are affected by the quality of aggregate used and air void content,
4. The results of this study indicate the importance of obtaining good quality aggregate and a low level of air void content in mixture through a high level of compaction, and
5. The Conditioning Effectiveness Factor is used for evaluating the quality of aggregate and the effectiveness of conditioning. High CEF represents a mixture more susceptible to moisture damage.

Table 1. Aggregate Gradation for Oregon Class B Mix for Each Project.

Sieve Size	Opening (mm)	Job Mix Tolerance NO-S	CR-CC	W-S
1"	25	---	100	100
3/4"	19	95 -100	95 -100	92 -100
1/2"	12.5	80 - 92	81 - 93	82 - 94
3/8"	9.5	---	---	73 - 85
1/4"	6.25	54 - 66	57 - 69	54 - 66
# 4	4.75	---	---	46 - 56
# 10	2.00	21 - 29	22 - 30	26 - 34
# 40	0.425	8 - 16	8 - 16	8 - 16
#200	0.075	3 - 7	3 - 7	2.6 - 6.6

Table 2. Range of Compaction Levels Considered.

Extent of Laboratory Compaction	Percent of Maximum Compaction		
	NO-S	CR-CC	W-S
2nd* Comapction	100	100	100
1st* Compaction	96	97	97
95 Blows at 100 psi and 500 psi Leveling Load	92	92	93
30 Blows at 100 psi and 300 psi Leveling Load	91	90	90

* see Reference 9.

Table 3. Bulk Specific Gravity and Air Void Content.

Degree of	Bulk Specefic Gravity			Air Void Content (%)		
Compaction (%)	NO-S	CR-CC	W-S	NO-S	CR-CC	W-S
100	2.41	2.30	2.45	3.3	5.3	1.6
97	2.31	2.23	2.38	7.3	8.2	4.4
92	2.22	2.11	2.29	10.9	13.2	8.0
90	2.19	2.08	2.20	12.0	14.4	11.6

Table 4. Resilient Modulus and Retained Resilient Modulus Ratio.

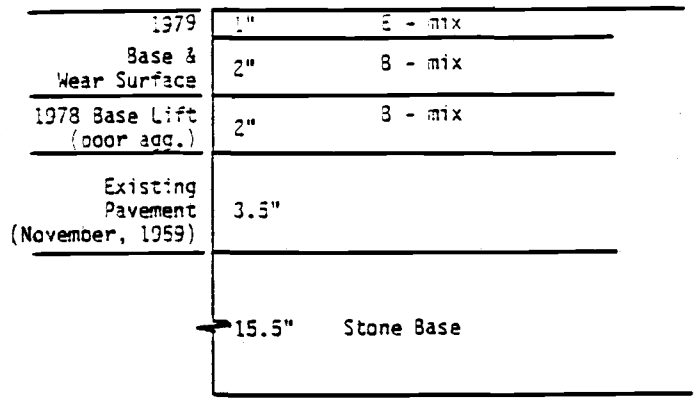
Degree of Compaction (%)	Resilient Modulus ($x10^3$,psi)						RCI_{mod}			$R100_{mod}$		
	As-Compacted			Conditioned								
	NO-S	CR-CC	W-S	NO-S	CR-CC	W-S	NO-S	CR-CC	W-S	NO-S	CR-CC	W-S
100	488	710	1082	435	638	1008	.89	.90	.93	.89	.90	.93
97	389	466	887	214	357	688	.55	.77	.78	.44	.50	.64
92	220	238	736	126	147	610	.57	.62	.83	.26	.21	.56
90	191	163	265	109	139	312	.57	.85	1.18	.22	.20	.29

Table 5. Indirect Tensile Strength and Retained Indirect Tensile Strength Ratio.

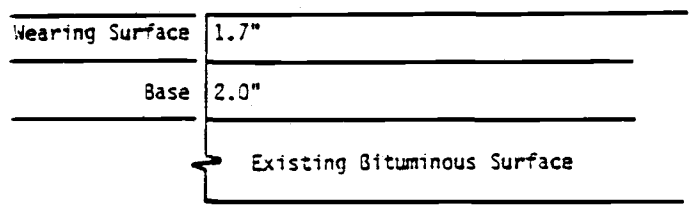
Degree of Compaction (%)	Indirect Tensile Strength (psi)						RCL_{ts}			$R100_{ts}$		
	As-Compacted			Conditioned								
	NO-S	CR-CC	W-S	NO-S	CR-CC	W-S	NO-S	CR-CC	W-S	NO-S	CR-CC	W-S
100	199	230	362	109	227	371	.55	.99	1.02	.55	.99	1.02
97	123	142	273	84	170	321	.68	1.20	1.18	.42	.74	.89
92	113	67	108	50	105	105	.44	1.57	.97	.25	.46	.29
90	95	71	65	27	84	75	.28	1.18	1.15	.14	.37	.21

Table 6. CEF at 90% Compaction and 50 Microstrain.

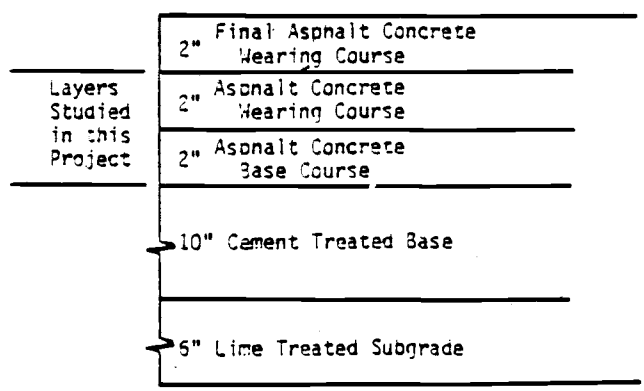
Project	RCL_{mod}	Fatigue Life As-Compacted	Fatigue Life Conditioned	CEF
NO-S	0.57	79,142	28,006	1.61
CR-CC	0.85	23,084	52,851	0.37
W-S	1.18	20,684	57,331	0.43



(a) NO-S



(b) CR-CC



(c) W-S

Figure 1. Cross Sections of Projects Studied.

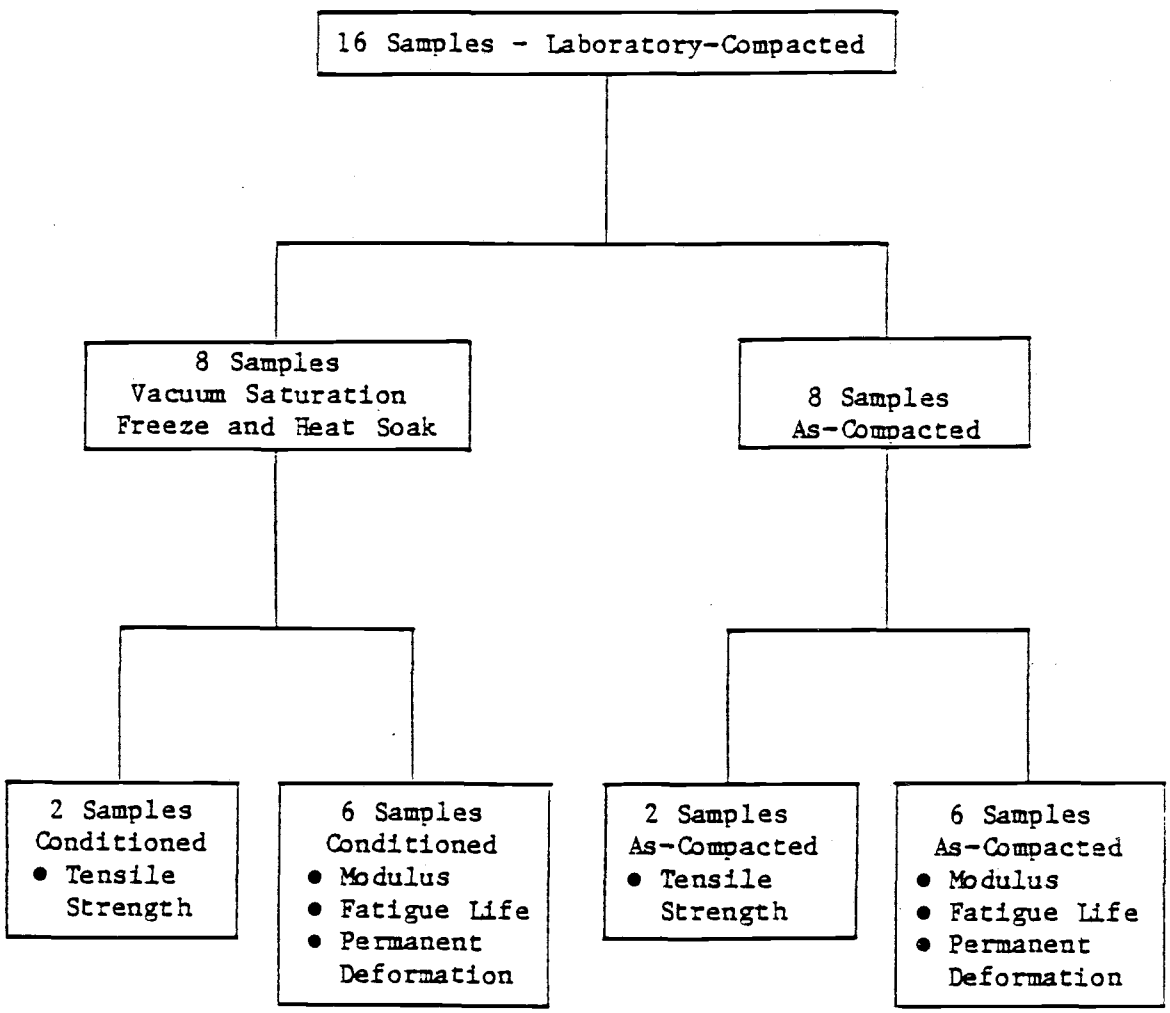
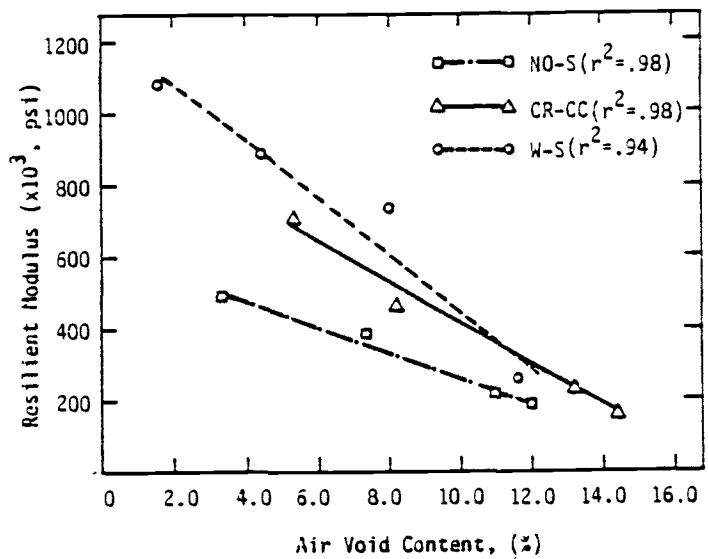
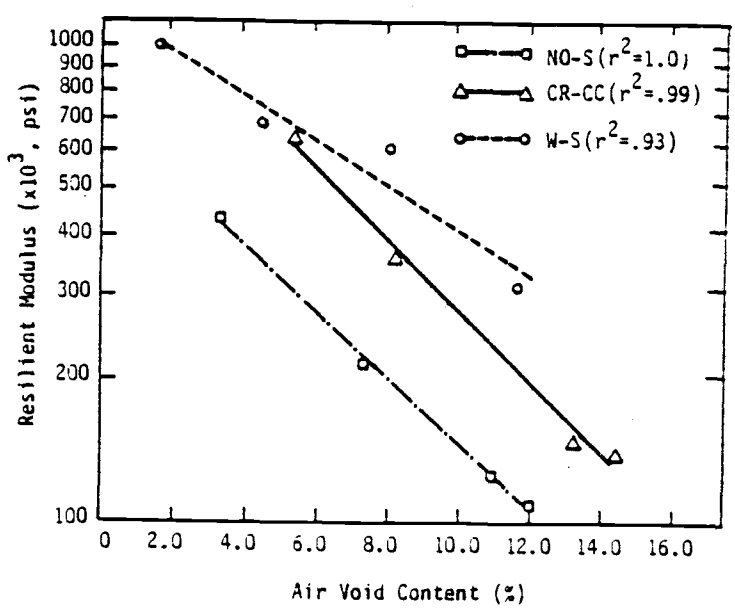


Figure 2. Test Program.



(a) As-Compacted



(b) Conditioned

Figure 3. Influence of Air Void Content on Resilient Modulus for Each Project.

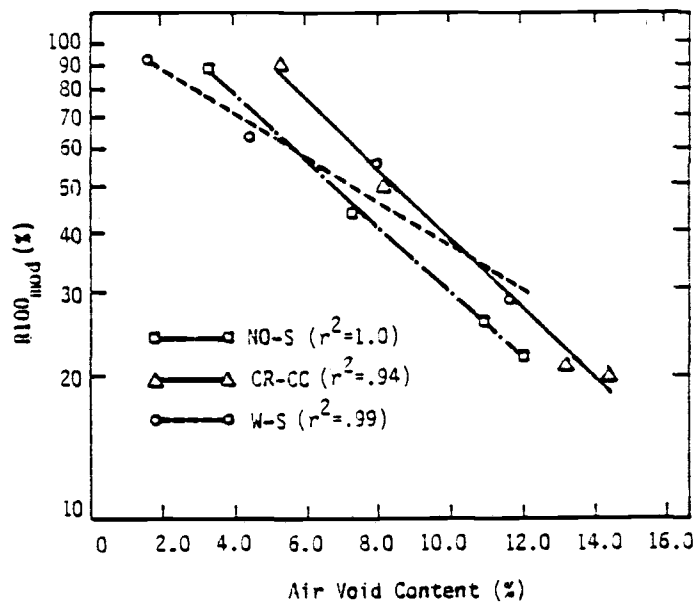


Figure 4. Influence of Air Void Content on $R100_{mod}$ for Each Project.

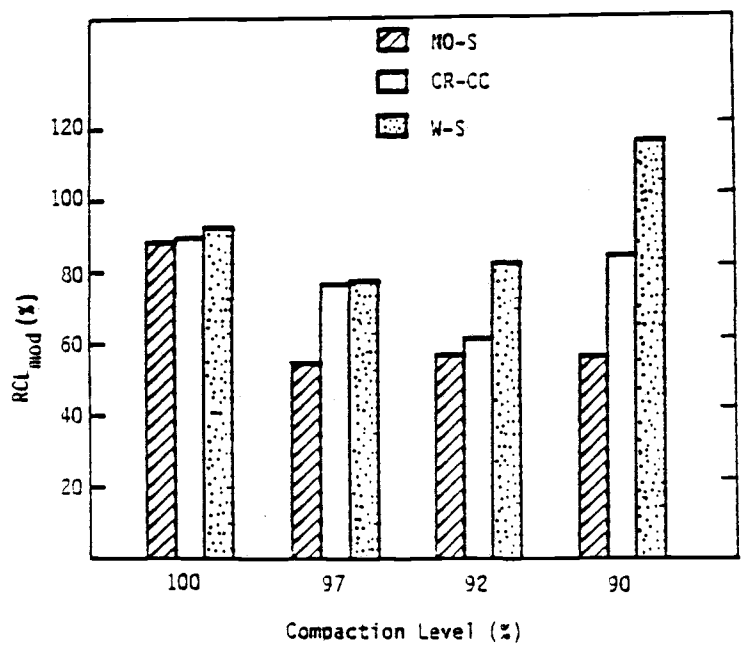
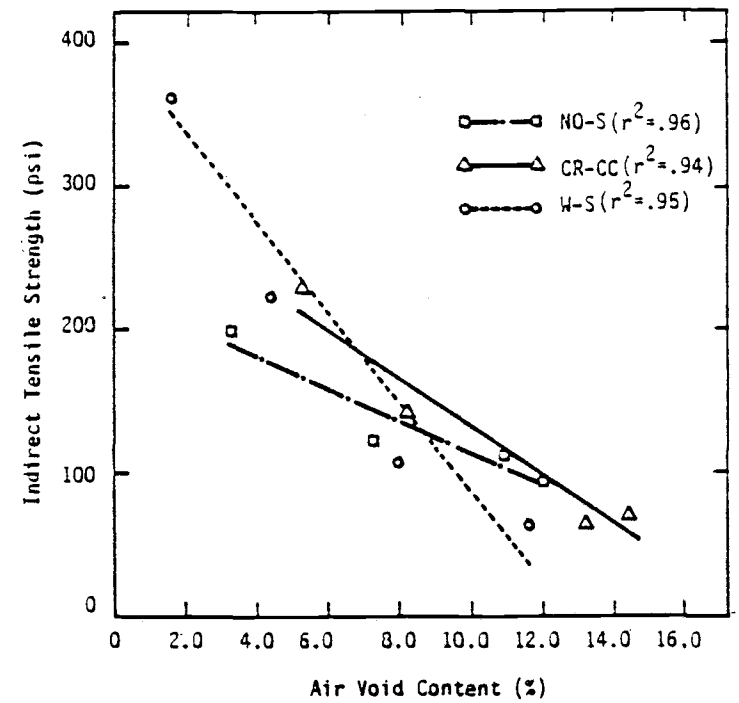
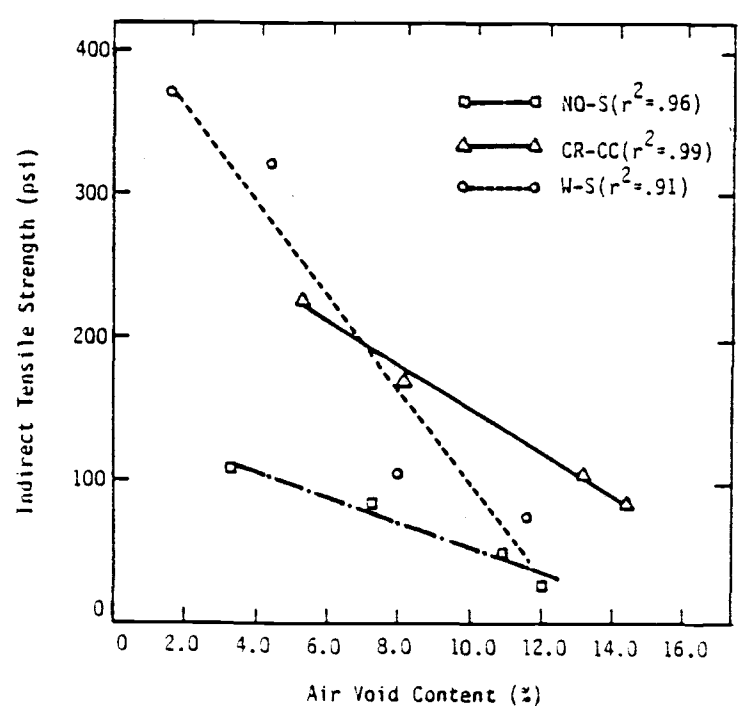


Figure 5. RCL_{mod} for Each Project at Four Compaction Levels.



(a) As-Compacted



(b) Conditioned

Figure 6. Influence of Air Void Content on Indirect Tensile Strength for Each Project.

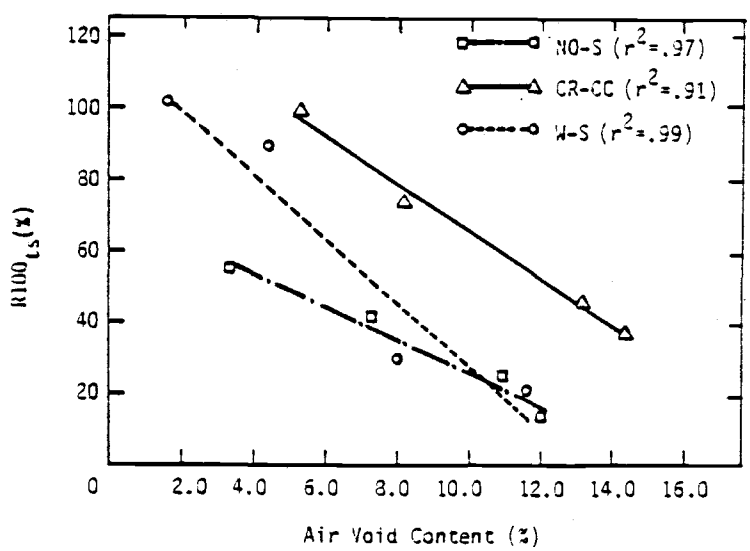


Figure 7. Influence of Air Void Content on $R100_{ts}$ for Each Project.

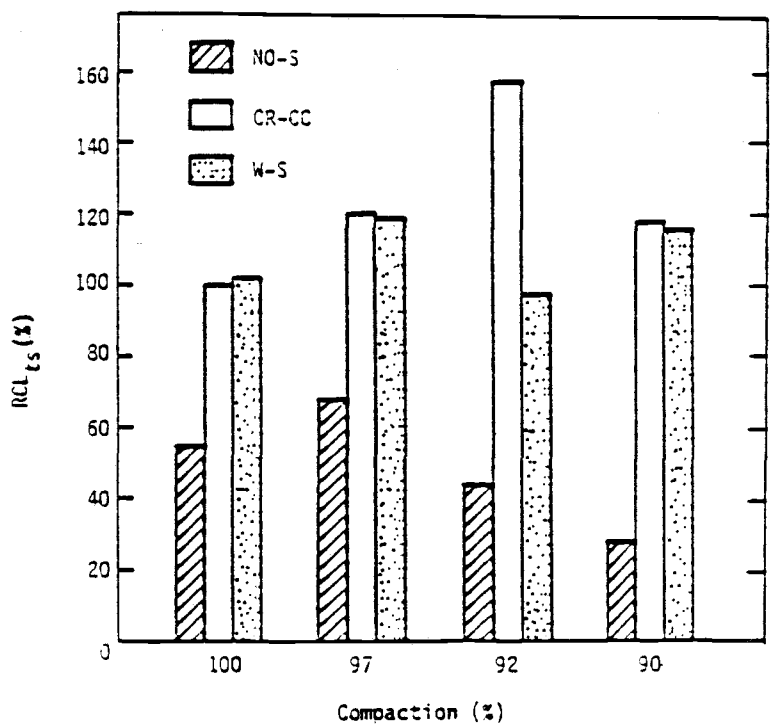
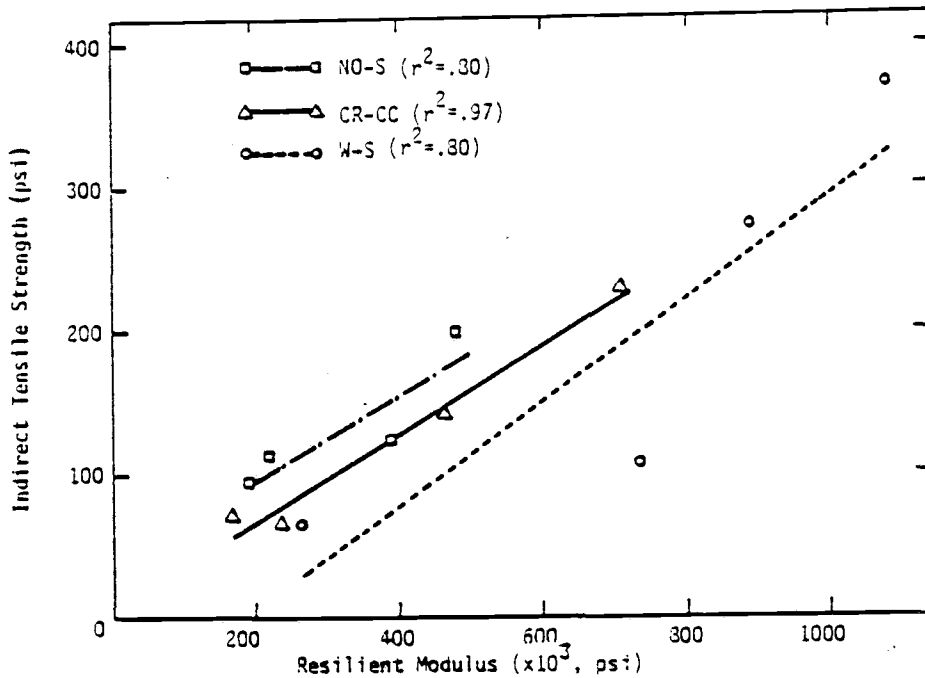
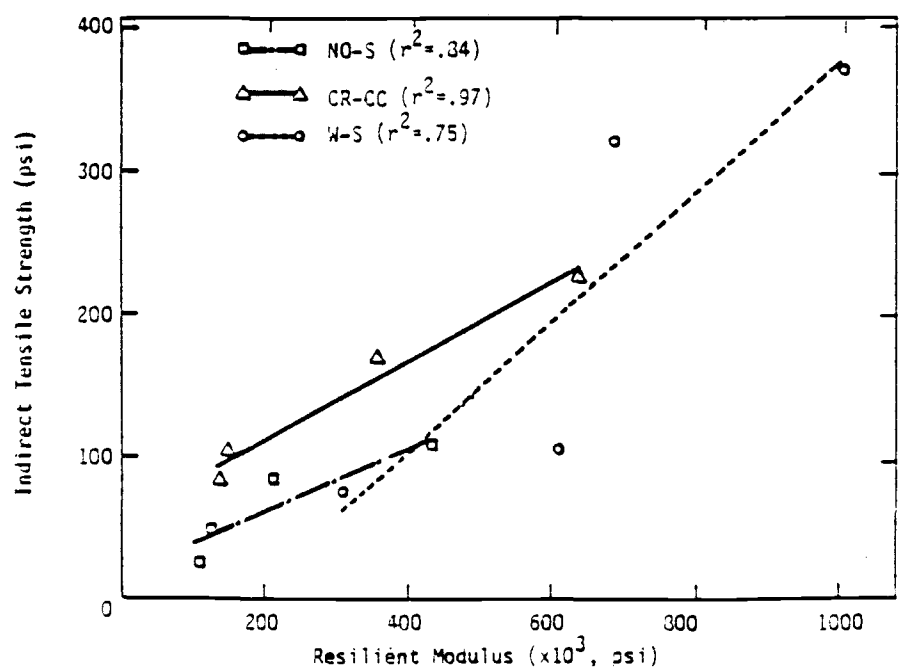


Figure 8. RCL_{ts} for Each Project at Four Compaction Levels.



(a) As-Compacted



(b) Conditioned

Figure 9. Relationship between Indirect Tensile Strength and Resilient Modulus for each Project.

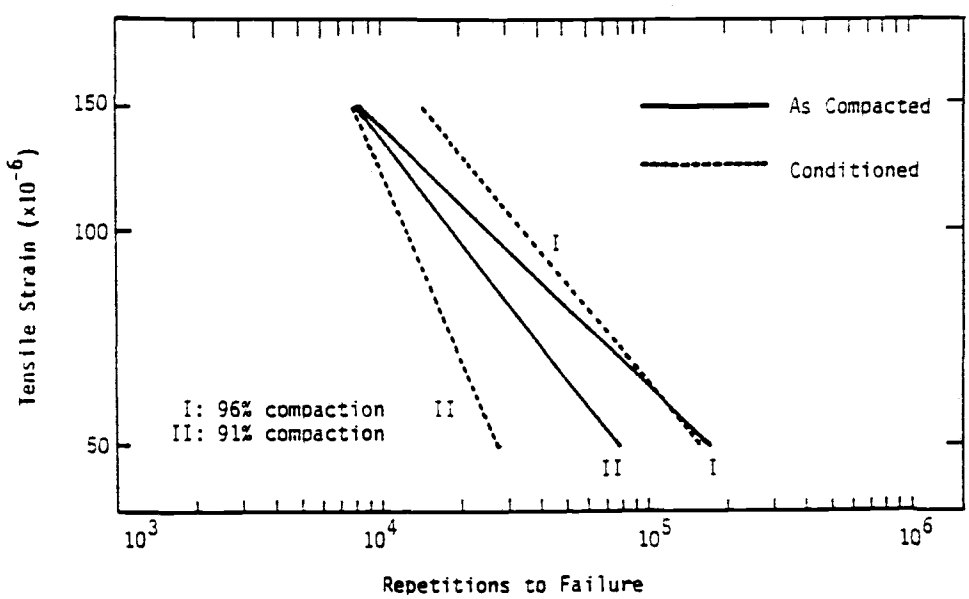


Figure 10. Horizontal Tensile Strain versus Number of Load Repetitions— North Oakland-Sutherlin.

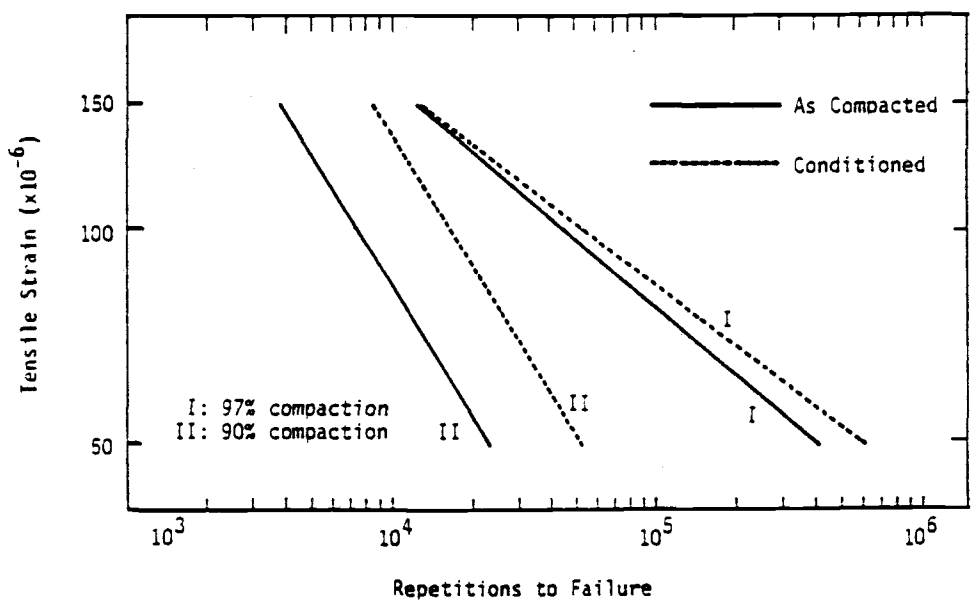


Figure 11. Horizontal Tensile Strain versus Number of Load Repetitions-- Castle Rock-Cedar Creek.

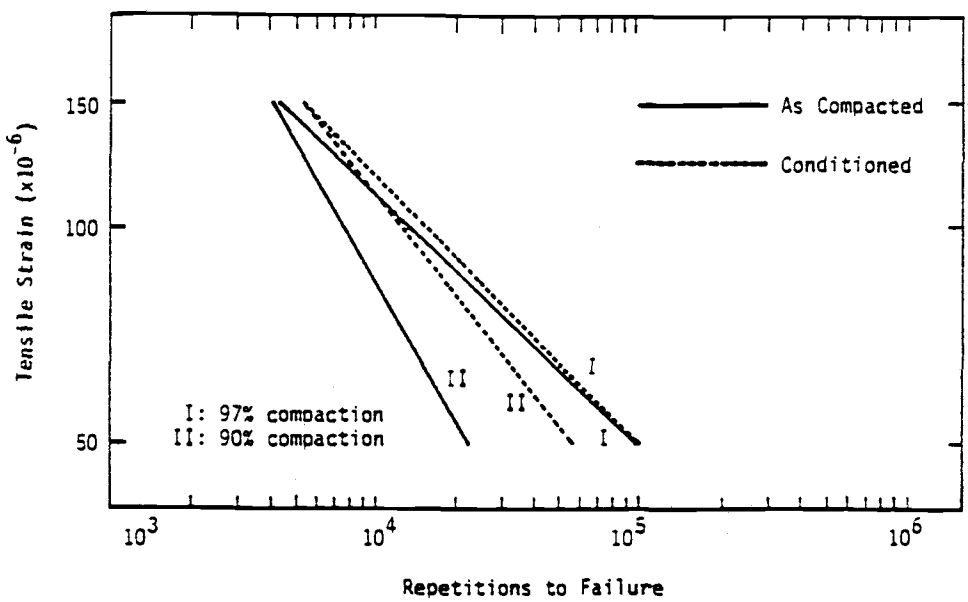


Figure 12. Horizontal Tensile Strain versus Number of Load Repetitions-- Warren-Scappoose.

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