IMPACT OF VARIATION IN MATERIAL PROPERTIES ON ASPHALT PAVEMENT LIFE:

EVALUATION OF NORTH OAKLAND-SUTHERLIN PROJECT

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Interim Report

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16. Abstract

Construction and short-term pavement performance problems were noted in the Pacific Northwest and throughout the United States during the past five years. Several reasons have been suggested to explain this sudden change in pavement performance, such as recent variabilities in asphalt properties and new developments in paving technology.

Using the data and construction materials from a recent project built in 1978-1979, Oregon Department of Transportation and Oregon State University initiated a laboratory study to determine the relationship between asphalt concrete pavement performance and mix level of compaction, asphalt content, gradation, and aggregate quality.

Conventional tests and improved dynamic tests were made on laboratory compacted samples to determine mix stiffness, fatigue life and permanent deformation characteristics. Based on fatigue and permanent deformation test results, preliminary pay adjustment factors were developed by comparing performance of mix prepared at the design optimum with the performance of mix out of specifications. It was found that performance was primarily affected by the mix level of compaction. Both the fatigue and permanent deformation tests indicated that an optimum asphalt content and optimum passing the No. 200 (074 mm) exist. Fatigue data also showed that antistrip agents are effective when added to the asphalt before mixing with aggregate. A summary of the most critical pay adjustment factors is developed in the conclusions and recommendations chapter.

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This report is the second of a series of reports concerned with the impact of variations in material properties on asphalt pavement life. The data developed in this report will be combined with that developed for two other projects (Castle Rock-Cedar Creek and Warren-Scappoose). All project data will be analyzed together to formulate recommendations for pay adjustment factors. These recommendations will appear in the final report. Assistance provided by Glen Boyle and staff, Oregon Department of Transportation, in the testing associated with Chapter 3 and that provided by Jose R. Montalvo and Michael Wynkoop, students of Oregon State University, in testing associated with Chapter 4 is acknowledged. The project was conducted in cooperation with the U.S. Department of Transportation Federal Highway Administration.

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1.0 INTRODUCTION

Problem Definition

Several changes have occurred in recent years in highway materials, and in asphalt paving technology. New asphalt sources have been brought on line, introducing changes in asphalt properties. New equipment has also been developed, affecting mixing (drum mixers, more efficient dust collector systems), storage (mix storage silos) and compaction (vibratory compactors). In the same period, economic constraints have resulted in increasing use of lower quality aggregate. As a result, there has been an increase in construction or short-term performance problems throughout the Pacific Northwest (1). The impact of such changes on the mix properties is, however, difficult to evaluate. Table 1 summarizes the main changes observed and their expected influence on the mix behavior.

One recent project, located on Interstate 5 between the North Oakland interchange and the Sutherlin interchange, was paved in October through December 1978, using an aggregate of borderline quality that resulted in reduced pavement serviceability. Evaluation of the reduction in pavement life resulting from changes in mix properties (e.g., aggregate quality, gradation, density, asphalt content) required a study of the mix properties under controlled conditions.

Purpose

The purpose of this report is to provide a better understanding of the causes of the pavement problems found in recent years, and to develop relationships between pavement performance and mix variables. Such information will be useful in developing pay-adjustment factors for projects not complying fully with specifications.

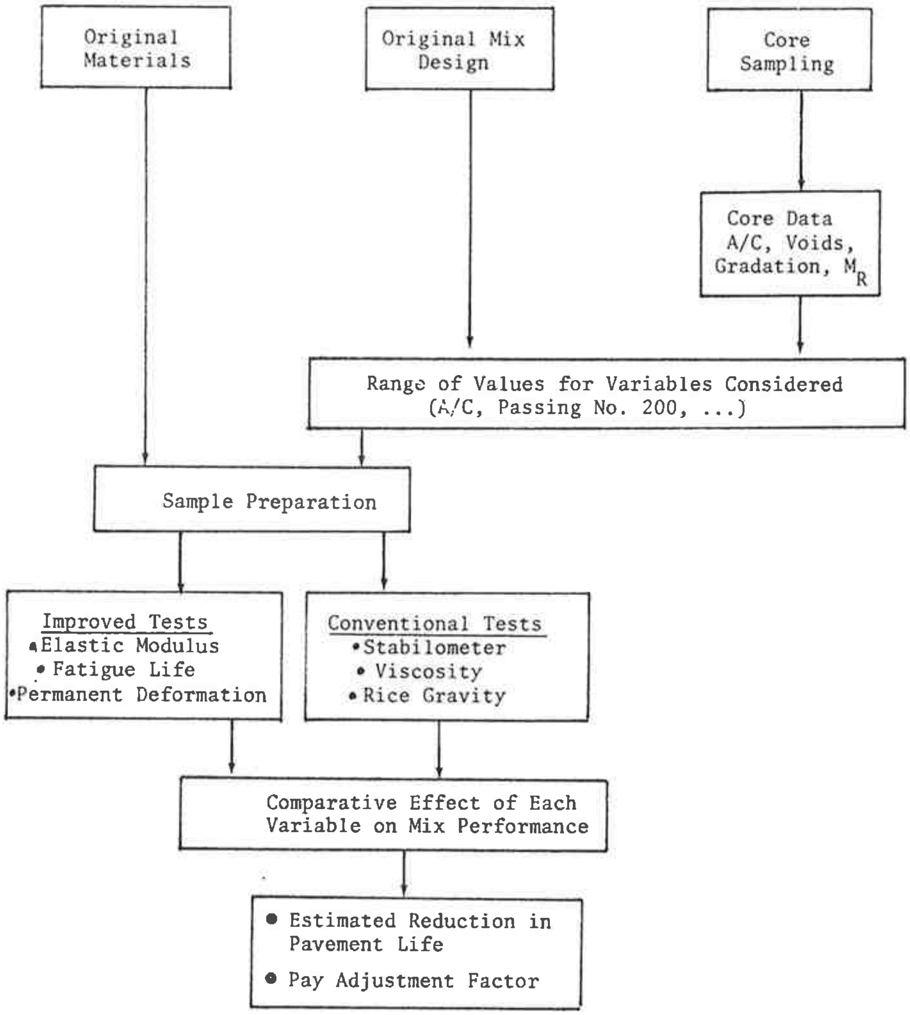
Table 2 illustrates a flow chart of the approach followed for the study of the North Oakland-Sutherlin project. Four primary mix variables were considered in the study:

(1) Mix density.
(2) Percent Passing No. 200 sieve (.074 mm).
(3) Asphalt content.
(4) Aggregate Quality.

Table 1. Recent Changes in Asphalt Paving Technology Affecting Pavement Behavior

ITEM	CHANGES OBSERVED	EXPECTED IMPACT ON PAVEMENT
ASPHALT	Wide difference between asphalt temperature-viscosity curves from various suppliers. Increased temperature - susceptibility	Compaction difficulty Slow setting mixes Reduced resistance to thermal and fatigue cracking
	Reduced compatibility between asphalt and aggregate	Increased ravelling Reduced resistance to damage from water and freeze-thaw effects
AGGREGATE	Reduced aggregate quality	Increased ravelling Reduced resistance to damage from water and freeze-thaw effects
	Single stockpile Elimination of Plant Screens	Reduced uniformity of gradation Segregation
EQUIPMENT	Use of collector dust	Reduced uniformity of gradation Flashing
	High Mix production rate	Reduced uniformity of gradation and asphalt content
	Lower mixing and laydown temperatures	Reduced uniformity of asphalt viscosity. Increased moisture. Reduced asphalt-aggregate adhesion
	Use of vibratory compactors	Breakage of aggregates Low compaction from improper use
	Drum mixers	Incomplete coating of aggregate
	Mix storage silos and Belly dump hauling equipment	Mix segregation from improper use

Table 2. Flow Chart of Study.



The range of values selected for each of the above variables was determined from project sampling and from cores taken in the spring of 1980. These are as follows:

(1) Mix Level of Compaction: 100% - 96%* - 92% - 91%
(2) Percent Passing No. 200: 2% - 6%* - 10%
(3) Asphalt Content: 5% - 6%* - 7%
(4) Percent Passing No. 10: 25%* - 30% - 35%

Following the standard ODOT procedure, 4 inches (10 cm) in diameter by 2-1/2 inches (7 cm) high samples were fabricated, for each set of conditions, using the same materials (asphalt and aggregate) as used during construction of the North Oakland-Sutherlin project.

The two types of pavement failure considered during the test program include fatigue cracking and rutting. All samples were tested in the diametral mode for elastic modulus, fatigue life and permanent deformation. To obtain complete characterization of the mixture, conventional tests were also run for Hveem stabilometer, void content, and index of retained strengths (2).

To identify the potential for stripping and ravelling, elastic modulus, fatigue life and permanent deformation tests are performed both before and after conditioning of the samples (vacuum saturation followed by a freeze-thaw cycle), using the procedure developed by Lottman (3).

Scope of Report

After a description of the North Oakland-Sutherlin project (Chapter 2), the test results are presented in Chapter 3 (ODOT research) and in Chapter 4 (OSU research). Tests performed by Oregon Department of Transportation include conventional tests. Most dynamic tests were performed at Oregon State University. Analysis of data includes the development of fatigue life and permanent deformation criteria for the as-compacted samples and the conditioned samples. Finally, pay adjustment factors are developed in Chapter 5 using the fatigue and permanent deformation models presented in Chapter 4.

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*Reference values for this project.

2.0 PROJECT DESCRIPTION

Location

The North Oakland-Sutherlin project is a section of the Interstate 5 located approximately 12 miles (19 km) north of Roseburg (see Figure 1). The precise location of the project is shown in Figure 2. The project overall length is 3.21 miles (5.14 km).

Cross Section

The pavement cross section is illustrated in Figure 3. The original pavement, constructed in 1959, was composed of a 3-1/2 inch (9 cm) asphalt concrete layer over a 15-1/2 inch (39 cm) stone base layer. The new section of the pavement was built in 1978 and 1979. The 1978 lift was Class B asphalt concrete base built with a borderline aggregate, which caused pavement distress (e.g. ravelling) soon after construction. Patching of the first life of base course was required prior to construction of the second 2 inch (5 cm) lift of asphalt concrete in 1979. A 1 inch (2.5 cm) Class E open graded friction course was placed as the wearing surface.

Mix Design

A summary of the original mix design for the 1978 asphalt concrete base lift is presented in Table 3. To achieve an Index of Retained Strength greater than 70%, 6.9% of an AR8000 asphalt cement treated with .85% "Pavebond" (an anti-strip agent) was recommended. The need for the anti-strip agent was related to the low quality of the aggregate. The aggregate soundness test (AASHTO T-104) indicated a percentage loss between 6.62 and 24.09 for the 3/4 inch - 1/4 inch (19.1 mm - 6.4 mm) fraction and between 17.7 and 45.15 for the 1/4 inch (6.4 mm) minus fraction.

The aggregate gradation used for the base layer corresponds to a Class B mix. Details of the mix formula gradation, tolerance and specification broad band are shown in Table 4. A Class E open graded mix was used for the wearing surface.

Project Data

Problems were experienced during the 1978 paving on the North Oakland-Sutherlin project from the use of paving aggregates containing unsound

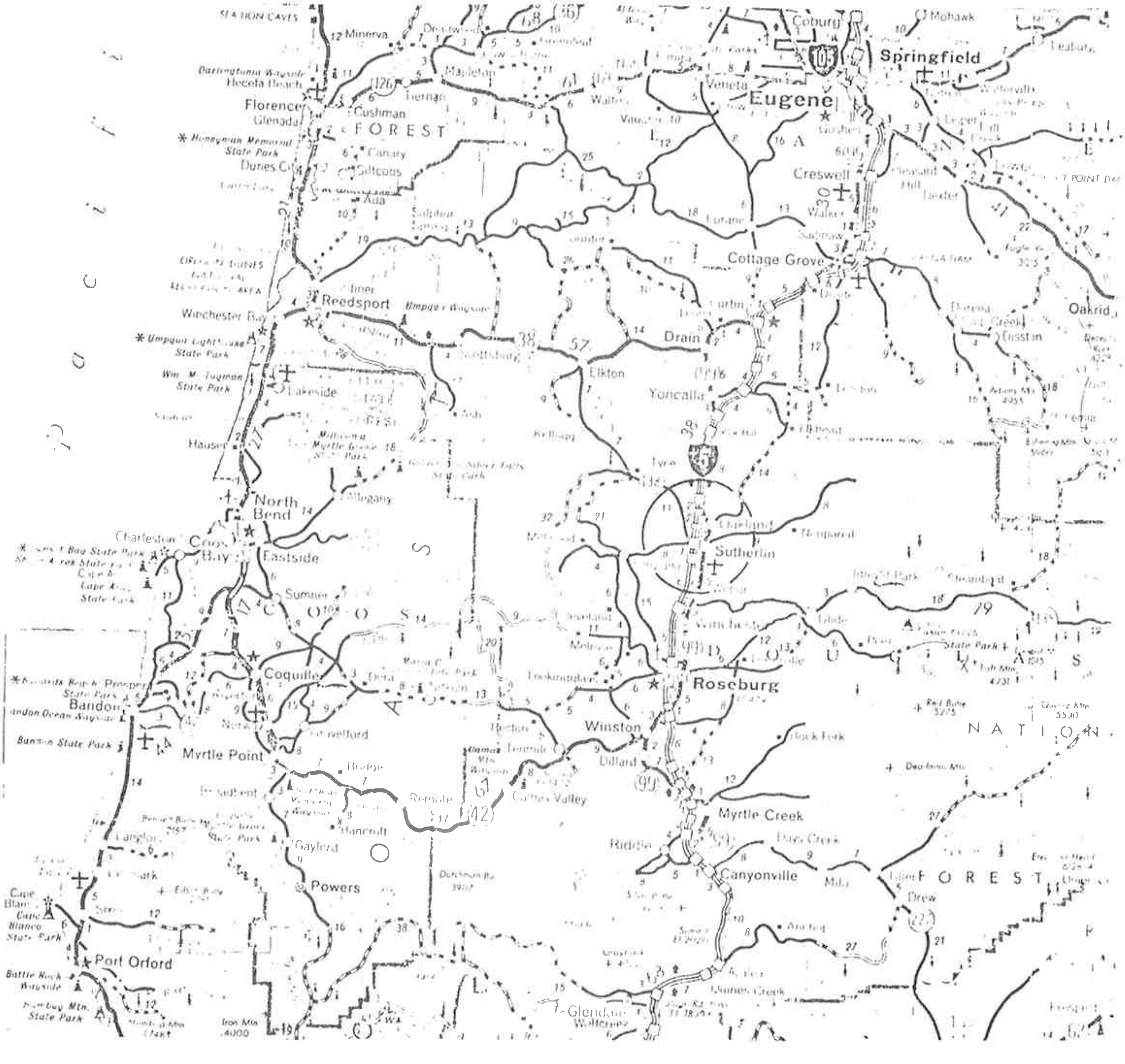

Figure 1. Map of Project Location, Southwest Part of Oregon

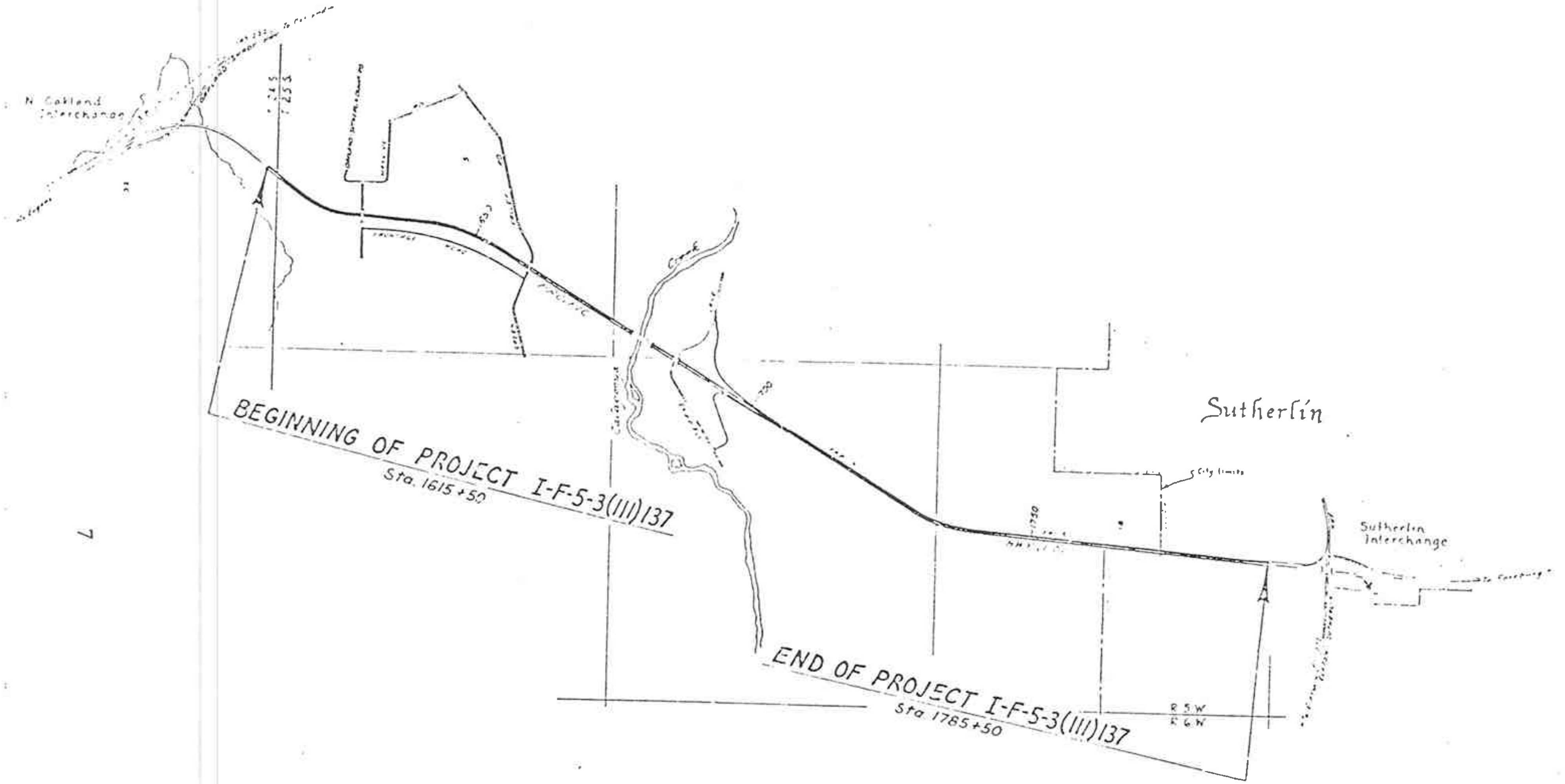

Figure 2. N. Oakland Interchange - Sutherlin Interchange, Detail of Project Location

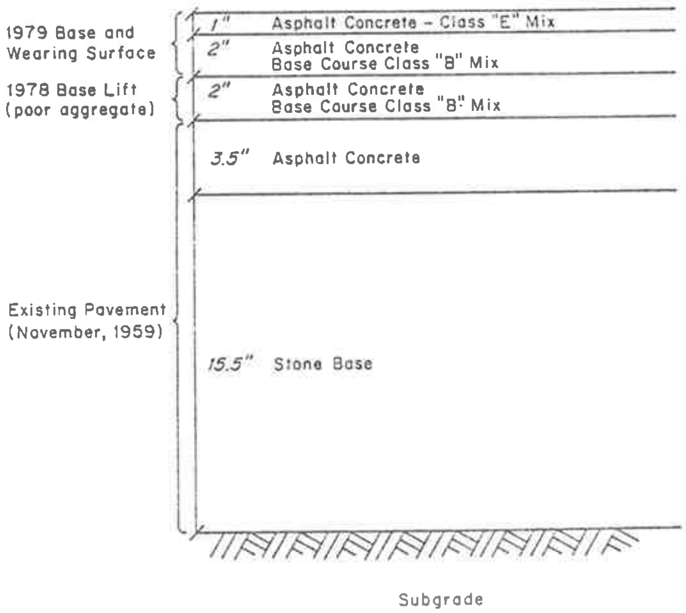

Figure 3. North Oakland - Sutherlin Project Pavement Cross-Section

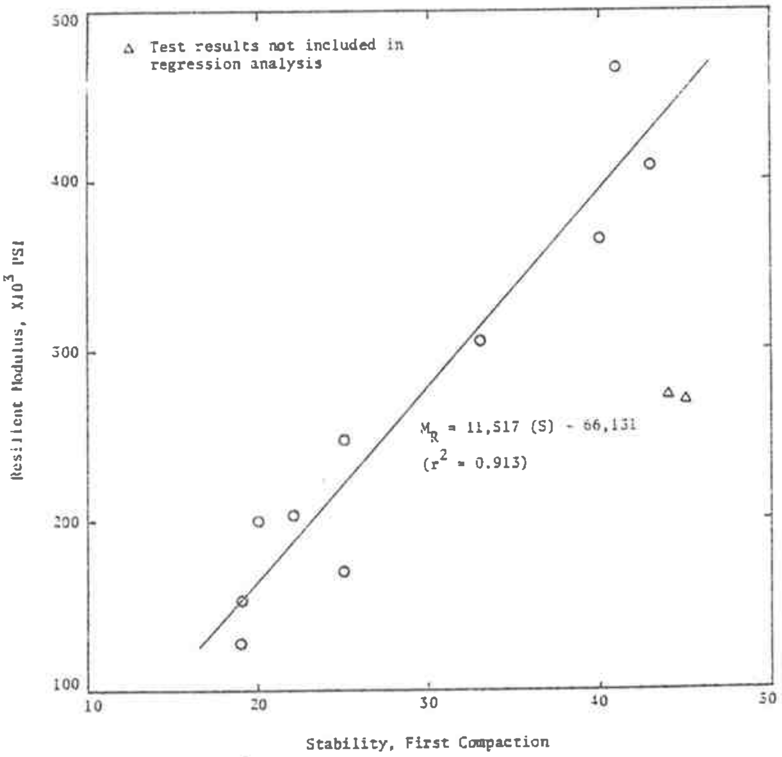

Figure 4. Relationship Resilient Modulus - Stability (from Table 5)

Table 3. Mix Design for 1978 Paving (ODOT)

Asphalt Concrete Base Layer, -B- Mix
North Oakland-Sutherlin Project

Asphalt Content (AR 8000)		5.5	6.0	6.5	7.0	7.5
Asphalt Film Thickness		Suff.	Suff.	Suff.	Suff.-Thick	Thick
Stability Value, 1st Compaction		32	34	35	38	39
Bulk Spec. Gravity, 1st Compaction		2.23	2.25	2.27	2.29	2.31
Percent Voids, 1st Compaction		11.2	9.6	8.1	6.5	4.9
Stability Value, 2nd Compaction		49	50	50	49	52
Bulk Spec. Gravity, 2nd Compaction		2.32	2.34	2.35	2.36	2.38
Percent Voids, 2nd Compaction		7.6	6.0	4.9	3.7	2.1
Rice Gravity		2.51	2.49	2.47	2.45	2.43
AASHTO T 165 Index of Retained Strength	AR 8000	11	-	20	-	45
	AR 8000 + 1% Sucon (No Strip)	62	-	79	-	82
	AR 8000 + 1% Pavebond	64	-	81	-	84
	AR 4000	16	-	25	-	53

Mix Gradation: Percent passing 3/4" - 100; 1/2" - 86; 1/4" - 60; No. 10 - 25; No. 40 - 12; No. 200 - 5

Recommended Asphalt Content (AR 800 - % of total mix): 6.9%

Table 4. Mix Design: Aggregate Gradation, Class B.

SIEVE SIZE	TARGET VALUE	MIX TOLERANCES	SPECIFICATION (Broad Band)
3/4"	100	95-100	95-100
1/2"	86	80 - 92	-
1/4"	60	54 - 66	52 - 72
#10	25	21 - 29	21 - 41
#40	12	8 - 16	8 - 24
#200	5.0	3.0-7.0	2 - 7

Table 5. Summary of Construction Daily Plant Report - 1978 Base Lift - Bituminous Mix Class "B"

In Place Mix Data	Average Value	Standard Deviation	Maximum and Minimum Values
Core Bulk Specific Gravity	2.28 (24 tests)	±.07	2.15 - 2.36
Asphalt Content	7.17 (72 tests)	±.61	5.1 - 8.9
Percent Passing No. 200	6.72 (72 tests)	±1.16	4.2 - 10.1
Percent Passing No. 10	25.66 (72 tests)	±3.55	16.00 - 35.00

materials and excess variation in mix gradation and asphalt content. Also, some problems from improper construction practices were reported.

Aggregates used in this project were from the Oak Creek Quarry, located in the western foothills of the Cascade Range. Rock from this quarry is a submarine basalt containing seams of sulfate compounds of calcium, sodium and magnesium. Such salts are almost nonwater soluble, but highly water absorptive (up to 7% by weight). Aggregate contamination by such salts results in asphalt coating problems and increases susceptibility of the mix to water. Sections of the quarry containing sulfate salts were avoided as much as possible; however, it is possible that part of the aggregate used for the construction of the 1978 base lift was partially coated with sulfate compounds.

Aggregate qualifying test results, field control testing and investigation of mix properties done by ODOT also showed that the reduced quality of the paving is basically the result of using varying amounts of unsound and nondurable aggregate in the mix. Soundness test results for produced aggregate used in the paving range from 4.16 to 38.94% loss for coarse aggregate and 11.56 to 48.23% loss for fine aggregate. With the amount of aggregate found containing material having a soundness loss (AASHTO T-104) greater than the 18% maximum now specified, aggregate breakdown problems would be expected.

Table 5 summarizes the field test results run during pavement construction. The variables considered are the mix bulk specific gravity, asphalt content and percent passing No. 200 and No. 10. Compared to the core data (Table 6) and the specified job mix tolerances (Table 4), it appears that much of the paving was far beyond the expected production tolerances, with the amount passing No. 200 being the most severe.

The construction reports (Table 5) also indicate that the mix variables were ranging within a very wide band, indicating quality control problems during mixing (asphalt content, gradation) and during compaction (mix bulk specific gravity). This is corroborated by the field inspection reports indicating problems with production control and workmanship.

Table 6. Comparison Between Construction Information, Core Data and Mix Specification for Base Lift

	Daily Plant Test Results	Average Core Data*	Job Mix Tolerance
Mix Bulk Specific Gravity	2.28	2.25	-
Asphalt Content, %	7.17	7.14	6.4 - 7.4
Percent Passing No. 200	6.72	6.33	3 - 7
Percent Passing No. 10	25.66	31.58	27 - 35

* For follow-up core data, see Appendix A

Consequently, the pavement quality was largely reduced. ODOT field tests for production control of the mix indicated that the pavement deficiencies found were mainly the result of excess passing No. 200 sieve.

3.0 TEST RESULTS - ODOT

The Oregon Department of Transportation testing program included conventional tests such as standard Hveem mix design, AASHTO T-165, index of retained strength and diametral modulus over a range of asphalt contents, gradations, and mix densities. Samples of pavement were tested for gradation, asphalt content, void content, percent compaction and recovered asphalt properties. This chapter presents the results of this work.

Mix Design Data

The results of the mix design tests are presented in Table 7. For each set of variables, standard samples were tested to determine mix characteristics. The percent voids of all samples prepared for this project were determined using the Rice gravities indicated in Table 7. Modulus and bulk specific gravities shown in this table were used as reference values during sample preparation at Oregon State University. A good correlation was found between the stability at first compaction and resilient modulus values, as indicated in Figure 4.

Core Data

Two core sampling sites were selected on the North Oakland-Sutherlin project. For each site, layer thicknesses were recorded and core samples of the top and the two base lifts were collected. Five cores were taken at 2 foot intervals across the pavement panel at each sampling site. Table 8 summarizes the results of the tests run on the core samples (See Appendix A for details). Shown in Figures 5 and 6 are the construction and core aggregate gradation for the top and base mixes. The recommended gradation shown along with the wearing course aggregate gradation (Figure 5) is the Oregon specification for Class E mixes. For the base lift, the recommended gradation shown corresponds to the job mix tolerance reported in Table 4. Figure 6 shows that the base lift mix is not entirely within the specifications. From Tables 5 and 6, it is possible to compare the laboratory samples with the field cores. For the B mix, the bulk specific gravity of the cores ranges between 2.15 and 2.36 for the 1978 paving with an asphalt content between 5.1 and 8.9% and the percent passing No. 200 between 4.2 and 10.1%.

Table 7. SUMMARY OF MIX DESIGN INVESTIGATION FOR EVALUATION OF MIX VARIATIONS

MIX TYPE		2% Passing No. 200			6% Passing No. 200			10% Passing No, 200			92% Compaction, 6% Pass No. 200			88% Compaction, 6% Pass No, 200		
ASPHALT CONTENT		5.0	6.0	7.0	5.0	6.0	7.0	5.0	6.0	7.0	5.0	6.0	7.0	5.0	6.0	7.0
Rice Gravity, T209		2.53	2.49	2.45	2.53	2.49	2.45	2.53	2.49	2.45	2.53	2.49	2.45	2.53	2.49	2.45
1st Bulk S.G.		2.23	2.27	2.31	2.32	2.36	2.36	2.34	2.38	2.41	2.16	2.20	2.24	2.14	2.17	2.17
Voids, %		11.9	8.8	5.7	8.3	5.2	3.7	7.5	4.4	1.6	14.6	11.6	8.6	15.4	12.9	11.4
2nd Bulk S.G.		2.32	2.36	2.40	2.39	2.43	2.43	2.42	2.46	2.45	-	-	-	-	-	-
Voids, %		8.3	5.2	2.0	5.5	2.4	.8	4.3	1.2	0.0	-	-	-	-	-	-
Modulus*, psi @ 1st Bulk S.G.		466,000	274,000	245,000	-	-		408,000	365,000	305,000	248,000	204,000	170,000	200,000	152,000	126,000
Stability, 1st		41	44	45	38	42	39	43	40	33	25	22	25	20	19	19
Stability, 2nd		59	58	-	62	50	44	56	41	16	-	-	-	-	-	-
Index of retained strength, %	Without Pave. Bond	18%	21%	41%	15%	32%	56%	11%	28%	60%	28%	39%	57%	30%	42%	64%
	With Pave. Bond	-	52%	-	-	69%	-	-	66%	-	-	84%	-	-	82%	-
P200 Extracted, Dry Sieve		-	3.2	-	-	6.0	-	-	8.8	-	-	6.0	-	-	6.0	-
P200 Extracted, Wet Sieve		-	4.2	-	-	7.1	-	-	9.7	-	-	7.2	-	-	7.4	-

Table 8. Core Data for North Oakland-Sutherlin
(Mean ± Standard Deviation)

		Core Data		
		Top Layer	Base Layer-1979	Base Layer-1978
Thickness		1.37" ± .18	3.74" ± 1.13	-
Bulk S.G.		2.09 ± .03	2.25 ± .06	2.23 ± 0.08
% Voids In Place		17.22 ± 1.22	9.32 ± 2.69	9.95 ± 3.68
Modulus (x 10^3 PSI)		218.0 ± 41.0	424.0 ± 92.0	331 ± 77
Gradation Range, % Passing	3/4"	100	100	N/A*
	1/2"	90.1 - 100.0	79.2 - 95.0	N/A
	3/8"	74.3 - 91.4	65.1 - 88.5	N/A
	1/4"	51.4 - 68.3	53.1 - 73.1	N/A
	No. 4	44.5 - 53.1	48.1 - 57.0	N/A
	No. 10	13.0 - 18.8	27.2 - 34.9	N/A
	No. 40	4.9 - 7.5	12.4 - 15.4	N/A
	No. 200	3.3 - 5.1	5.5 - 7.0	N/A
% A.C.		6.1 - 8.3	6.4 - 8.0	N/A
Recovered Asphalt Properties AASHTO T-170	Penetration at 77°F (25°C) (cm/100)	15 ± 2	14 ± 2	-
	Kinematic Viscosity at 275°F (135°C) (CS)	836 ± 7	814 ± 43	-
	Absolute Viscosity at 140°F (60°C)	16,100 ±880	13,650 ±3,492	-

*Not Available

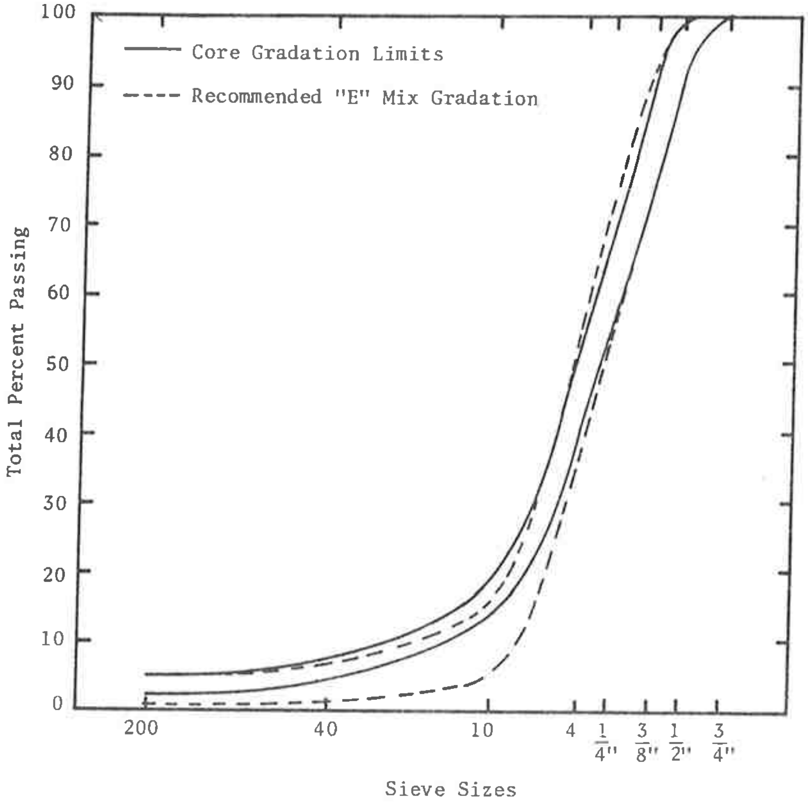

Figure 5. Core Gradation, Top Layer

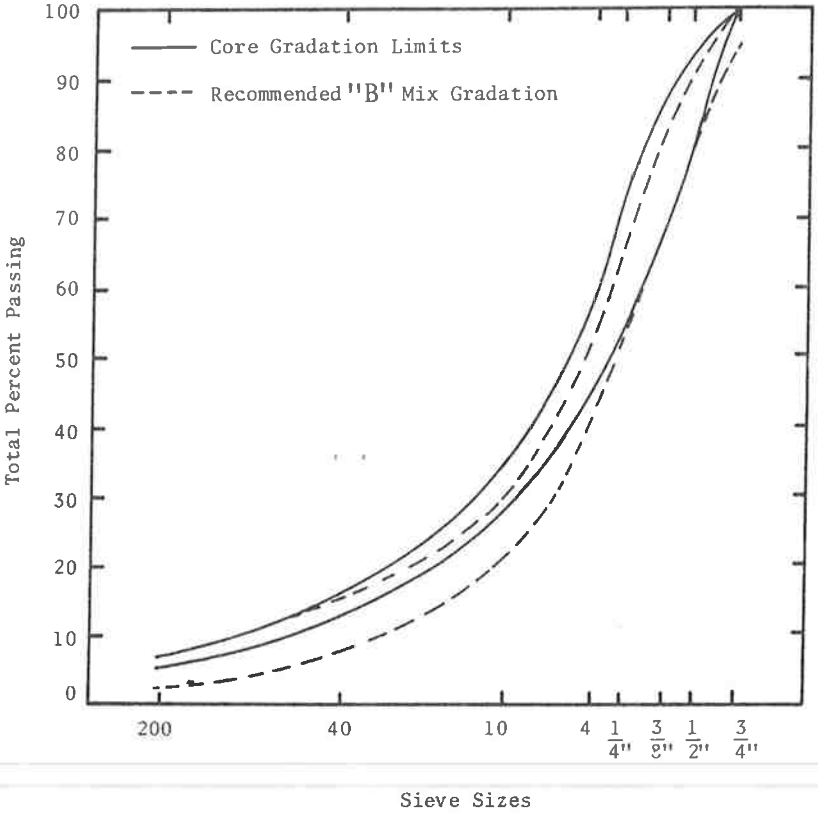

Figure 6. Core Gradation, Base Layer

This shows an extreme variation from the laboratory design. From Tables 7 and 8, the resilient modulus of the laboratory compacted samples is in the range of 127,000 to 466,000 psi, while the core samples showed an average resilient modulus of 331,000 to 424,000 psi. This is reasonably good agreement when considering the effect of sample densities and asphalt properties. Also, differentiation between the 1978 and 1979 base lifts was not easy during core sampling. It is likely that the 2.5 inch (6.4 cm) thick samples used for testing were partially trimmed off from the 1979 base layer. This would certainly result in a substantial increase in mix resilient modulus since the 1979 base layer was built using good quality aggregate.

The results presented in this chapter indicate there was considerable variation in mix properties on this project. To evaluate the effects of these variations on pavement life, a laboratory program was developed. It, together with the results, are described in the next chapter.

4.0 TEST RESULTS - OSU

The purpose of the tests performed at Oregon State University was to determine the fatigue life and permanent deformation characteristics of the asphalt mix. All tests were performed over the selected range of variables on standard laboratory samples using the repeated load indirect tensile test. The samples were prepared according to the Oregon State Highway Division standard procedure (2). The materials used are the same as used for the mix design reported in Section 3.0.

Test Program

A minimum of 16 samples were prepared for each condition. Eight samples were tested as compacted, and eight samples tested after conditioning.* Table 9 gives the flow chart for the test program followed. Results of the indirect tensile tests run for each set of conditions on two as-compacted samples and two conditioned samples are presented at the end of this chapter. These data are useful to compare the dynamic tests results with a standard mix design test. The principal variables studied included:

(1) Mix density: 100-96-92-91%

(2) Percent passing No. 200 sieve: 2-6-10%

(3) Percent asphalt content: 5-6-7%

(4) Aggregate quality

Two secondary variables were also studied: the percent passing No. 10 and use of Pavebond (anti-strip agent). Each of the above variables was studied relative to a standard mix, consisting of 6% passing No. 200 sieve and 6% asphalt content. When studying the influence of the mix density, the standard mix was compacted at 96%, while a 92% compaction standard mix was selected to

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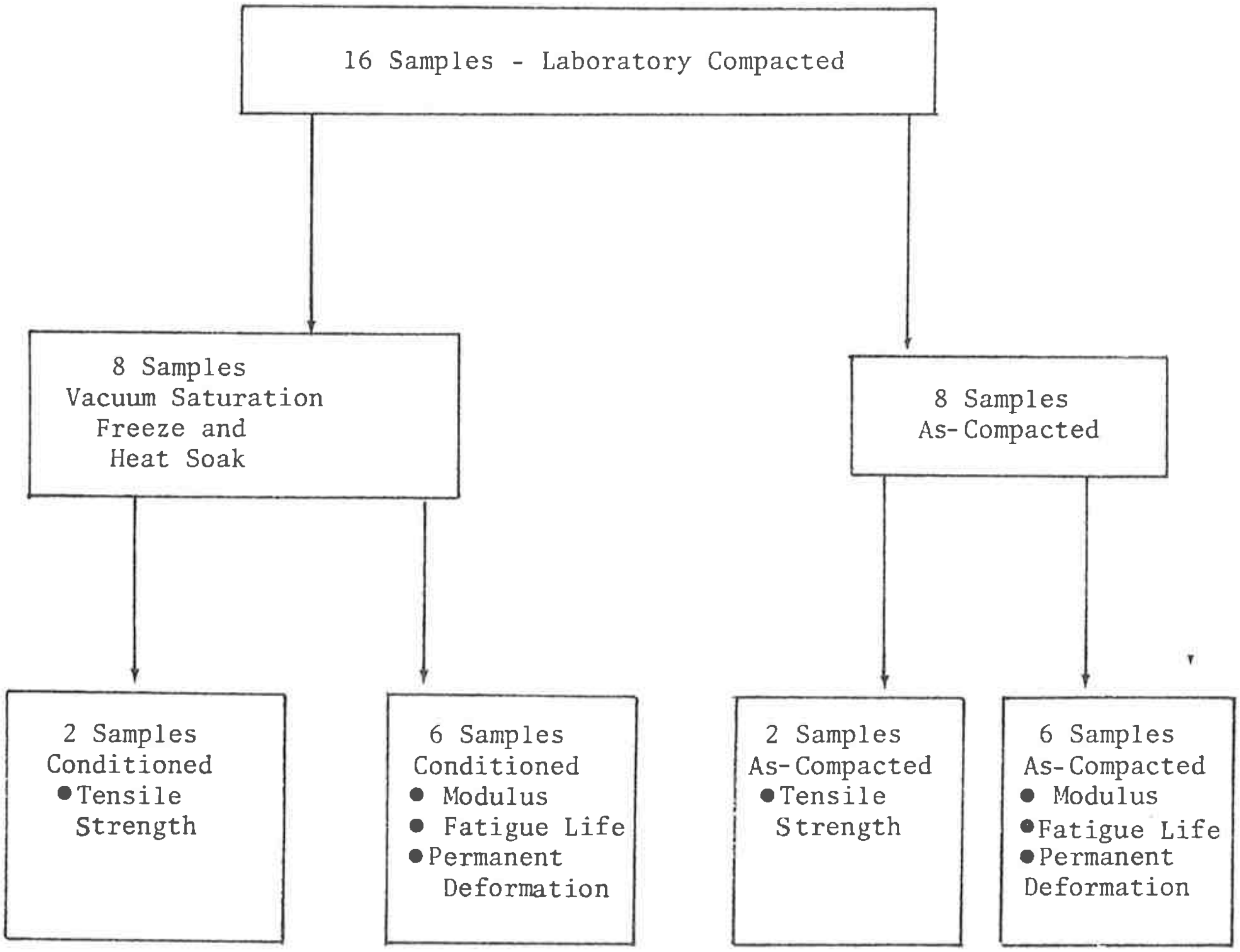
*The sample conditioning procedure followed was based on the moisture damage test defined by Lottman (3). The following are the main steps:

1. Determine the resilient modulus of the as-compacted samples, after overnight cure. Mark along which samples axis the modulus was measured.
2. Vacuum saturate the samples for two hours.
3. Place the saturated samples in a freezer at -18°C for 15 hours.
4. Place the frozen, saturated specimen in a warm water bath for 24 hours.
5. Place the specimen in a water bath at room temperature for 3 hours.
6. Run the mechanical property tests, along the same sample axis as the as-compacted modulus was measured (Step 1).

Table 9. Experimental Test Program



study the influence of the amount of fines, asphalt content and aggregate quality. Details of the combination of variables used in this analysis are shown in Table 10.

Test Equipment

Figure 7 shows the testing equipment used to determine the resilient modulus, fatigue life and permanent deformation characteristics of the specimens. Testing conditions were kept constant, and are summarized as follows:

(1) A static load of 10 lbs. was applied to hold the sample in place.
(2) The dynamic load duration was fixed at 0.1 seconds and the load frequency at 60 cycles per minute.
(3) Test temperature was defined as the average sample temperature during testing (normally 22°C ± 2°C).
(4) Load platens are 1/2 inch (1.3 cm) wide.

Test Procedure and Calculations

All tests were run for mix tensile strain ranging between 50 and 150 microstrain. The parameters recorded during the dynamic diametral test are the maximum load applied, the sample horizontal elastic deformation and the sample vertical permanent deformation (Figure 8).

Dynamic diametral tests were run using the following procedure:

(1) Place the sample in the dynamic diametral test apparatus.
(2) Apply approximately 100 load applications until the permanent deformation recorded is negligible compared to the sample elastic response.
(3) Adjust the dynamic load to achieve the desired initial mix tensile strain.
(4) Maintain the control set at the load level required and start the fatigue life tests (also monitor permanent deformation).
(5) Record the number of repetitions to failure.

The maximum load applied and the horizontal elastic tensile deformation were recorded to determine the modulus using the following equation (4):

$$M_R = \frac{P}{\Delta H \times h}(.2692 + .9974\nu) \qquad (1)$$

Table 10. Range of Mix Variables Considered in This Study (Crossed Boxes)
North Oakland - Sutherlin Project

Level of Compaction	2% Passing No. 200			6% Passing No. 200			10% Passing No. 200		
	Asphalt Content			Asphalt Content			Asphalt Content		
	5	6	7	5	6	7	5	6	7
2nd Compaction					☒				
1st Compaction					☒				
95 Blows at 100 psi 500 psi Leveling Load	☒		☒	☒	☒	☒	☒		☒
30 Blows at 100 psi 300 psi Leveling Load					☒				

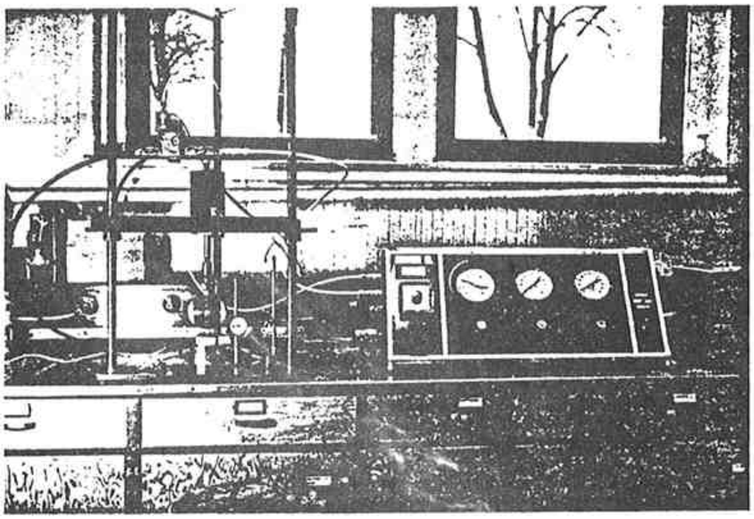


Figure 7. Diametral Test Apparatus.

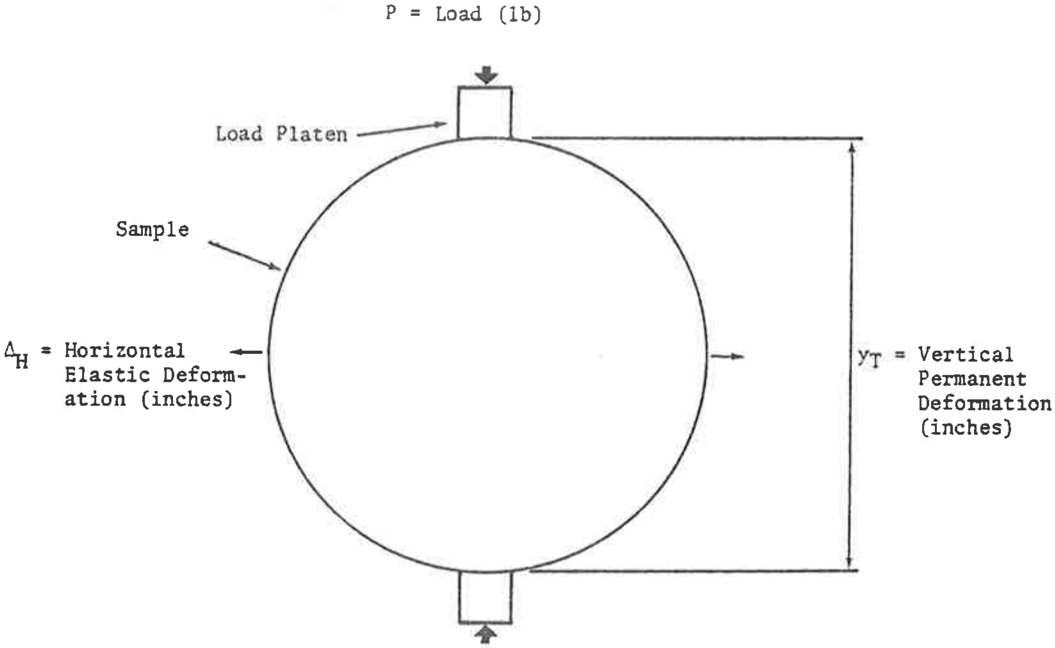


Figure 8. Diametral Test · Variables Recorded

where ΔH = Horizontal elastic tensile deformation, inches
P = dynamic load, pounds
h = sample thickness, inches
ν = Poisson's ratio

Poisson's ratio was assumed constant and equal to .35, which simplifies equation (1) to:

$$M_R = \frac{P \times .6183}{\Delta H \times h} \tag{2}$$

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 sample 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 (5,6,7,8):

$$N_f = K\left(\frac{1}{\varepsilon_t}\right)^m \tag{3}$$

where: N_f = Number of load repetitions to failure
K, m = Regression constants
ε_t = Horizontal elastic tensile strain

The fatigue life of a specific mix is therefore defined by the constants K and m. For each set of mix variables, six samples were tested at different values of the initial tensile strain. The number of load repetitions to failure was then measured and recorded. The constants K and m are determined using linear regression by the method of least squares. The tensile strain ε_t is calculated from the following equation (3):

$$\varepsilon_t = \Delta H \left[\frac{.03896 + .1185\nu}{.0673 + .2494\nu}\right] \tag{4}$$

where ε_t = Horizontal elastic tensile strain
ΔH = Horizontal elastic tensile deformation, inches
ν = Poisson's ratio

Assuming the Poisson's ratio is constant and equal to .35, equation (4) becomes:

$$\varepsilon_t = \Delta H \times .5203 \tag{5}$$

The number of load repetitions to fatigue failure was defined as the number of repetitions required to get a vertical crack approximately 1/4 inch (.64 cm) wide in the samples. To stop the test at the specified level of sample deformation, a thin aluminum strip was attached to the sides of the samples, along a plane perpendicular to the plane formed by the load platen (see Figure 9). The aluminum strip is connected to a normally closed relay, which controls the dynamic load system. As the sample deforms, the aluminum strip is stressed. When the sample deformation exceeds a certain level, the aluminum strip breaks and opens the relay, which shuts off the test. Proper calibration of the length of the aluminum strip will cause the test to stop for a specific sample crack width (Figure 10).

The vertical permanent strain is also recorded during the fatigue test as a function of the number of load repetitions. The permanent deformation strain is given by (3):

$$\varepsilon_c = \mu_t \left[\frac{-.03896\nu - .1185}{.0156\nu - .8954} \right] \tag{6}$$

where ε_c = Vertical permanent compressive strain
μ_t = Vertical permanent compressive deformation, inches
ν = Poisson's ratio

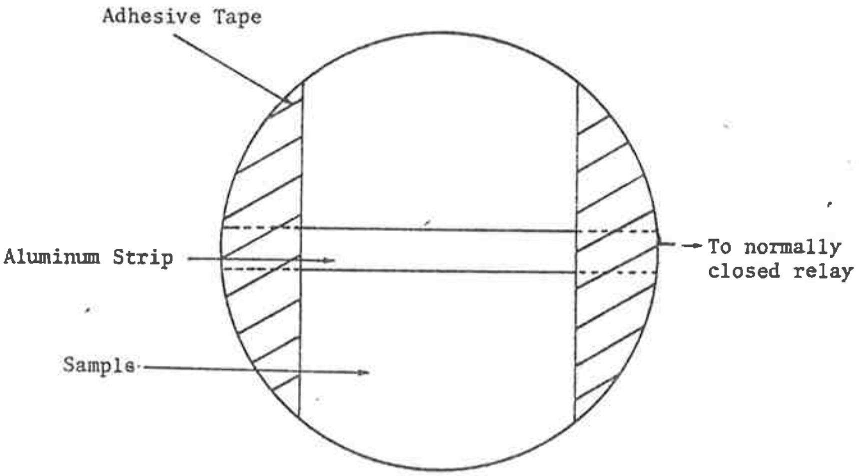
If the Poisson's ratio is assumed constant and equal to .35, equation (6) becomes:

$$\varepsilon_c = \mu_t \times .1485 \tag{7}$$

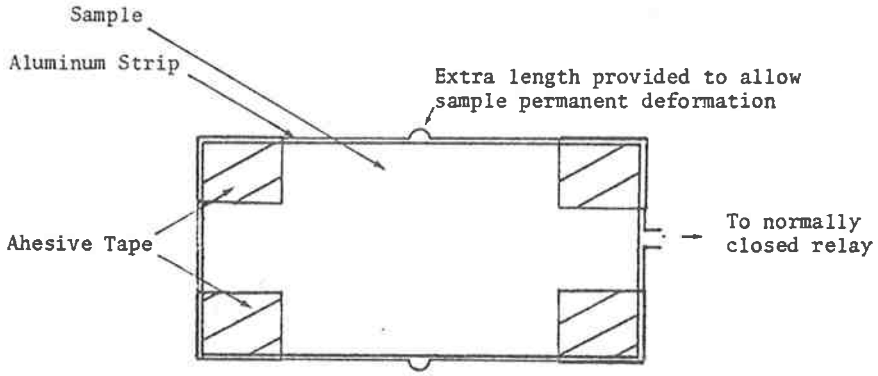
Resilient modulus, fatigue and permanent deformation models and tensile strength values have been determined for each set of variables considered in this study. The significance of these results and their correlation with other mix properties are developed later in this chapter.

Resilient Modulus Data

The resilient modulus of the as-compacted samples are presented in Table 11a. The influence of the bulk specific gravity on the mix resilient modulus, independent of the other variables, can be observed for the 6% asphalt



a. Side view



b. Top view

Figure 9. Schematic View of the Automatic Shut-Off Device for Fatigue Testing

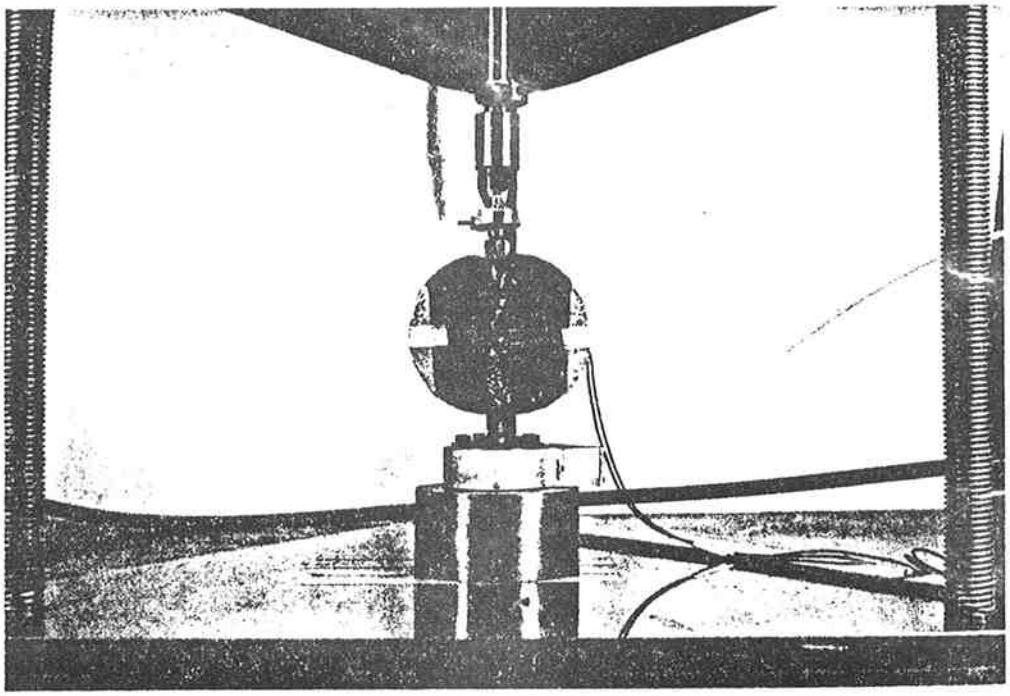


Figure 10. Sample at End of Fatigue Test

content and 6% passing No. 200 conditions. The relationship between modulus and bulk specific gravity appears to be almost linear and is by changes in asphalt content and percent passing the No. 200 sieve as shown in Figure 11. The nonlinearity of the curves joining the points of equal asphalt content indicate the interaction of the asphalt content and fines on the mix stiffness. Figure 12 illustrates the relationship between the mix resilient modulus and the amount of fines at a constant level of compaction and for 5, 6 and 7% asphalt content. The rapid increase in stiffness for increasing percentage of fines is particularly important at higher percentages of asphalt content. Figures 11 and 12 show a decrease in the mix resilient modulus when the asphalt content increases.

Moduli values of conditioned samples are presented in Table 11b together with their percentage of retained stiffness (compared with the samples modulus measured before conditioning). These data indicate that higher retained strengths are obtained at high asphalt contents and/or mix densities. The relationship between conditioned modulus and the mix bulk specific gravity (Figure 13) appears less affected by asphalt content and percent fines than in the case of as-compacted samples. Figure 14 indicates that mix stiffness is relatively independent of asphalt content for low percent fines (2%). However, at higher percent fines (10%), increasing amount of asphalt does increase the mix stiffness.

Average resilient modulus determined for samples prepared with good and low quality aggregate are compared in Table 12. All other mix variables being kept constant for both mixes: 6% passing No. 200, 25% passing No. 10, 6% asphalt content and 97% compaction. The slight increase in the as-compacted moduli values of the good quality aggregate mix shows that aggregate quality affects directly the aggregate-asphalt bond in the mix, even before any weathering action occurred. The difference between the two mixes is emphasized after conditioning. A 45% reduction in modulus can be observed for the low quality aggregate, whereas the stiffness lost after conditioning does not exceed 30% of the initial modulus for the samples prepared with good quality aggregate.

The influence of the two secondary variables, Pavebond and percent passing No. 10, is shown in Tables 13 and 14. Two test series were prepared with the anti-strip agent. The first series consisted of adding asphalt and Pavebond to the aggregate without pre-blending, while the second consisted

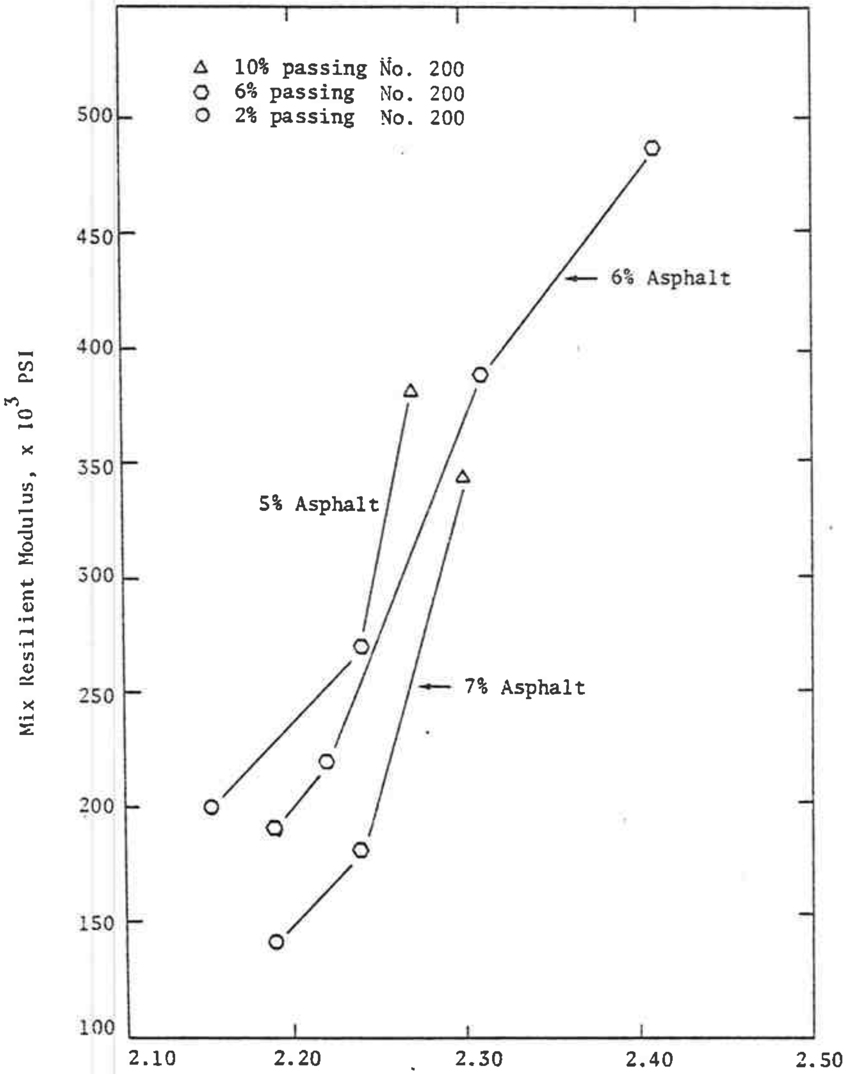


Figure 11. Influence of Bulk Specific Gravity on Resilient Modulus - as Compacted Samples

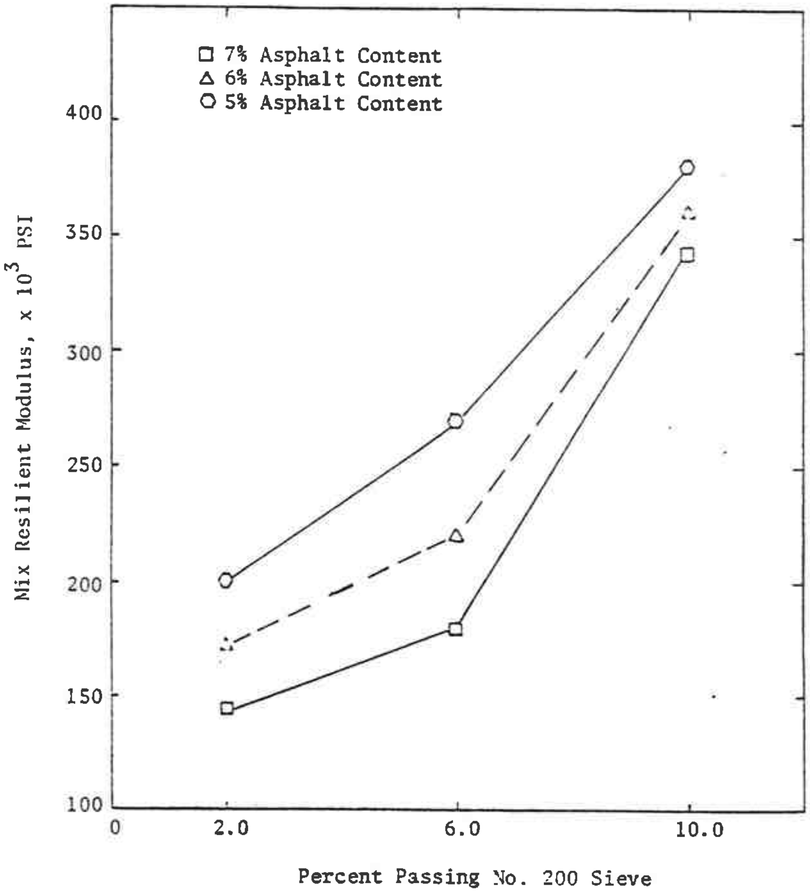


Figure 12. Influence of Amount of Fines on Resilient Modulus - As Compacted Samples.

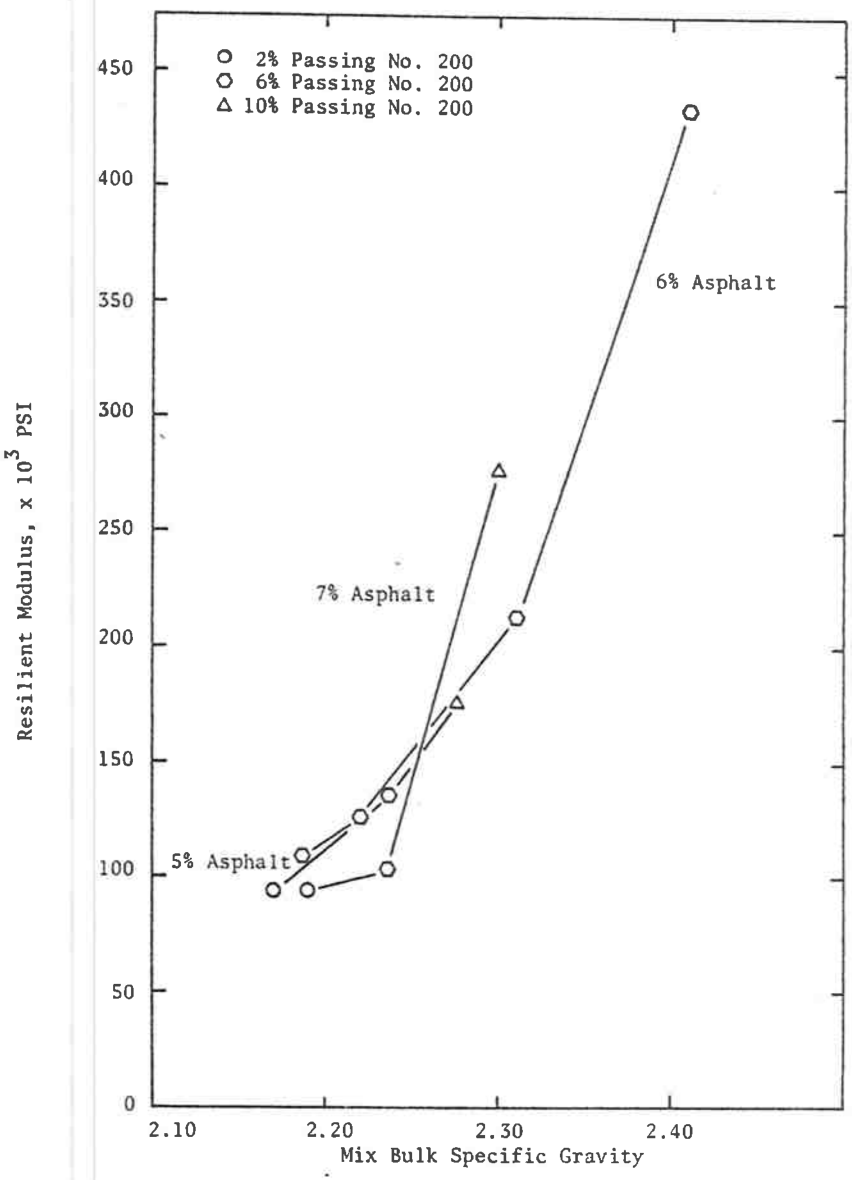


Figure 13. Influence of Bulk Specific Gravity on Resilient Modulus. Conditioned Samples

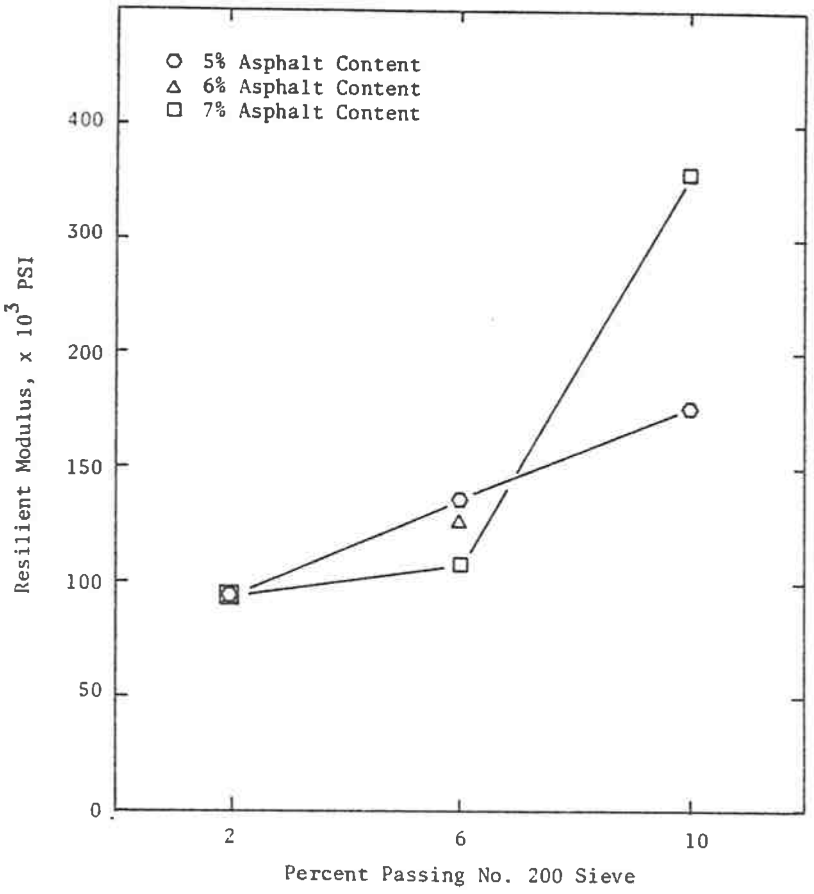


Figure 14. Influence of Amount of Fines on Resilient Modulus - Conditioned Samples

Table 11a. Resilient Modulus Data, as Compacted Samples
- Resilient Modulus, (PSI)
- Percent Voids
- Bulk Specific Gravity

Level of Compaction	2% Passing No. 200			6% Passing No. 200			10% Passing No. 200		
	Asphalt Content, %			Asphalt Content, %			Asphalt Content, %		
	5	6	7	5	6	7	5	6	7
2nd Compaction					488,000 3.28 2.41				
1st Compaction					389,000 7.33 2.31				
95 Blows at 100 psi 500 psi Leveling Load	200,000 14.23 2.17		143,000 10.46 2.19	270,000 11.46 2.24	220,000 10.85 2.22	180,000 8.45 2.24	381,000 10.47 2.27		343,000 6.00 2.30
30 Blows at 100 psi 300 psi Leveling Load					191,000 11.95 2.19				

Table 11b. Resilient Modulus Data - Conditioned Samples
- Resilient Modulus, PSI
- % Retained Stiffness $\left(\frac{\text{Conditioned Modulus}}{\text{As Compacted Modulus}} \times 100\right)$

Level of Compaction	2% Passing No. 200			6% Passing No. 200			10% Passing No. 200		
	Asphalt Content, %			Asphalt Content, %			Asphalt Content, %		
	5	6	7	5	6	7	5	6	7
2nd Compaction					435,000 89.1%				
1st Compaction					214,000 55.0%				
95 Blows at 100 psi 500 psi Leveling Load	93,000 46.5%		93,000 65.0%	136,000 50.4%	126,000 57.3%	103,000 57.2%	176,000 46.2%		279,000 81.3%
30 Blows at 100 psi 300 psi Leveling Load					109,000 57.1%				

Table 12. Influence of Aggregate Quality on Resilient Modulus.

Aggregate Quality	Resilient Modulus, PSI	
	As Compacted	Conditioned
Good	444,000	311,000
Low	389,000	214,000

All Samples Prepared With:

6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt Content - 96% Compaction

Table 13. Effect of Pave-Bond on Resilient Modulus

Type of Mix	Resilient Modulus, psi	
	As-Compacted	Conditioned
Standard*	389,000	214,000
Standard with Pave-Bond Added During Mixing	412,000	292,000
Standard With Pave-Bond Added To Asphalt	464,000	443,000

---

*6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt - 96% Compaction

Table 14. Influence of Percent Passing No. 10 on Mix Resilient Modulus

Type of Mix	Resilient Modulus, psi		Percent Voids
	As-Compacted	Conditioned	
Standard*	220,000	126,000	10.85
Mix With 30% Passing No. 10	230,000	129,000	10.69
Mix With 35% Passing No. 10	204,000	129,000	11.32

* 6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt - 92% Compaction

of thoroughly blending the Pavebond with the asphalt before mixing. A comparison between the results from the two methods used to add the Pavebond shows the importance of proper dilution of the Pavebond in the asphalt before mixing. A reduction of 45% of the standard mix modulus was noted after sample conditioning. This stiffness reduction dropped to 29% when the Pavebond was directly added to the mix and dropped to 4.5% when the Pavebond was added to the asphalt before the mixing process. Also to be noted is the 16% increase in the as-compacted modulus between the standard mix and the mix prepared with Pavebond added to the asphalt.

From the results presented in Table 14, variations of plus or minus 5% from the optimum percent passing the No. 10 sieve do not affect substantially the mix resilient modulus.

In summary, the as-compacted mix stiffness increases with increasing percent fines and decreases with increasing asphalt content, whereas the conditioned sample modulus behave in a more complex way depending on the relative amount of fines and asphalt in the mix.

Fatigue Data

The fatigue life of asphalt mixes is a function of initial tensile strain and follows the equation:

$$N_f = K(\varepsilon_t)^m \qquad (8)$$

where N_f = Number of load repetitions to failure
ε_t = Initial tensile strain
K,m = Regression constants

Both K and m are affected by the mix variables. For each set of mix conditions, six samples were tested at the following initial tensile strains: 50, 65, 85, 100, 125 and 150 microstrains. The coefficients K and m were then determined by linear regression analysis. Table 15a shows the K and m values found for different percentages of asphalt, amount of fines and level of compaction. From this data, the relationship between -m and log (K) was plotted in Figure 15. The relationship m versus $\ell n(K)$ follows the equation:

$$m = A_1 \ \ell n(K) + A_o \qquad (9)$$

Table 15a. Fatigue Data - As Compacted Samples
$(N_F = K(\varepsilon_T)^m)$*

Level of Compaction	2% Passing No. 200			6% Passing No. 200			10% Passing No. 200		
	Asphalt Content			Asphalt Content			Asphalt Content		
	5	6	7	5	6	7	5	6	7
2nd Compaction					K=2.58x10^{-17} m=-5.44 log K=-16.59				
1st Compaction					K=2.13x10^{-17} m=-5.32 log K=-16.67				
95 Blows at 100 psi 500 psi Leveling Load	K=.131 m=-1.24 log K=-.88		K=1.45x10^{-2} m=-1.49 log K=-1.84	K=1.18x10^{-7} m=-2.80 log K=-6.93	K=1.51x10^{-6} m=-2.52 log K=-5.82	K=4.16x10^{-6} m=-2.44 log K=-5.38	K=1.58x10^{-7} m=-2.81 log K=-6.80		K=6.25x10^{-5} m=-2.21 log K=-4.20
30 Blows at 100 psi 300 psi Leveling Load					K=1.20x10^{-4} m=-2.05 log K=-3.92				

* N_F : Number of load repetitions to failure
ε_T : Mix elastic tensile strain
K,m: Regression constants

Table 15b. Fatigue Data - Conditioned Samples
$(N_F = K(\varepsilon_T)^m)$*

Level of Compaction	2% Passing No. 200			6% Passing No. 200			10% Passing No. 200		
	Asphalt Content			Asphalt Content			Asphalt Content		
	5	6	7	5	6	7	5	6	7
2nd Compaction					K=3.80x10^{-8} m=-3.08 log K=-7.42				
1st Compaction					K=1.02x10^{-6} m=-2.65 log K=-5.99				
95 Blows at 100 psi 500 psi Leveling Load	K=19.4 m=-.64 log K=1.29		K=2.68x10^{-1} m=-1.16 log K=-0.57	K=1.02x10^{-3} m=-1.84 log K=-2.99	K=1.55x10^{-5} m=-2.33 log K=-4.81	K=5.16x10^{-4} m=-1.96 log K=-3.29	K=1.56x10^{-5} m=-2.35 log K=-4.81		K=1.55x10^{-5} m=-2.42 log K=-4.81
30 Blows at 100 psi 300 psi Leveling Load					K=3.6x10^{-3} m=-1.64 log K=-2.44				

* N_F : Number of load repetitions to failure
ε_T : Mix elastic tensile strain
K,m: Regression constants

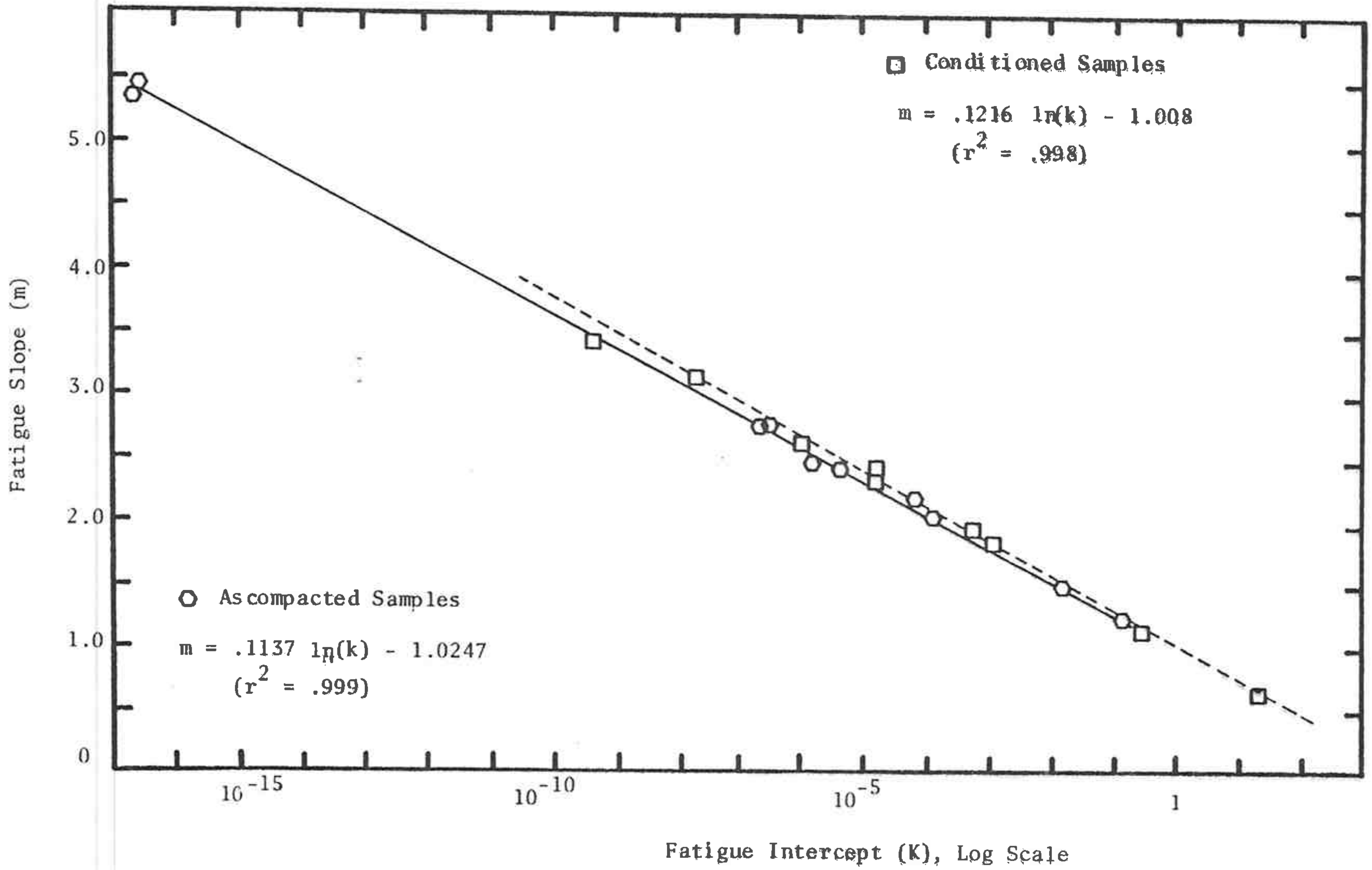


Figure 15. Relationship Between K and m

Linear regression run on the as compacted data give:

$A_1 = .114$
$A_o = -1.03$
$r^2 = .999$

It can be deduced from this relationship between m and $\ell n(K)$ that the fatigue curves, expressed in Number of Load Repetitions versus Mix Tensile Strain, should intercept at a common point, called the focus point (4,12,13). The coordinates of this focus point (ε_o, N_o) can be deduced from equations (8) and (9):

$$m = A_o + A_1 \, \ell n(K) \tag{10}$$

$$N_F = K(\varepsilon)^m \Longleftrightarrow \ell n(N_F) = \ell n(K) + m \, \ell n(\varepsilon) \tag{11}$$

(11) is also true at the focus point

$$\ell n(N_o) = \ell n(K) + m \, \ell n(\varepsilon_o)$$

$$m = \frac{\ell n(N_o)}{\ell n(\varepsilon_o)} - \frac{1}{\ell n(\varepsilon_o)} \times \ell n(K) \tag{12}$$

Comparison between equations (10) and (12) gives:

$$A_o = \frac{\ell n(N_o)}{\ell n(\varepsilon_o)} \qquad \text{and} \qquad A_1 = \frac{-1}{\ell n(\varepsilon_o)}$$

Coordinates of the as compacted samples focus point gave:

$$\varepsilon_o = 151 \times 10^{-6}$$

$$N_o = 8.21 \times 10^3$$

Knowing the coordinates of the focus point, linear regression analysis were rerun for each set of samples, and fitted through the focus point. Table 16a gives the new K and m values, noted K' and m'.

The same approach was followed for the conditioned test results. Table 15b gives the K and m values computed from the test results. The relationship found between K and m is :

Table 16a. Corrected Fatigue Data - As Compacted Samples
$(N_f = K (\epsilon_t)^m)$*

Level of Compaction	2% Passing #200			6% Passing #200			100% Passing #200		
	Asphalt Content			Asphalt Content			Asphalt Content		
	5	6	7	5	6	7	5	6	7
2nd Compaction					$K = 1.41 \times 10^{-22}$ $m = -6.74$				
1st Compaction					$K = 3.36 \times 10^{-7}$ $m = -2.72$				
95 Blows at 100 PSI 500 PSI Leveling Load	$K = 3.28 \times 10^{-1}$ $m = -1.15$		$K = 4.96 \times 10^{-2}$ $m = -1.37$	$K = 1.12 \times 10^{-5}$ $m = -2.32$	$K = 1.57 \times 10^{-5}$ $m = -2.28$	$K = 1.5. \times 10^{-6}$ $m = -2.55$	$K = 5.70 \times 10^{-8}$ $m = -2.92$		$K = 2.23 \times 10^{-8}$ $m = -3.03$
30 Blows at 100 PSI 500 PSI Leveling Load					$K = 1.47 \times 10^{-4}$ $m = -2.03$				

*N_f = Number of Load Repetitions to Failure

ϵ_t = Mix Elastic Tensile Strain

K m = Corrected Regression Constraints $(m = .11369 \, l_N(K) - 1.0247)$

Table 16b. Corrected Fatigue Data Conditioned Samples
$(N_f = K^1 (\epsilon_t)^{m^1})$*

Level of Compaction	2% Passing #200			6% Passing #200			10% Passing #200		
	Asphalt Content			Asphalt Content			Asphalt Content		
	5	6	7	5	6	7	5	6	7
2nd Compaction					$K_1^1 = 7.75 \times 10^{-8}$ $m^1 = -3.00$				
1st Compaction					$K_1^1 = 6.67 \times 10^{-5}$ $m^1 = -2.18$				
95 Blows at 100 PSI 500 PSI Leveling Load	$K^1 = 2.93 \times 10^{1}$ $m^1 = -.597$		$K^1 = 5.40 \times 10^{-1}$ $m^1 = -1.08$	$K^1 = 1.21 \times 10^{-3}$ $m^1 = -1.82$	$K^1 = 1.21 \times 10^{-4}$ $m^1 = -2.10$	$K^1 = 5.81 \times 10^{-5}$ $m^1 = -2.19$	$K^1 = 1.64 \times 10^{-5}$ $m^1 = -2.35$		$K_1^1 = 3.59 \times 10^{-7}$ $m^1 = -2.81$
30 Blows at 100 PSI 300 PSI Leveling Load					$K^1 = 3.17 \times 10^{-1}$ $m^1 = -1.15$				

*N_f = Number of Load, Repetitions to Failure

ϵ_t = Mix Elastic Tensile Strain

K , m = Corrected Regression Constraints $(m = .1269 \, l_N(K) - 1.008)$

$$m = .1216\ \ell n(K) - 1.008 \tag{13}$$

The coordinates of the focus point for the conditioned samples are ε_o = 269.1 x 10^{-6} microstrain and N_o = 3.96 x 10^3 load repetitions. The corrected fatigue parameters (K', m') for the conditioned samples are given in Table 16b.

The effect of asphalt content, percent passing No. 200 and level of compaction on fatigue life can be estimated directly by plotting for each set of conditions, mix tensile strain versus the number of repetitions to failure. The level of compaction fatigue curves for 6% asphalt content and 6% passing the No. 200 sieve are presented in Figure 16a for the as-compacted samples and Figure 16b for the conditioned samples. The as-compacted mix shows a gradual decrease in fatigue life as the mix density drops from 96% to 91%, whereas conditioned samples fatigue life dropped slightly from 96% to 92% and considerably from 92% to 91% compaction. At this time,there is no explanation for the differences noted. The influence of the asphalt content is illustrated in Figure 17. As indicated, asphalt content has very little influence on the fatigue life of the as-compacted samples. Conditioned samples were more sensitive to changes in asphalt content, and indicate that fatigue life increases when the asphalt content increases as found by others.

The influence of the percent passing No. 200 sieve on the mix fatigue life is shown in Figure 18 for 5% asphalt content and in Figure 19 for 7% asphalt content. These figures show the importance of the percent of fines in the mix on fatigue performance. The fatigue life increases with increasing percent of passing No. 200, independent of the asphalt content. Sample conditioning emphasizes the importance of the fines and indicates that a mix with 10% passing No. 200 performs better in the fatigue mode than a mix with 6% (or 2%) passing No. 200 sieve due to increased binder stiffness.

A comparison between fatigue life of samples prepared with good and low quality aggregate is developed in Figure 20, from the data presented in Table 17. Test results of as-compacted samples show that good quality aggregate samples perform better than the samples prepared with low quality aggregate. Similar results were found with the conditioned samples (Figure 20b). The difference in fatigue life between good and low quality aggregate samples is, however, slight in both cases (Figure 20b). The influence of aggregate quality is therefore more important on the mix resilient modulus than on the mix fatigue life.

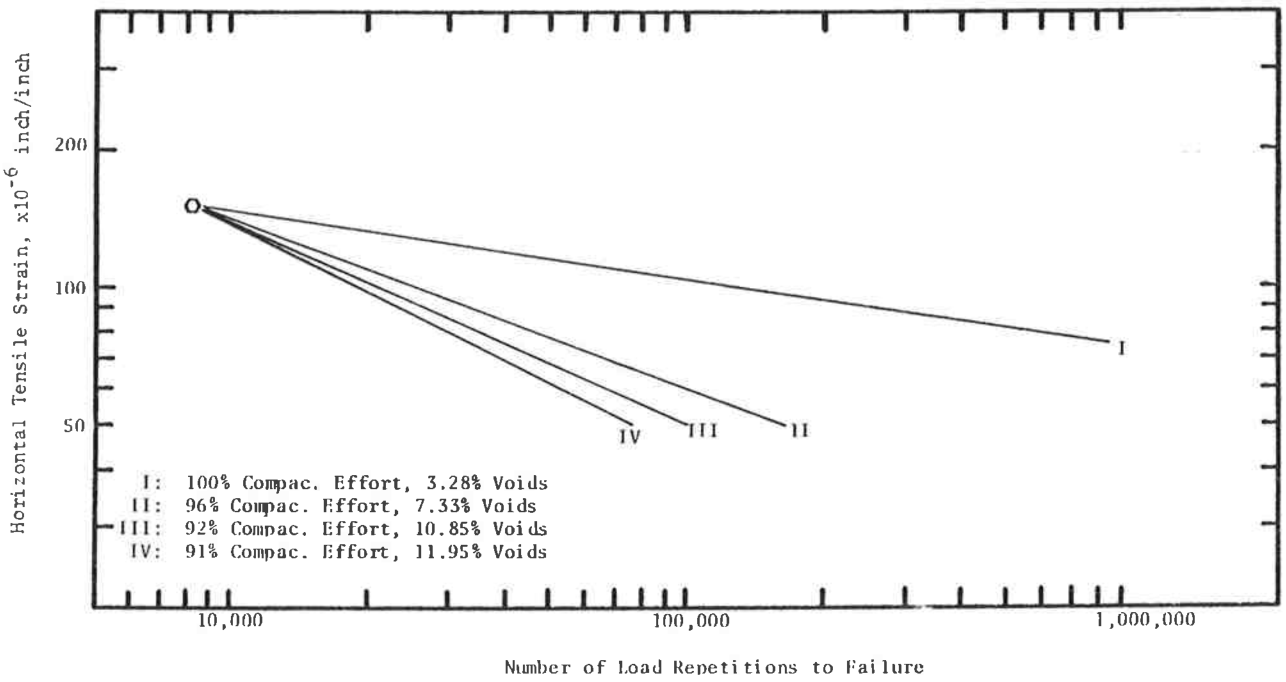


Figure 16a. Influence of Mix Density on Fatigue Life as Compacted Samples

6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt

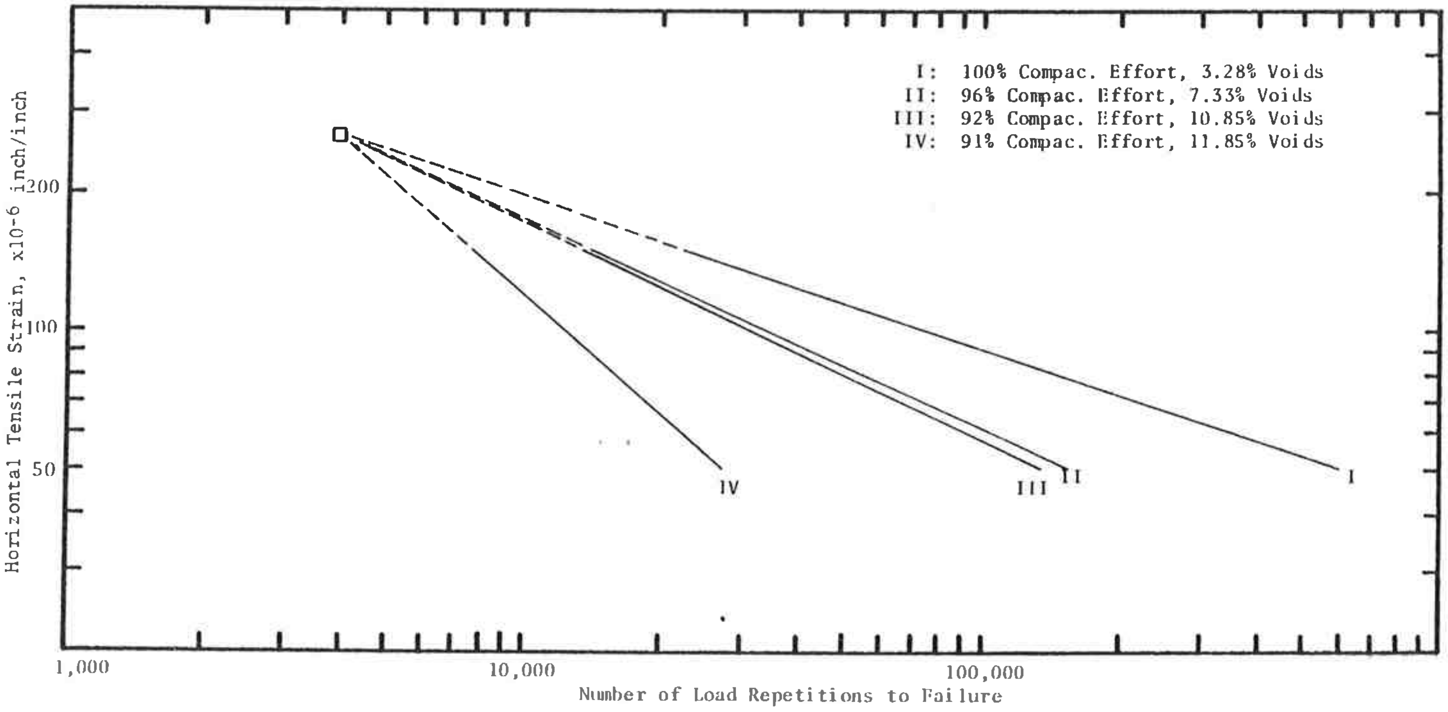


Figure 16b: Influence of Mix Density on Fatigue Life Conditioned Samples
6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt

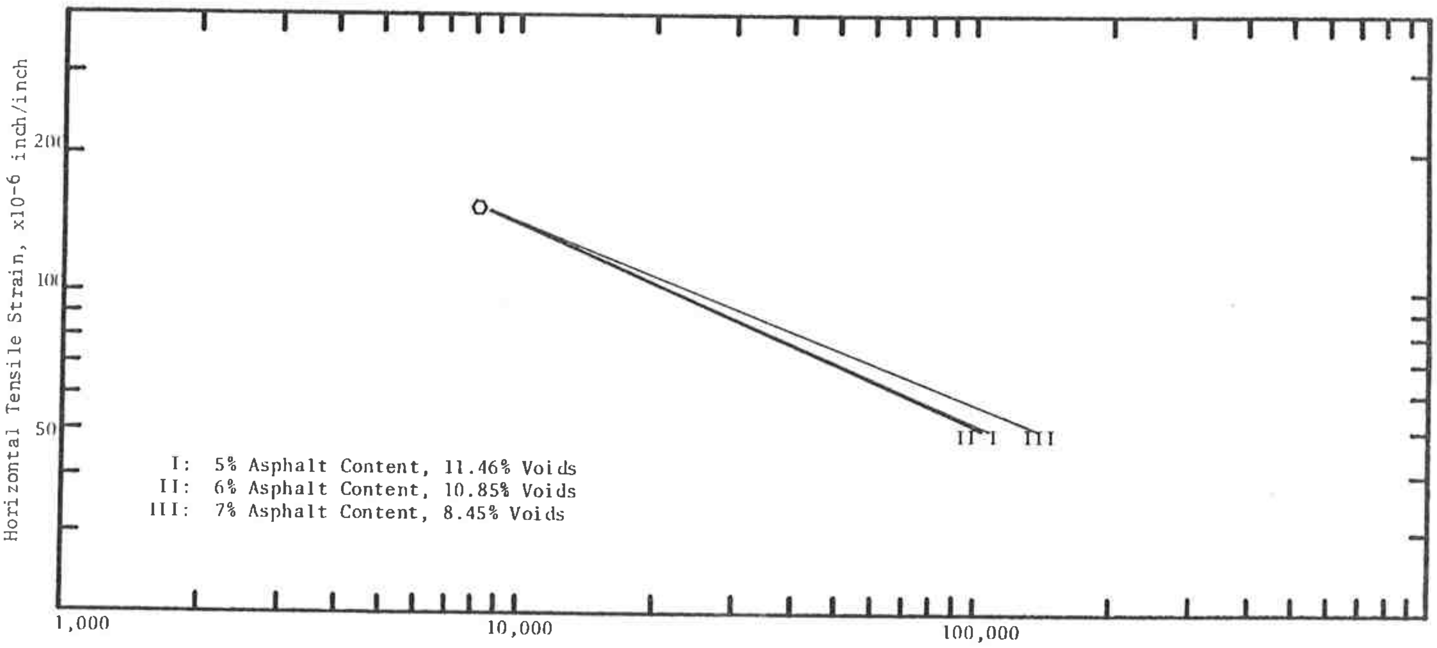


Figure 17a. Influence of Asphalt Content on Fatigue Life
As-Compacted Samples
6% Passing No. 200 - 25% Passing No. 10 - 92% Compaction

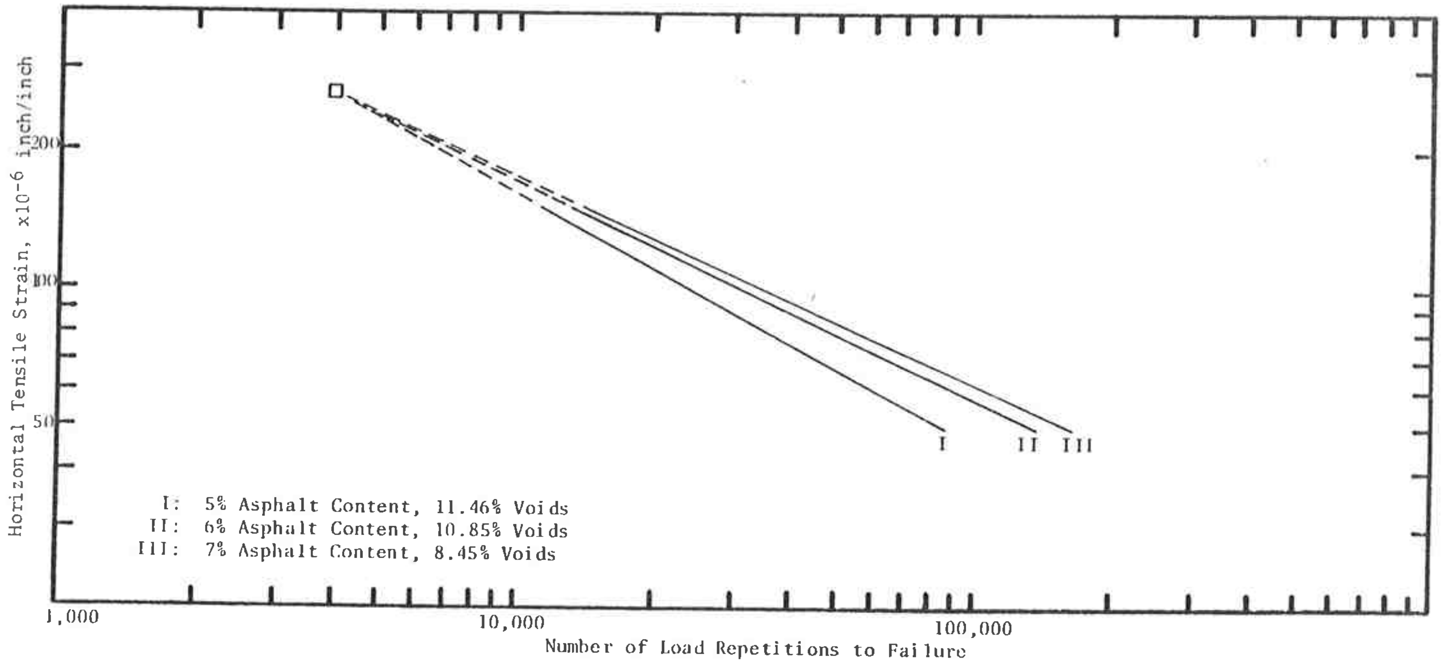


Figure 17b. Influence of Asphalt Content on Fatigue Life
Conditioned Samples
6% Passing No. 200 - 25% Passing No. 10 - 92% Compaction

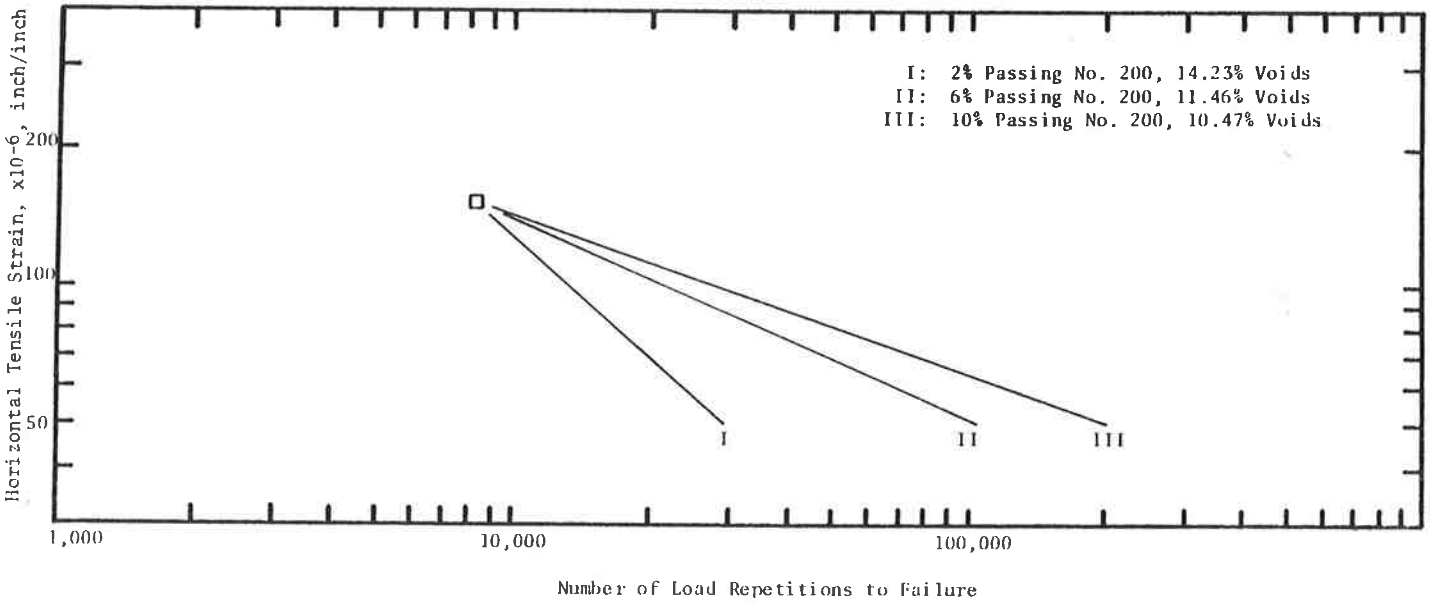


Figure 18a. Influence of Passing No. 200 on Fatigue Life
As-Compacted Samples
5% Asphalt Content - 92% Compaction

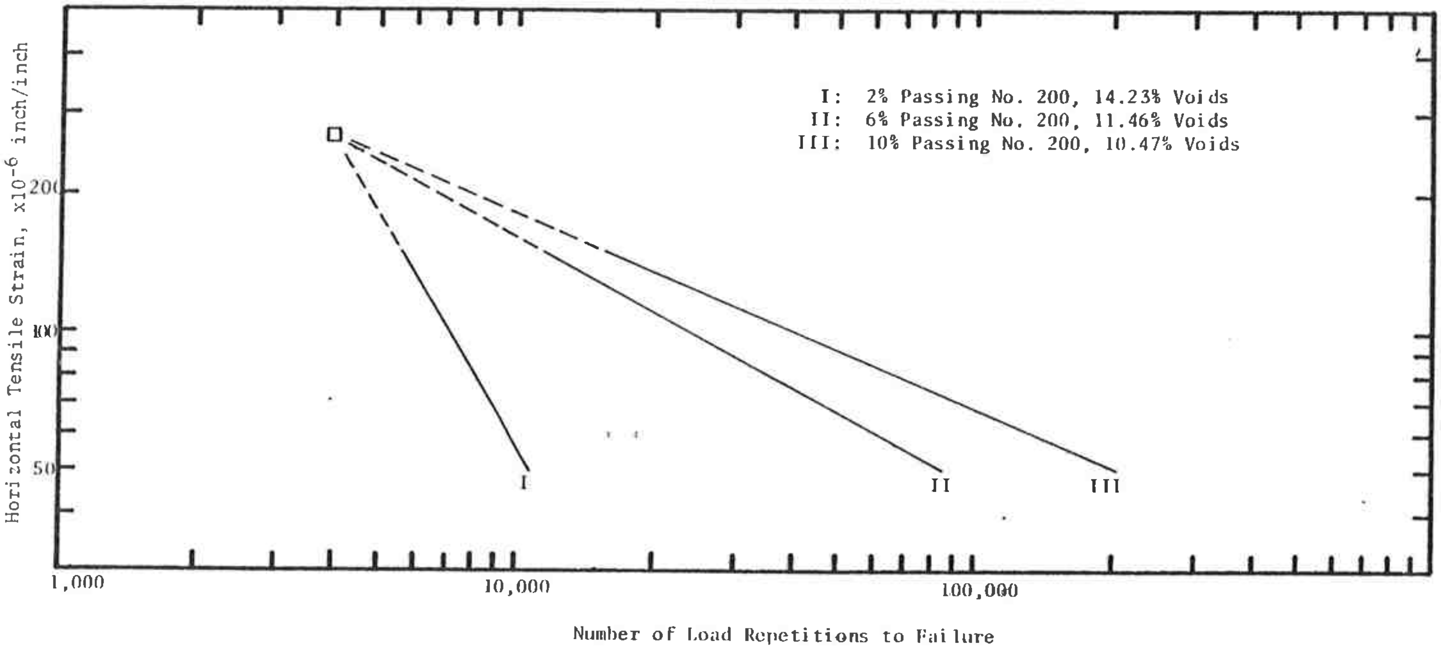


Figure 18b. Influence of Percent Passing No. 200 on Fatigue Life
Conditioned Samples
5% Asphalt Content - 92% Compaction

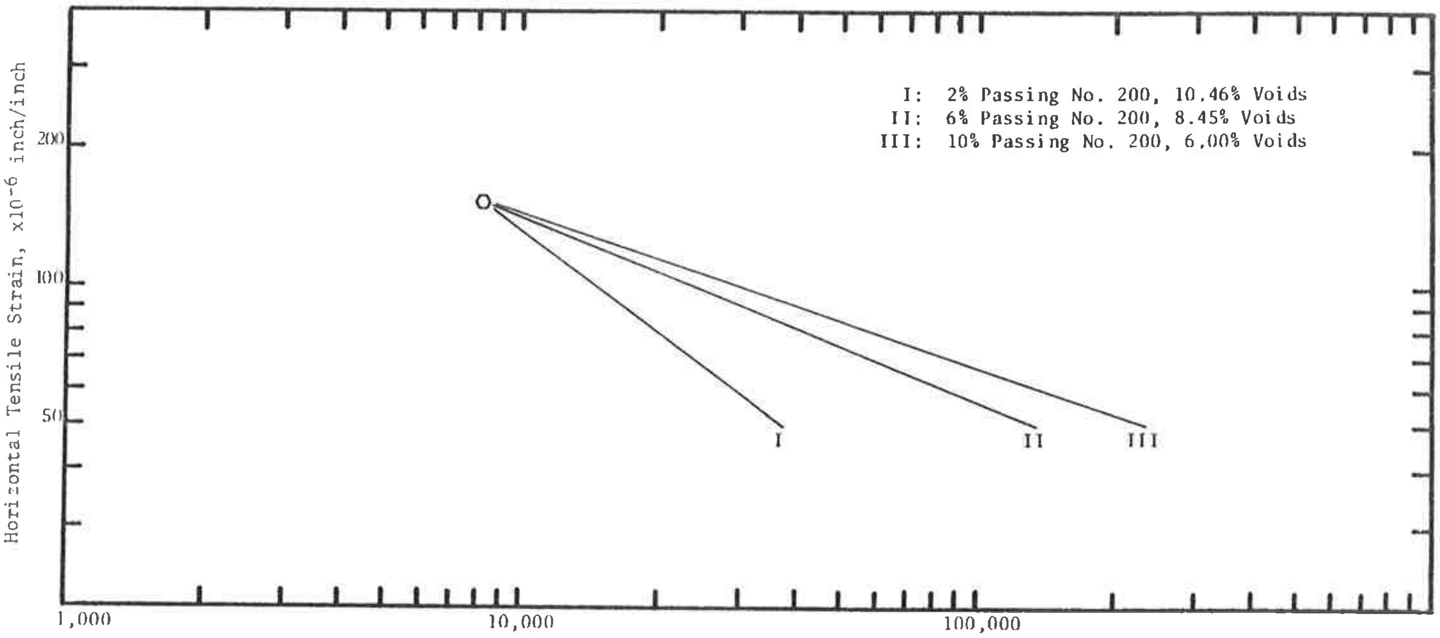


Figure 19a. Influence of Passing No. 200 on Fatigue Life
As-Compacted Samples

7% Asphalt Content - 92% Compaction

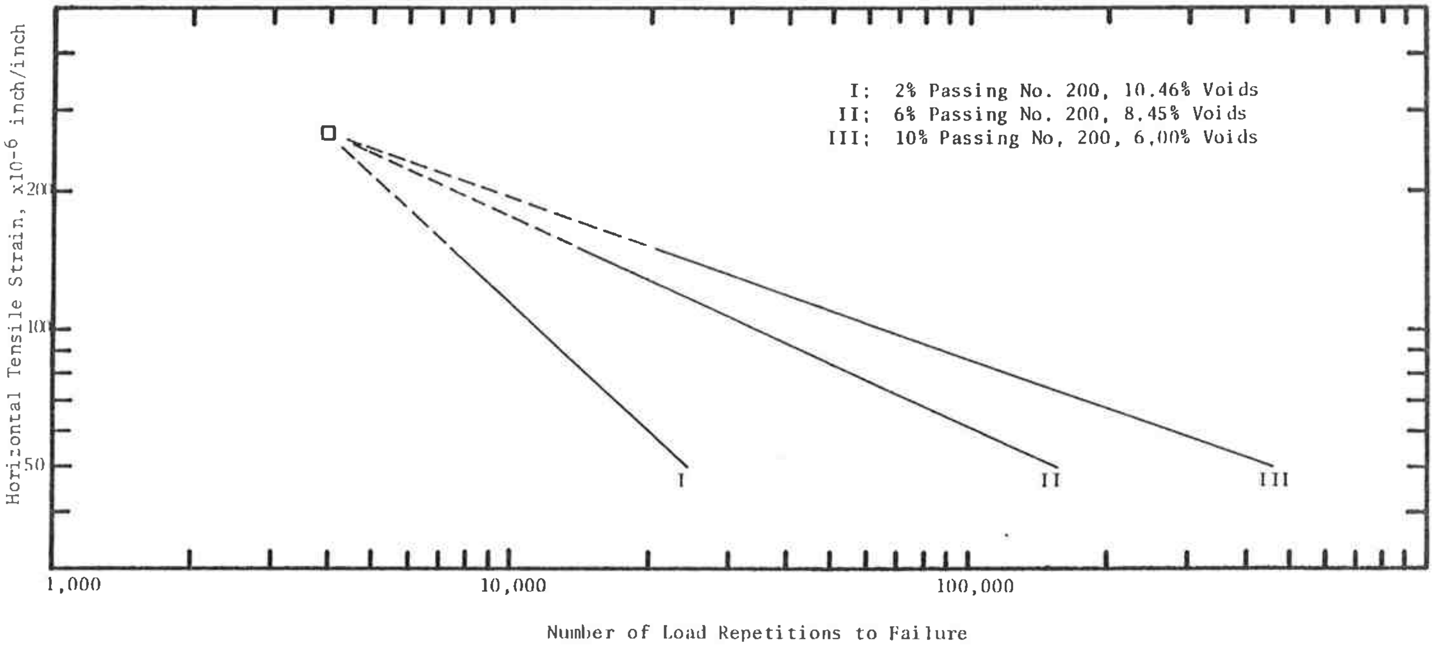


Figure 19b. Influence of Passing No. 200 on Fatigue Life
Conditioned Samples

7% Asphalt Content - 92% Compaction

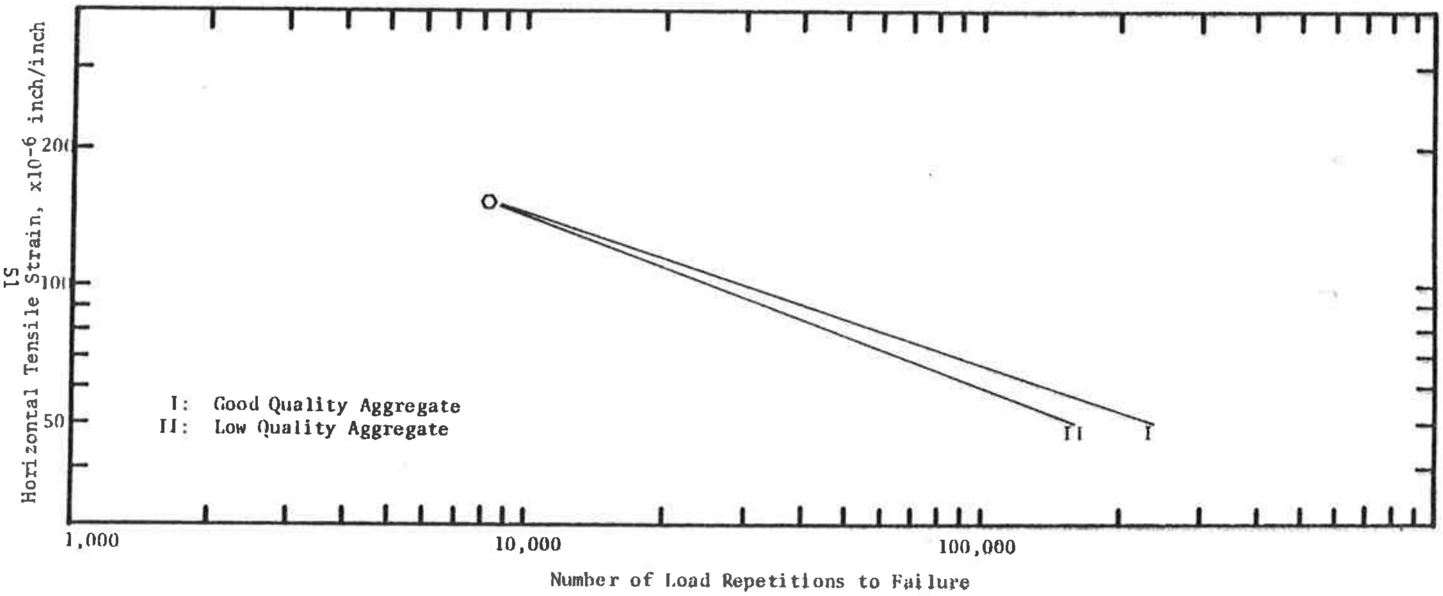


Figure 20a. Influence of Aggregate Quality on Fatigue Life
As-Compacted Samples

6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt Content - 96% Compaction

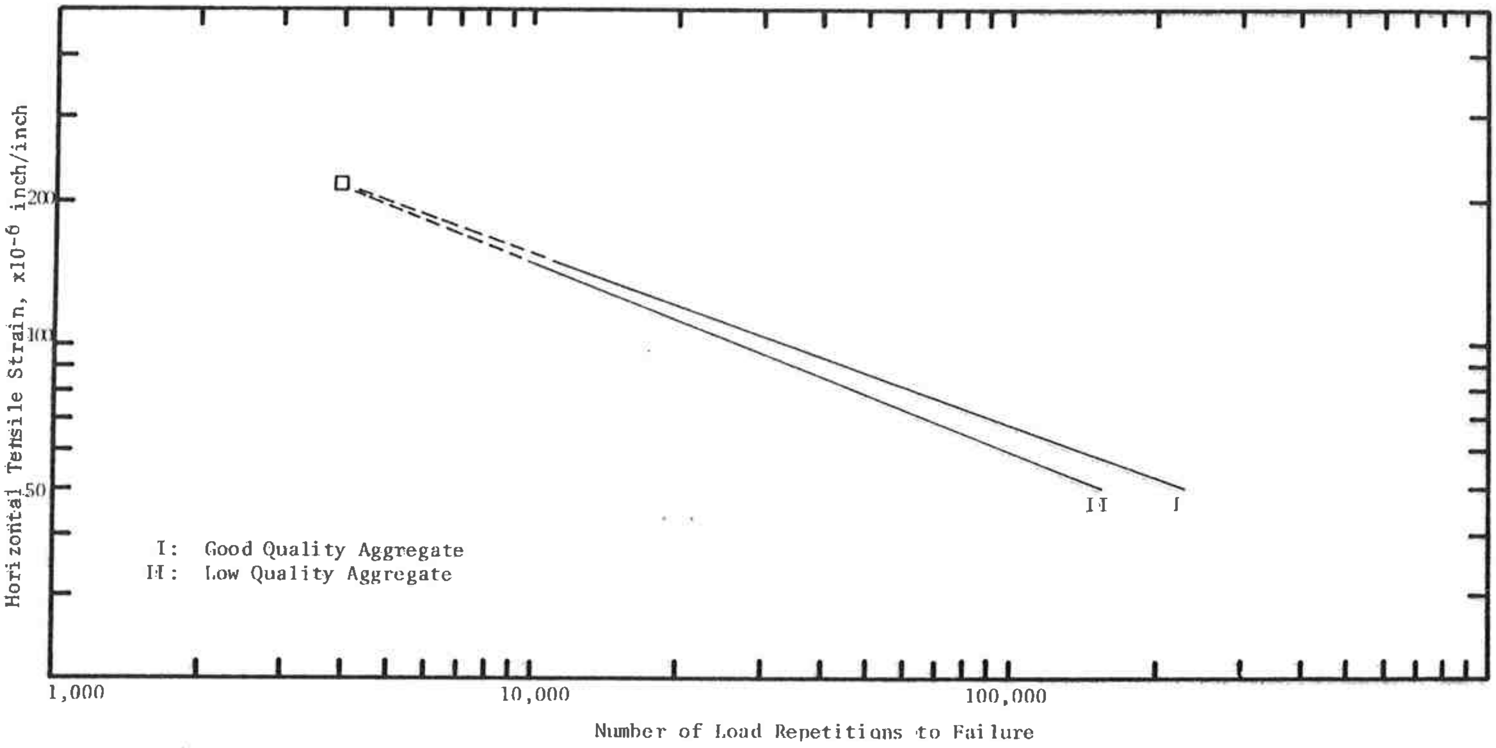


Figure 20b. Influence of Aggregate Quality on Fatigue Life
Conditioned Samples

6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt Content - 97% Compaction

Table 17. Influence of Aggregate Quality on Fatigue Life 6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt Content - 96% Compaction (corrected data)

Aggregate Quality	Fatigue Life (N_F)*		Bulk Specific Gravity
	As-Compacted	Conditioned	
Good	$N_f = 2.43 \times 10^{-8} (E_t)^{-3.02}$	$N_f = 1.11 \times 10^{-5} (E_t)^{-2.40}$	2.35
Low	$N_f = 3.36 \times 10^{-7} (E_t)^{-2.72}$	$N_f = 6.67 \times 10^{-5} (E_t)^{-2.18}$	2.31

E_t: Horizontal Tensile Strain

*: Corrected Data

Table 18. Influence of Pavebond on Fatigue Life (corrected data)

Type of Mix	Fatigue Data	
	As-Compacted	Conditioned
Standard*	$N_f = 3.36 \times 10^{-7} (E_t)^{-2.72}$	$N_f = 6.67 \times 10^{-5} (E_t)^{-2.18}$
Standard with Pavebond Added During Mixing	$N_f = 6.58 \times 10^{-6} (E_t)^{-2.38}$	$N_f = 1.42 \times 10^{-4} (E_t)^{-2.09}$
Standard With Pavebond Added to Asphalt	$N_f = 7.87 \times 10^{-6} (E_t)^{-2.36}$	$N_f = 1.49 \times 10^{-6} (E_t)^{-2.64}$

*6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt - 97% Compaction

N_f: Number of Load Repetitions to Failure

E_t: Horizontal Tensile Strain

Results from fatigue tests run on samples prepared with .85% Pavebond are summarized in Table 18 and presented in Figure 21. The results indicate that the Pavebond does not affect the as-compacted mix fatigue life (Figure 21a). The conditioned results indicate a fatigue life unchanged when the Pavebond is added directly to the mix and an increase in mix life by a factor of 1.7 when the Pavebond is added to the asphalt (Figure 21b). This result corroborates the resilient modulus data which showed that the Pavebond affects positively the mix stiffness only when the Pavebond has been previously mixed with the asphalt.

Table 19 gives the results of the study on the percent passing No. 10. These results are illustrated on Figure 22. The as-compacted as well as the conditioned data clearly show an optimum mix fatigue life for 25% passing the No. 10 sieve.

Permanent Deformation Data

Vertical compressive permanent deformation was measured during fatigue testing using a dial gauge. All tests were performed at ambient temperatures; readings were taken until failure of the sample occurred. The vertical permanent strain was calculated from the vertical permanent deformation (4), according to:

$$\varepsilon_c = \mu_t \frac{-.03896\nu - .1185}{.0156\nu - .8954} \qquad (14)$$

where ε_c = Compressive permanent strain

μ_t = Total vertical deformation, inches

ν = Poisson's ratio

For a Poisson's ratio of .35, equation (10) becomes:

$$\varepsilon_c = \mu_t \times .1485 \qquad (15)$$

A typical plot of the compression permanent vertical strain (ε_c) versus number of load repetitions at different loads is shown in Figure 23 for the mix with 6% passing No. 200, 25% passing No. 10, 6% asphalt and 96% relative compaction. Each sample was tested at a different stress level, resulting in a different rate of permanent deformation for each sample. For each test, the relationship between vertical permanent strain and number of load repetitions appears to be linear on log-log scale (10). Using a power curve fit

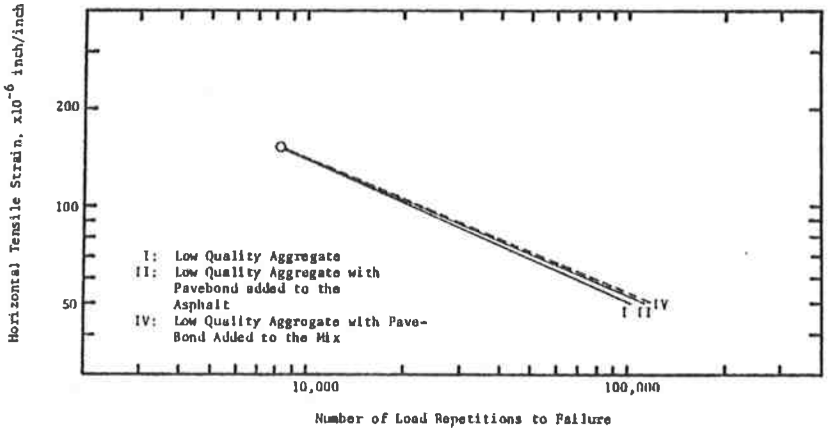


Figure 21a. Influence of Pavebond on Fatigue Life as Compacted Samples

6% Passing - No. 200 - 25% Passing No. 10
6% Asphalt - 96% Compaction

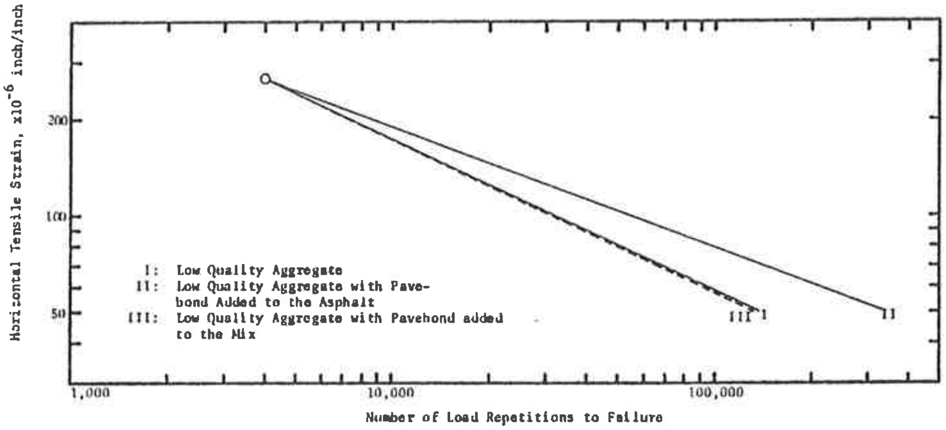


Figure 21b. Influence of Pavebond on Fatigue Life Conditioned Samples

6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt - 96% Compaction

Table 19. Influence of Percent Passing No. 10 on Fatigue Life (corrected data)

Type of Mix	Fatigue Data	
	As-Compacted	Conditioned
Standard*	$N_f = 1.57 \times 10^{-5}(E_t)^{-2.28}$	$N_f = 1.24 \times 10^{-4}(E_t)^{-2.10}$
Mix With 30% Passing No. 10	$N_f = 2.20 \times 10^{-3}(E_t)^{-1.72}$	$N_f = .828(E_t)^{-1.03}$
Mix With 35% Passing No. 10	$N_f = 2.46 \times 10^{-3}(E_t)^{-1.71}$	$N_f = 2.97 \times 10^{-2}(E_t)^{-1.44}$

*6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt - 92% Compaction

N_f: Number of Load Repetitions to Failure

E_t: Horizontal Tensile Strain

Table 20. Permanent Deformation Constants - as Compacted Samples 6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt Content - 96% Compaction

Sample	Initial Tensile Strain (inch/inch)	Permanent Deformation Constants		
		I	S	r^2
I	148.0×10^{-6}	2.56×10^{-4}	.499	.967
II	111.0×10^{-6}	7.89×10^{-5}	.568	.986
III	91.0×10^{-6}	2.93×10^{-4}	.360	.993
IV	138.0×10^{-6}	1.76×10^{-4}	.510	.980
V	144.0×10^{-6}	1.65×10^{-3}	.302	.990
VI	97.0×10^{-6}	2.22×10^{-6}	.214	.997

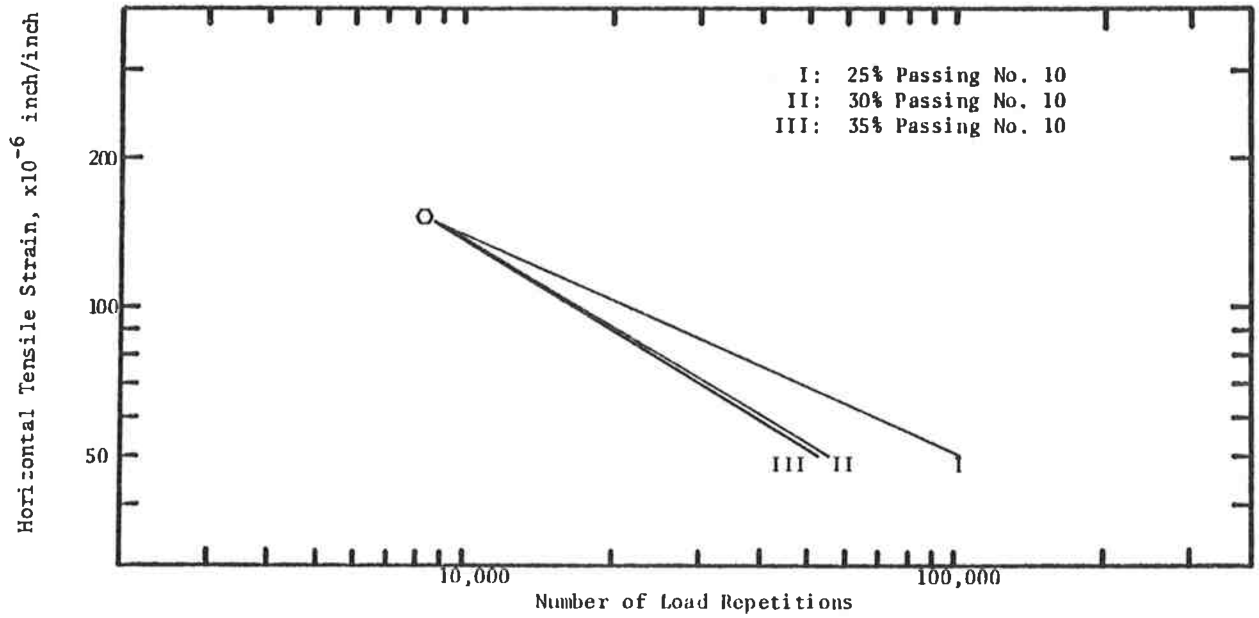


Figure 22a. Influence of Percent Passing No. 10 on Fatigue Life as Compacted Samples

6% Passing No. 200 - 6% Asphalt - 92% Compaction

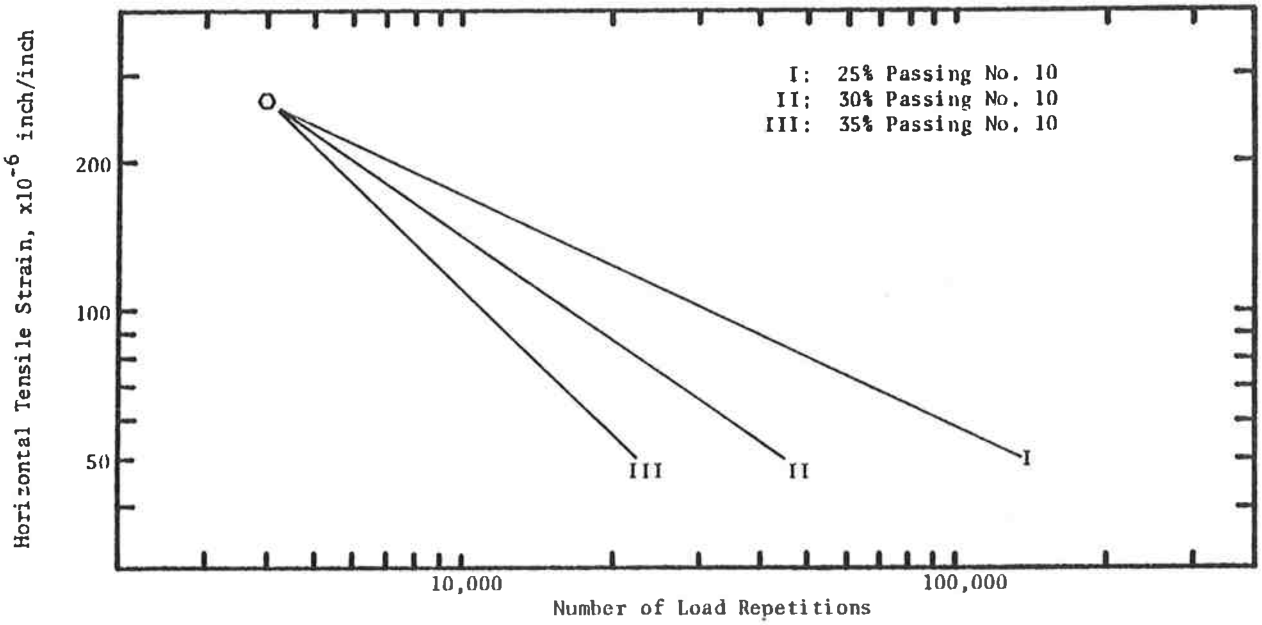


Figure 22b. Influence of Percent Passing No. 10 on Fatigue Life Conditioned Samples

6% Passing No. 200 - 6% Asphalt - 92% Compaction

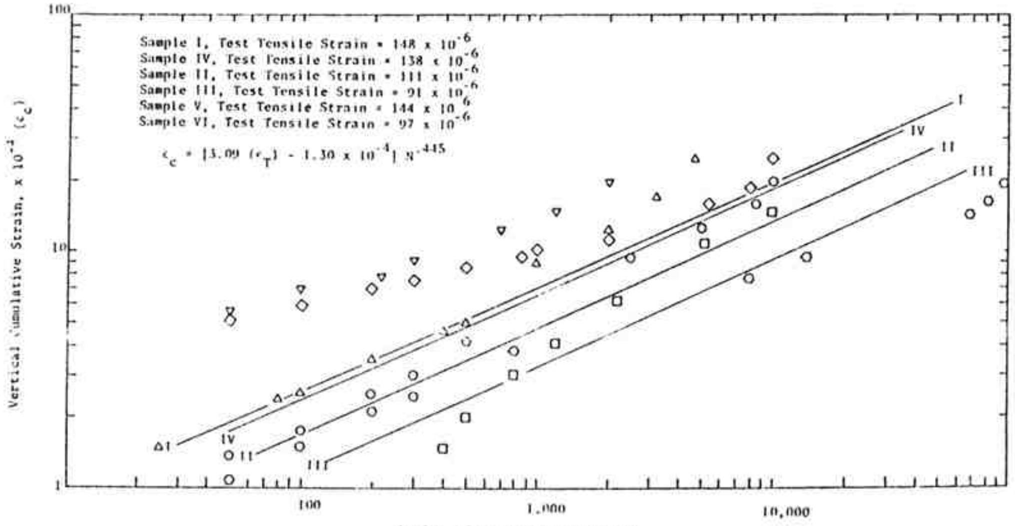


Figure 23. Permanent Deformation Results for Six Samples.

program, it is possible to express the vertical permanent deformation as a function of N:

$$\varepsilon_c = I(N)^s \qquad (16)$$

where ε_c = Compressive permanent vertical strain
I,s = Regression constants
N = Number of load repetitions

Constants I and s, computed from the test results, are presented in Table 20. Nonconsistent values (early readings and readings close to sample failure) were not included in the linear regression.

A recent study showed that the slope s in equation (12) is constant for a specific mix, whereas the intercept I is a function of the mix tensile strain ε_t (10). Table 20 presents the intercept (I) and slope (s) computed for 6 samples tested at different levels of tensile strain. All samples were prepared in the same conditions: 6% passing the No. 200 sieve, 25% passing the No. 10 sieve, 6% asphalt content and 96% compaction. A direct plot of the I values shown in Table 20 versus the corresponding tensile strain indicates no correlation (Figure 24), which can be explained by the nonuniformity of the computed slope s (Table 20). The linear regressions were therefore rerun using a fixed value for the slope equal to the average of the slope values shown in Table 20: 0.445. Plotted in Figure 25, the new relationship I versus ε_t appears more consistent and linear, if samples V and VI are not included. Using the data (ε_t and I with constant slope s), a linear regression was run, giving the following equation:

$$I = 1.30 \times 10^{-4} + 3.09\ (\varepsilon_t) \qquad (17)$$

The samples prepared with 6% passing No. 200, 25% passing No. 10, 6% asphalt and compacted at 96% are then characterized by a permanent deformation expressed as follows:

$$\varepsilon_c = [3.09 \times (\varepsilon_t) + 1.30 \times 10^{-4}]\ (N)^{.445} \qquad (18)$$

where ε_c = Compressive permanent strain

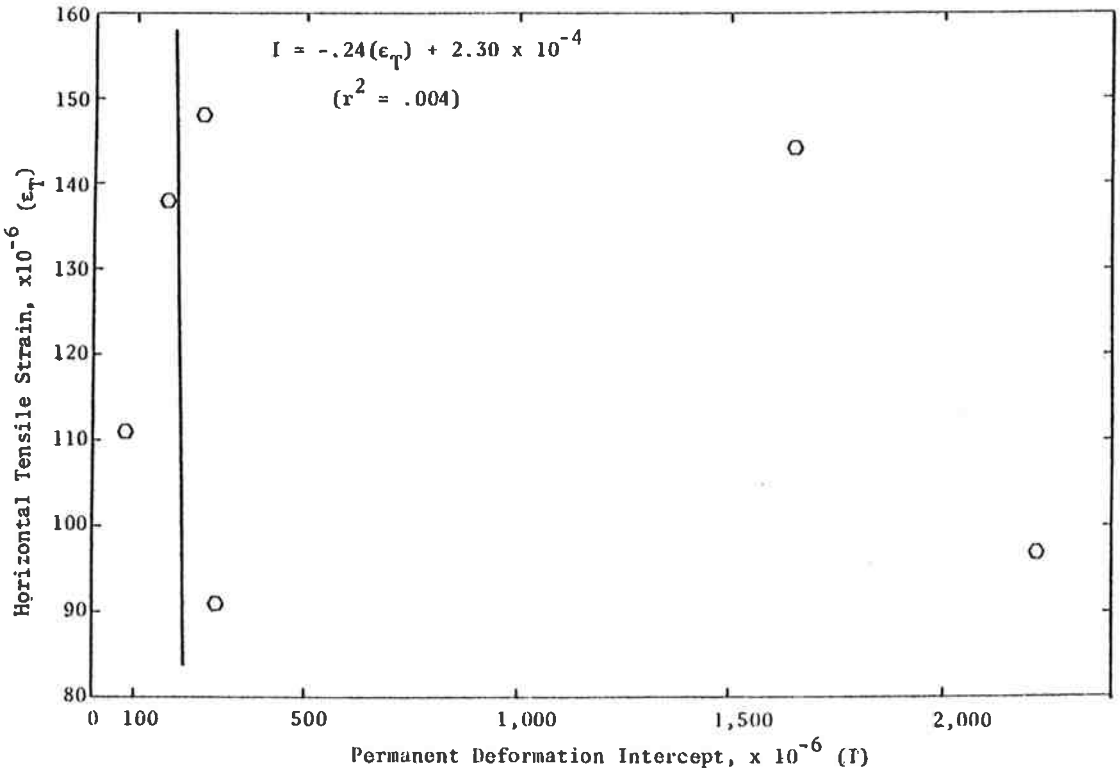


Figure 24. Relationship Between Permanent Deformation Intercept and Horizontal Tensile Strain.

6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt Content - 96% Compaction

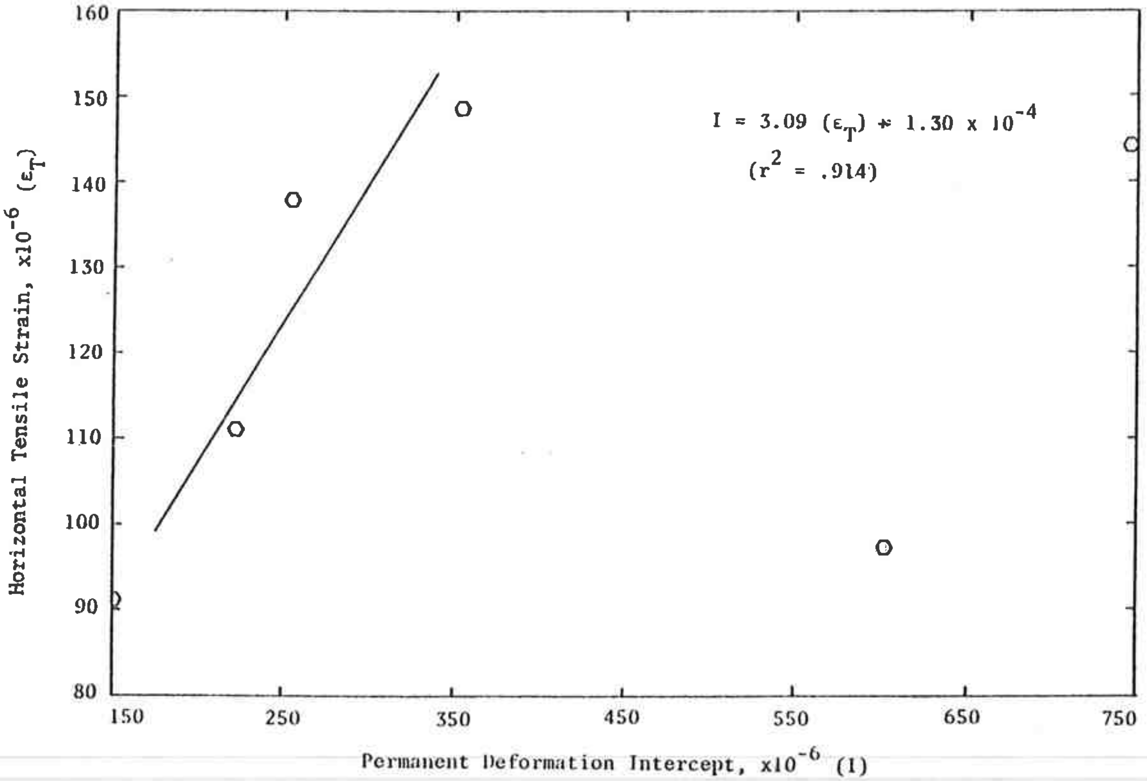


Figure 25. Relationship between permanent Deformation Intercept and Horizontal Tensile Strain for a Constant Slope = .445

6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt Content - 96% Compaction

ε_t = Horizontal tensile strain

N = Number of load repetitions

The same approach was used for all samples. The results are shown in Table 21a for the as-compacted samples and Table 21b for the conditioned samples.

A comparison between results for different mixes was accomplished by setting the mix tensile strain at 100 microstrain and plotting on log-log scale the permanent compressive strain as a function of the number of load repetitions. Figure 26a (as-compacted samples) and 26b (conditioned samples) show the influence of mix density on permanent deformation. As expected, low density asphalt concrete mixes are more susceptible to permanent deformation than the dense mixes. Sample conditioning affects especially the low density samples and therefore emphasizes strongly the difference between the dense samples (100% compaction) and the poorly compacted samples (91-92% compaction). The influence of asphalt content on the mix permanent deformation is illustrated in Figures 27a and 27b for samples prepared with 6% passing No. 200. According to these results, an asphalt concrete mix would be less susceptible to permanent deformation at high (7%) and low (5%) asphalt content than at the design optimum, 6%; but the 7% asphalt content mix also had the lowest percent air voids. When comparing the results of the 5% and the 7% asphalt content samples (Figures 28a and 28b) it appears that increasing the asphalt content decreases the mix susceptibility to permanent deformation, for both the as-compacted and the conditioned samples. The conditioning process shows the importance of the complex asphalt-fines interaction. Before conditioning (Figure 28a), the results for the 2 and 10% passing No. 200 sieve are very similar, indicating that asphalt content is a controlling factor for the as-compacted samples. After conditioning (Figure 28b), the permanent deformation curve for the 10% passing No. 200 shows little change, whereas the permanent deformation curve for the 2% passing No. 200 sieve indicates an increased susceptibility of this mix to permanent deformation. The particular importance of the asphalt content for low fines content is illustrated in Figures 29 and 30. At 5% asphalt content, conditioning affected slightly the 10% and 6% passing No. 200 samples (Figures 29a and 29b) and resulted in a substantial shift of the 2% passing No. 200 samples. At 7% asphalt content (Figures 30a and 30b), conditioning did not dramatically affect the results, which remained almost unchanged after conditioning. In all cases, it is interesting to note that minimum permanent deformation was recorded at 6%

Table 21a. Permanent Deformation Data- As Compacted Samples

I = Intercept S = Slope ϵ_T = Mix Tensile Strain

			LEVEL OF COMPACTION			
			Second Compaction	First Compaction	95 Blows at 100 PSI 500 PSI Leveling Load	30 Blows at 100 PSI 300 PSI Leveling Load
10% Passing No. 200	Asphalt Content, %	7			$I=.831(\epsilon_T)+1.42\times10^{-4}$ $S=.462$	
		6				
		5			$I=.817(\epsilon_T)+1.21\times10^{-4}$ $S=.500$	
6% Passing No. 200	Asphalt Content, %	7			$I=3.50(\epsilon_T)+1.93\times10^{-4}$ $S=.374$	
		6	$I=6.29(\epsilon_T)-2.33\times10^{-4}$ $S=.291$	$I=3.09(\epsilon_T)-1.30\times10^{-4}$ $S=.445$	$I=2.60(\epsilon_T)-2.77\times10^{-5}$ $S=.506$	$I=2.71(\epsilon_T)+6.05\times10^{-5}$ $S=.491$
		5			$I=2.41(\epsilon_T)+1.18\times10^{-5}$ $S=.460$	
2% Passing No. 200	Asphalt Content, %	7			$I=1.46(\epsilon_T)+1.17\times10^{-4}$ $S=.501$	
		6				
		5			$I=3.12(\epsilon_T)+3.25\times10^{-5}$ $S=.446$	

Table 21b. Permanent Deformation Data - Conditioned Samples

I = Intercept S = Slope ϵ_T = Mix Tensile Strain

			LEVEL OF COMPACTION			
			Second Compaction	First Compaction	95 Blows at 100 PSI 500 PSI Leveling Load	30 Blow at 100 PSI 300 PSI Leveling Load
10% Passing No. 200	Asphalt Content, %	7			$I=5.33(\epsilon_T)-1.27\times10^{-6}$ $S=.392$	
		6				
		5			$I=1.59(\epsilon_T)+5.92\times10^{-5}$ $S=.497$	
6% Passing No. 200	Asphalt Content, %	7			$I=5.40(\epsilon_T)+4.61\times10^{-5}$ $S=.404$	
		6	$I=8.58(\epsilon_T)-5.34\times10^{-5}$ $S=.277$	Not Available $S=.420$	$I=2.22(\epsilon_T)-5.42\times10^{-5}$ $S=.610$	$I=2.03(\epsilon_T)-2.42\times10^{-4}$ $S=.492$
		5			$I=3.13(\epsilon_T)+2.06\times10^{-4}$ $S=.425$	
2% Passing No. 200	Asphalt Content, %	7			$I=2.12(\epsilon_T)+2.23\times10^{-4}$ $S=.481$	
		6				
		5			$I=.735(\epsilon_T)+1.64\times10^{-4}$ $S=.549$	

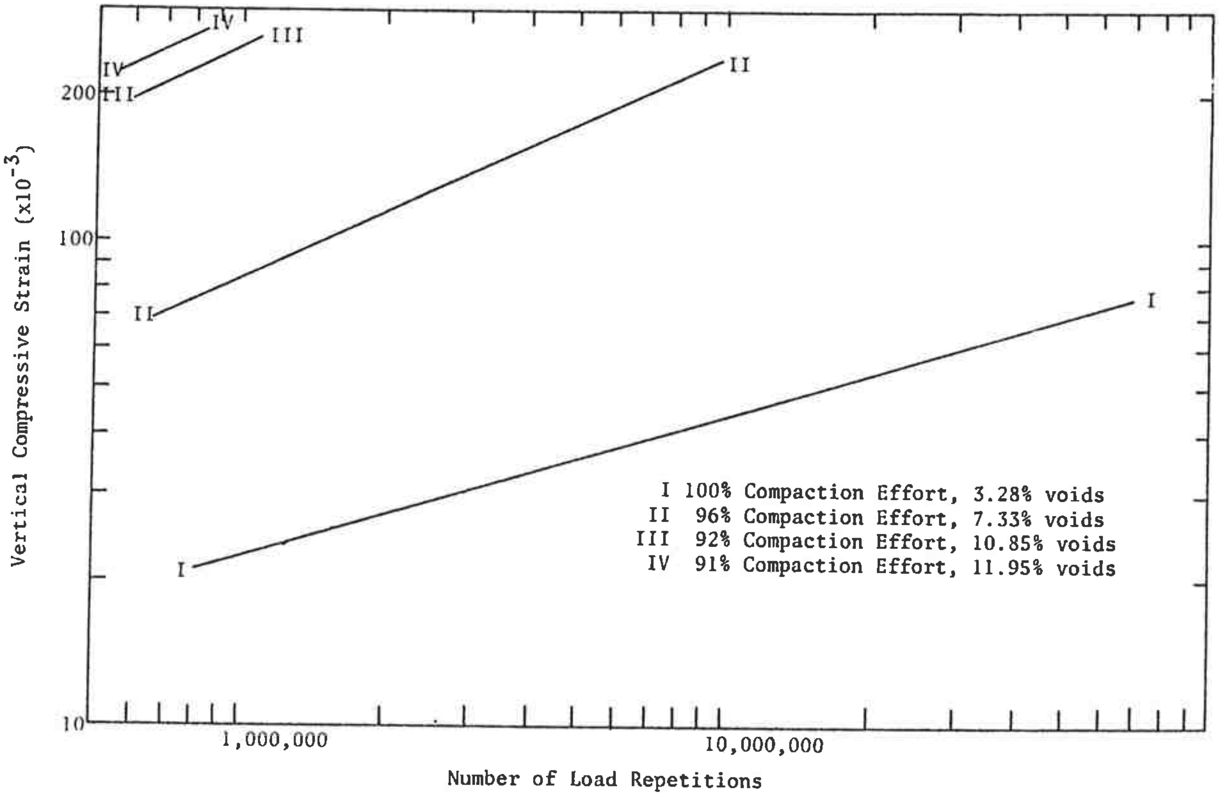


Figure 26a. Influence of Mix Density on Permanent Deformation-As Compacted Samples

6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt Content

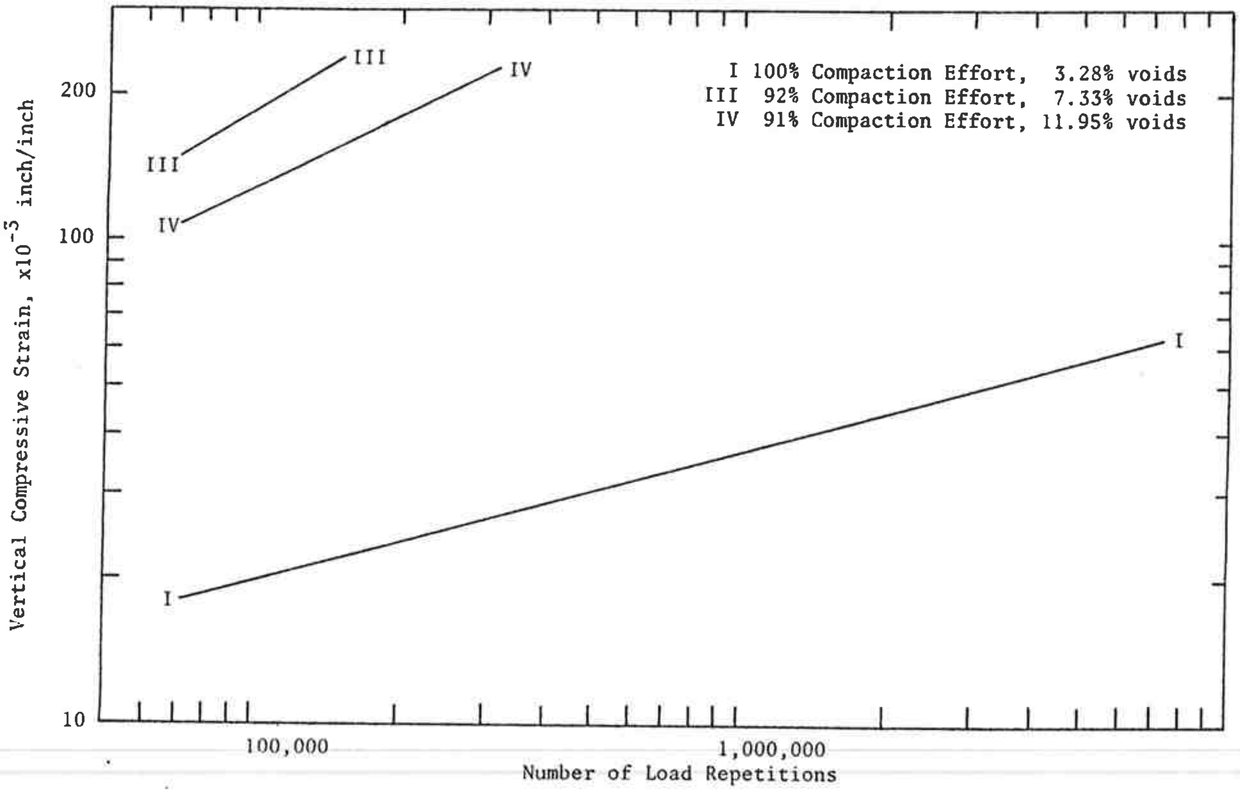


Figure 26b. Influence of Mix Density on Permanent Deformation-Conditioned Samples

6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt Content

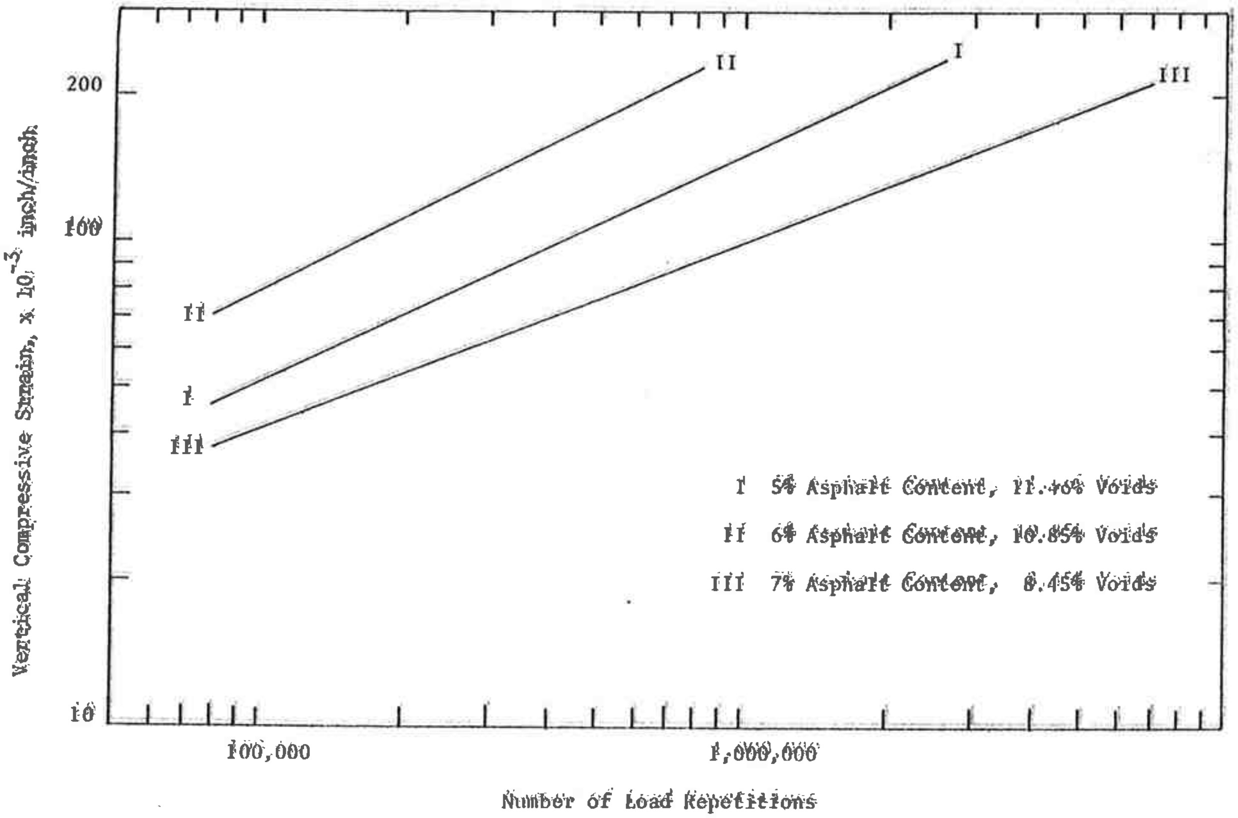


Figure 27a. Influence of Asphalt Content on Permanent Deformation - As Compacted Samples

6% Passing No. 200 - 25% Passing No. 10 - 92% Compaction

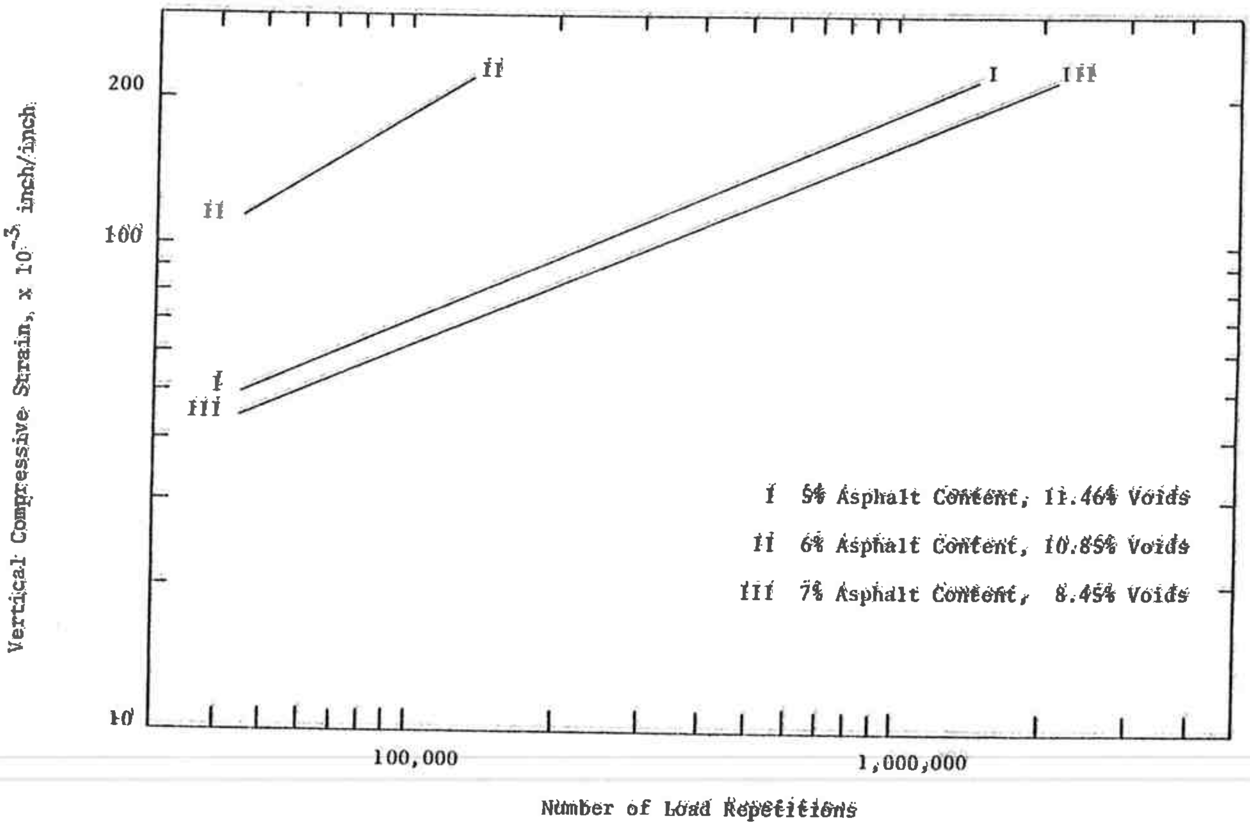


Figure 27b. Influence of Asphalt Content on Permanent Deformation-Conditioned Samples

6% Passing No. 200 = 25% Passing No. 10 = 92% Compaction

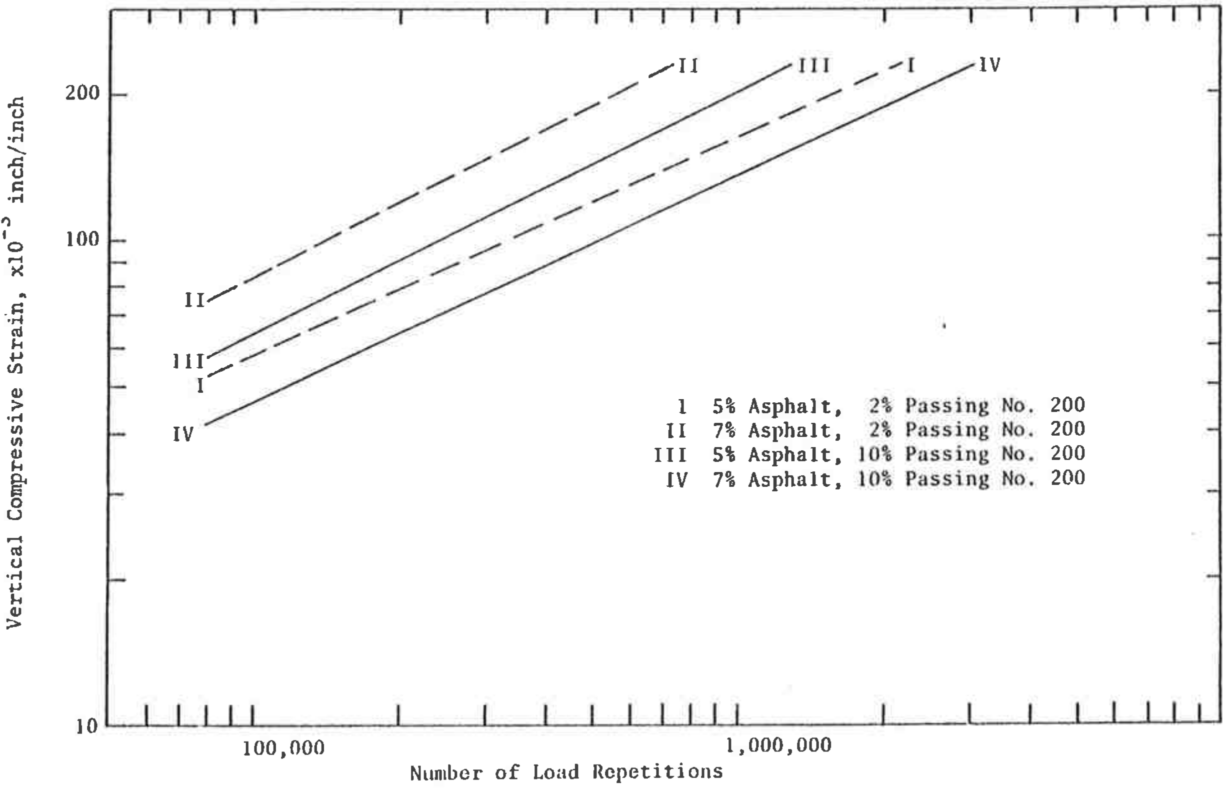


Figure 28a. Influence of Asphalt Content on Permanent Deformation – As Compacted Samples
25% passing No. 10 - 92% Compaction

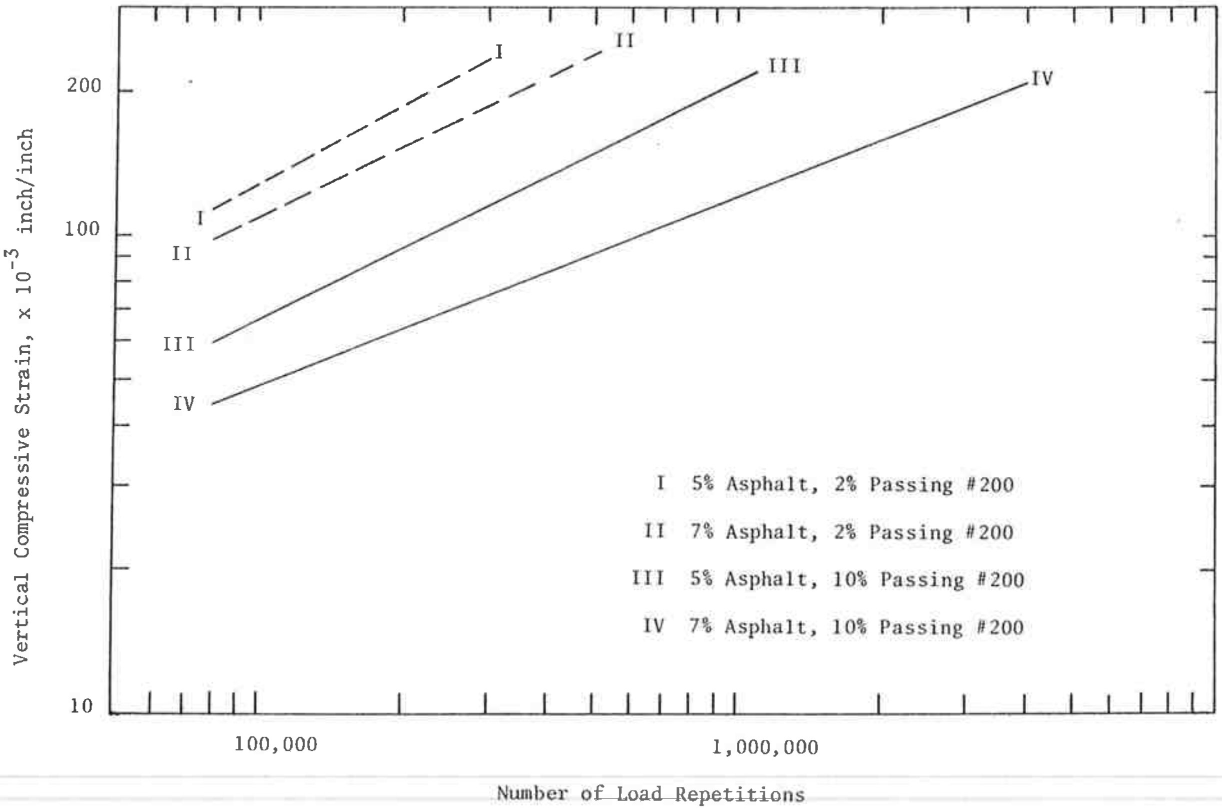


Figure 28b. Influence of Asphalt Content on Permanent Deformation – Conditioned Samples
25% Passing No. 10 - 92% Compaction

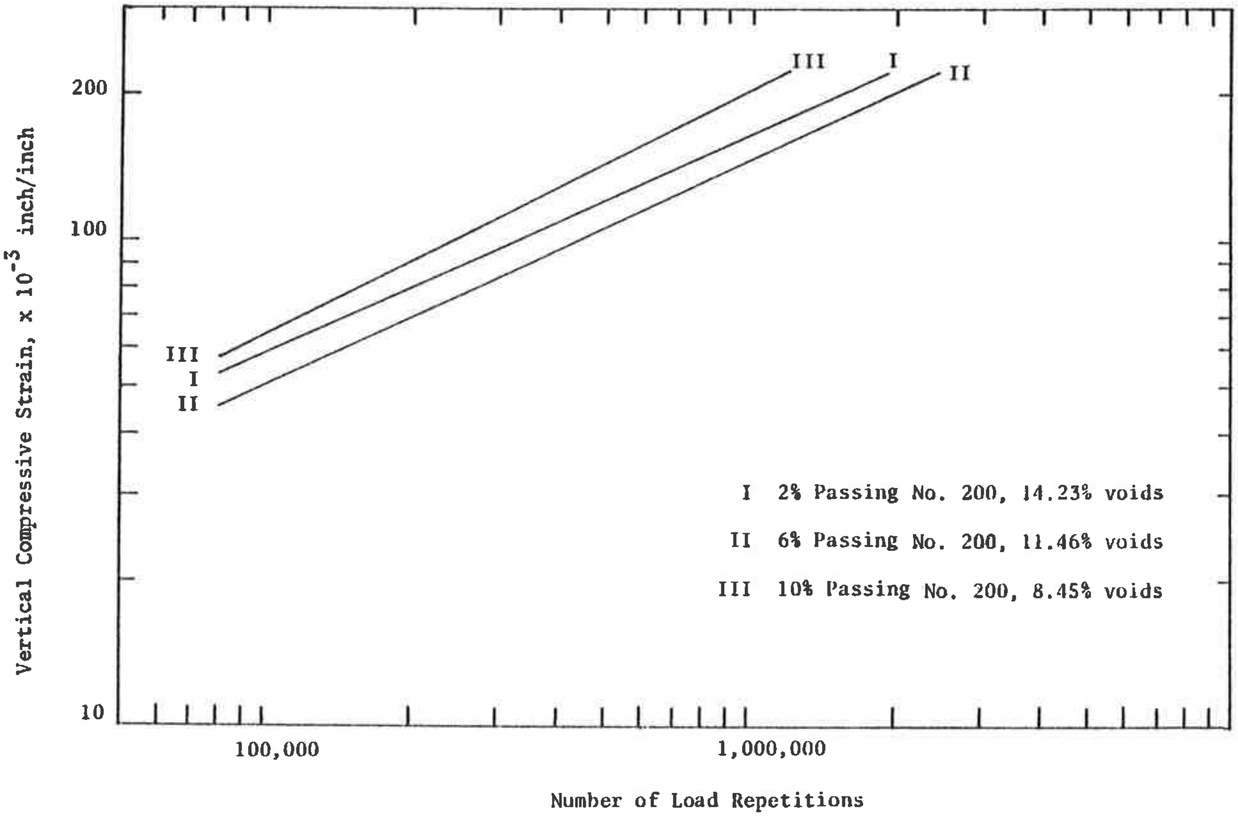


Figure 29a. Influence of Passing No. 200 on Permanent Deformation - As Compacted Samples

25% Passing No. 10 - 5% Asphalt Content - 92% Compaction

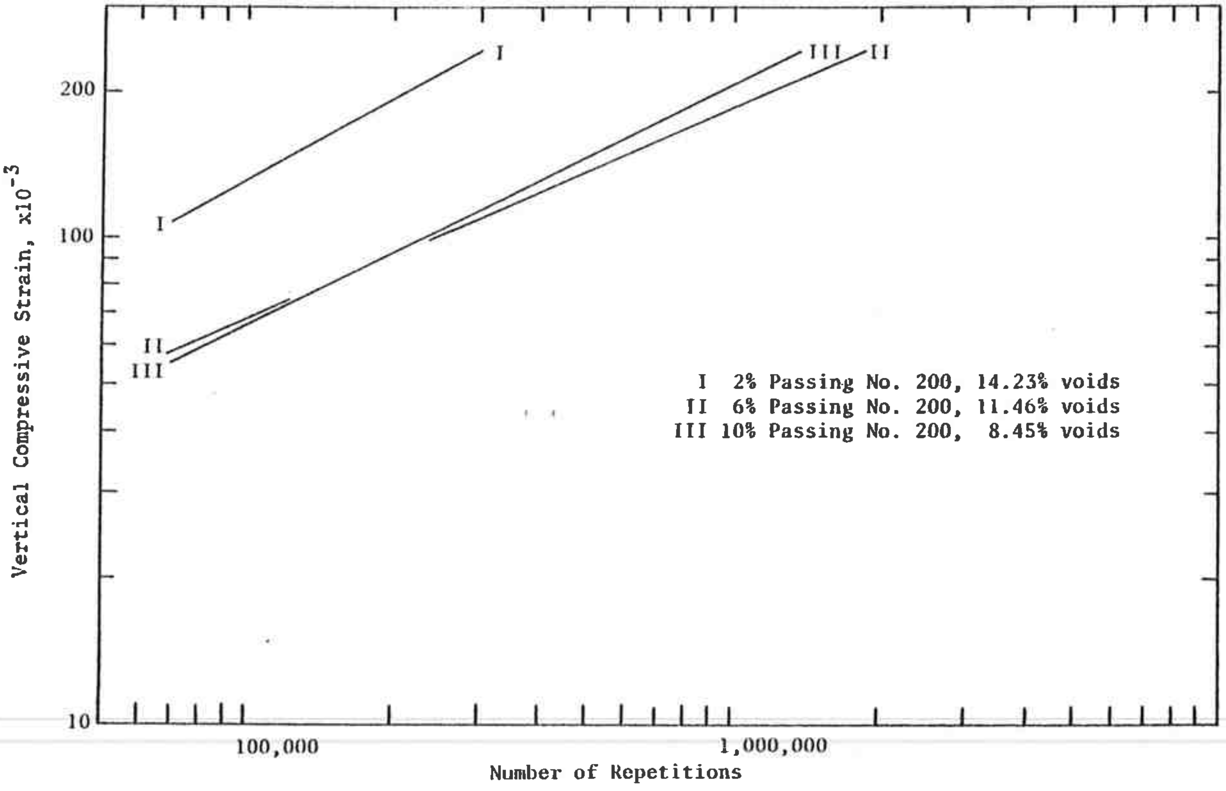


Figure 29b. Influence of Passing No. 200 on Permanent Deformation - Conditioned Samples

25% Passing No. 10 - 5% Asphalt - 92% Compaction

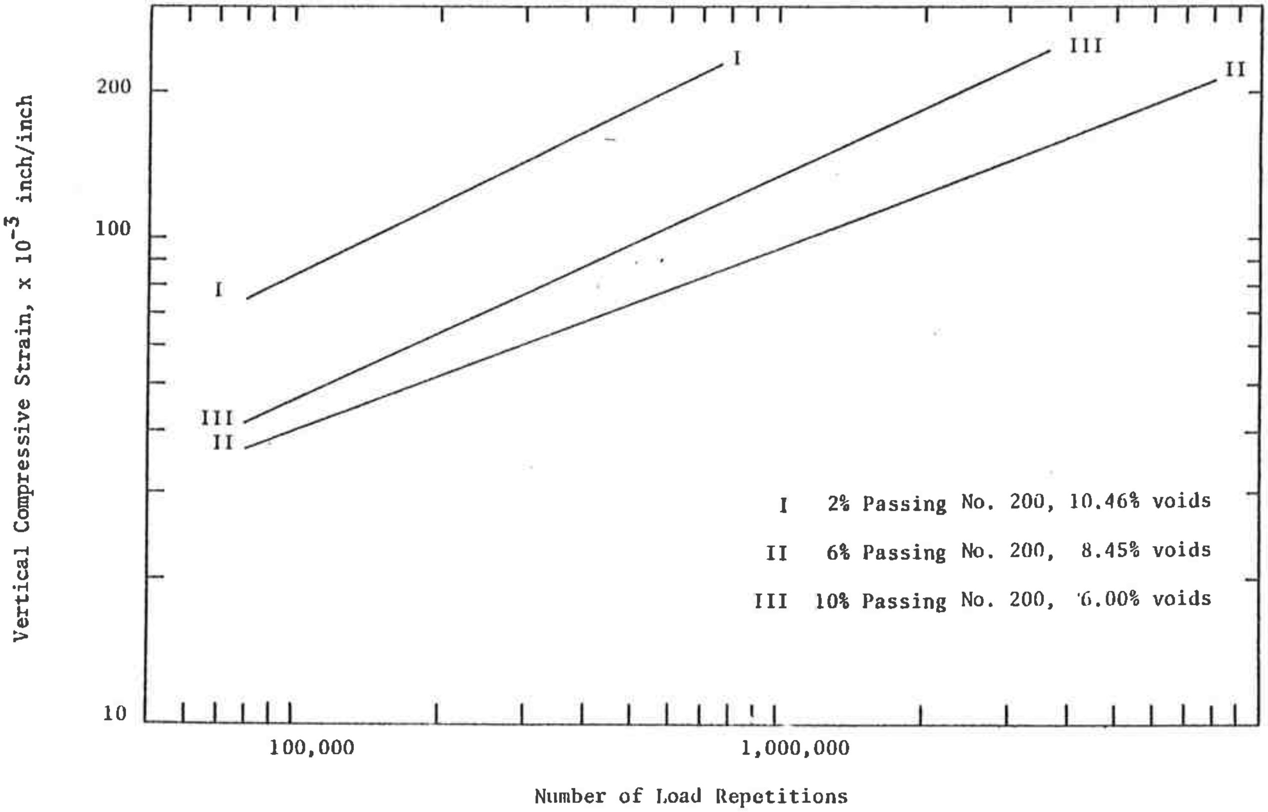


Figure 30a. Influence of Passing No. 200 on Permanent Deformation - As Compacted Samples

25% Passing No. 10 - 7% Asphalt Content - 92% Compaction

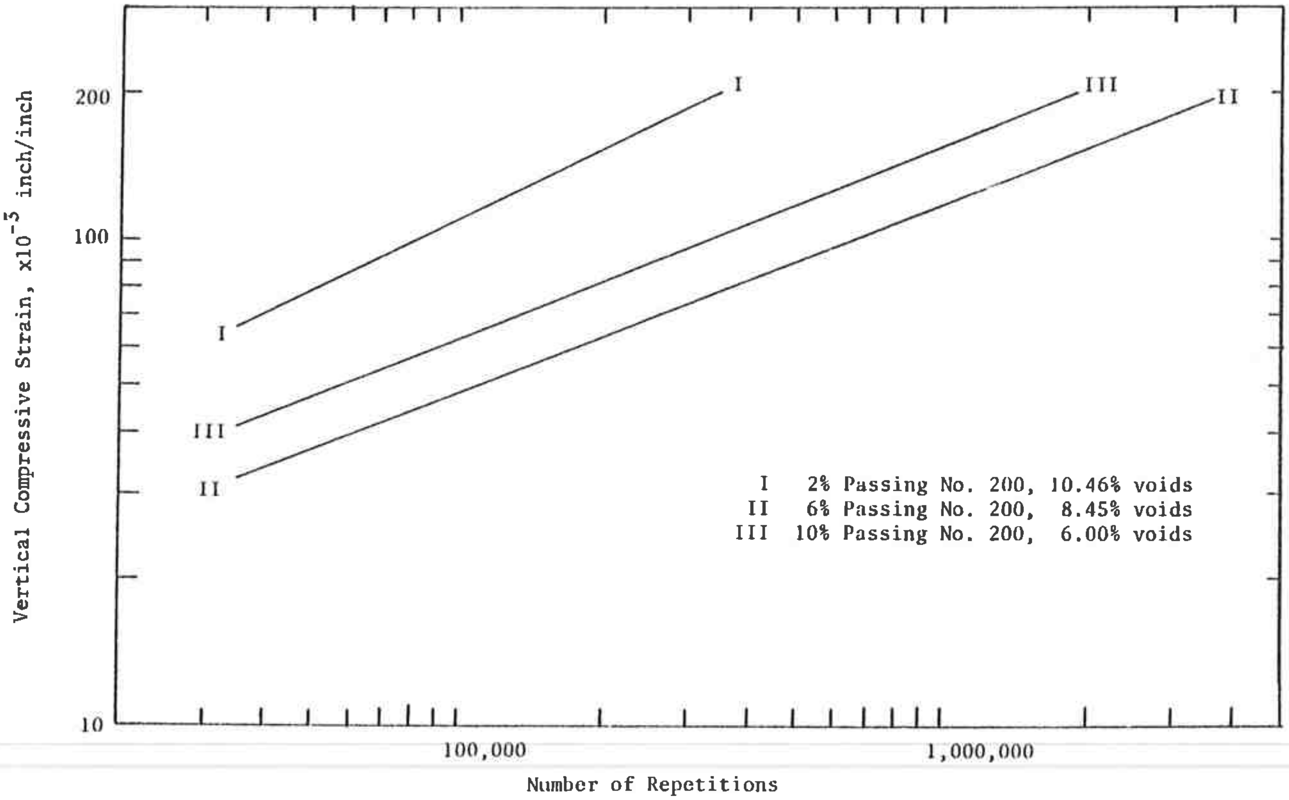


Figure 30b. Influence of Passing No. 200 on Permanent Deformation - Conditioned Samples

25% Passing No. 10 - 7% Asphalt Content - 92% Compaction passing No. 200 sieve.

Aggregate quality is also a factor to consider in the study of permanent deformation. Table 22 gives the permanent deformation parameters determined for the good quality aggregate samples, along with the results found for the low quality aggregate samples. It appears from Figure 31 that the two mixes performed in a similar way, although the good quality aggregate samples seem less susceptible to permanent deformation than the low quality aggregate samples. Aggregate quality, therefore, did not seem to be a controlling factor in the permanent deformation characteristics of this type of mix.

Permanent deformation characteristics of samples prepared with Pavebond added to the mix were also studied. These results are compared to the standard low quality and good quality mix performances in Table 23 and Figure 32. The as-compacted as well as the conditioned results indicate:

(1) The samples prepared with Pavebond performed better than the standard mixes.

(2) The Pavebond mix performance is not affected substantially by the method used to add the anti-strip agent to the mix.

(3) The low quality aggregate mix with Pavebond added to the mix performs slightly better than the good quality aggregate mix.

Finally, the influence of the amount passing the No. 10 sieve was checked. The results are presented in Table 24 and shown in Figure 33. As-compacted and conditioned results indicate that increasing the amount passing the No. 10 sieve decreases the mix sensitivity to permanent deformation.

In summary, the test results indicate that permanent deformation is reduced when:

(1) the mix density is increased

(2) the amount of fines is close to optimum (6% for the North Oakland-Sutherlin project)

(3) the asphalt content is above the design optimum (7% for the range of values used in this study).

Indirect Tensile Test Data

Table 9 shows that for each set of conditions considered in this study, 4 samples have been tested for indirect tensile strength. Two samples were tested as compacted, and two samples were tested after conditioning. Test

Table 22. Influence of Aggregate Quality on Permanent Deformation.

6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt Content - 96% Compaction

Aggregate Quality	Permanent Deformation Strain, ε_c	
	As Compacted	Conditioned
Good	$\varepsilon_c = [1.58(\varepsilon_T) + 2.34 \times 10^{-5}](N)^{.438}$	$\varepsilon_c = [2.47(\varepsilon_T) + 1.05 \times 10^{-4}](N)^{.429}$
Low	$\varepsilon_c = [3.09(\varepsilon_T) - 1.30 \times 10^{-4}](N)^{.445}$	Not Available

ε_T: Horizontal Tensile Strain

N : Number of Load Repetitions

Table 23. Influence of Pavebond on Permanent Deformation

Type of Mix	Permanent Deformation	
	As Compacted	Conditioned
Standard*	$\varepsilon_c = [3.09(\varepsilon_t)+1.30 \times 10^{-4}]N^{.445}$	Not Available
Standard With Pavebond Added During Mixing	$\varepsilon_c = [2.06(\varepsilon_t)-5.21 \times 10^{-6}]N^{.406}$	$\varepsilon_c = [4.21(\varepsilon_t)-7.67 \times 10^{-5}]N^{.416}$
Standard With Pavebond Added to Asphalt	$\varepsilon_c = [2.21(\varepsilon_t)+5.64 \times 10^{-6}]N^{.412}$	$\varepsilon_c = [3.40(\varepsilon_t)+5.53 \times 10^{-5}]N^{.401}$
Standard, Good Quality Aggregate	$\varepsilon_c = [1.58(\varepsilon_t)+2.34 \times 10^{-5}]N^{.438}$	$\varepsilon_c = [2.47(\varepsilon_t)+1.05 \times 10^{-4}]N^{.429}$

*Passing No. 200 - 25% Passing No. 10 - 6% Asphalt - 97% Compaction

ε_c: Vertical Compressive Strain

ε_t: Horizontal Tensile Strain

N: Number of Load Repetitions

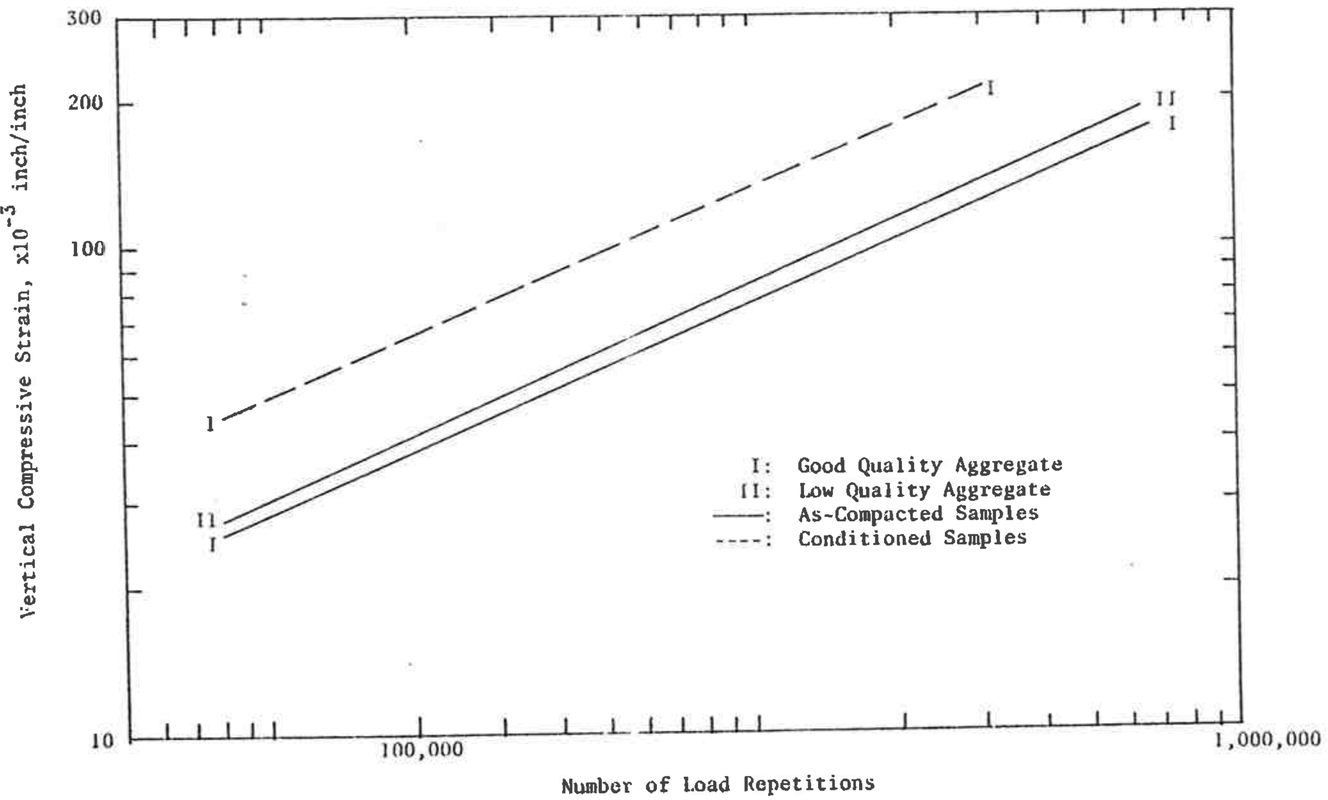


Figure 31. **Influence of Aggregate Quality on Permanent Deformation.**

6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt Content - 96% Compaction

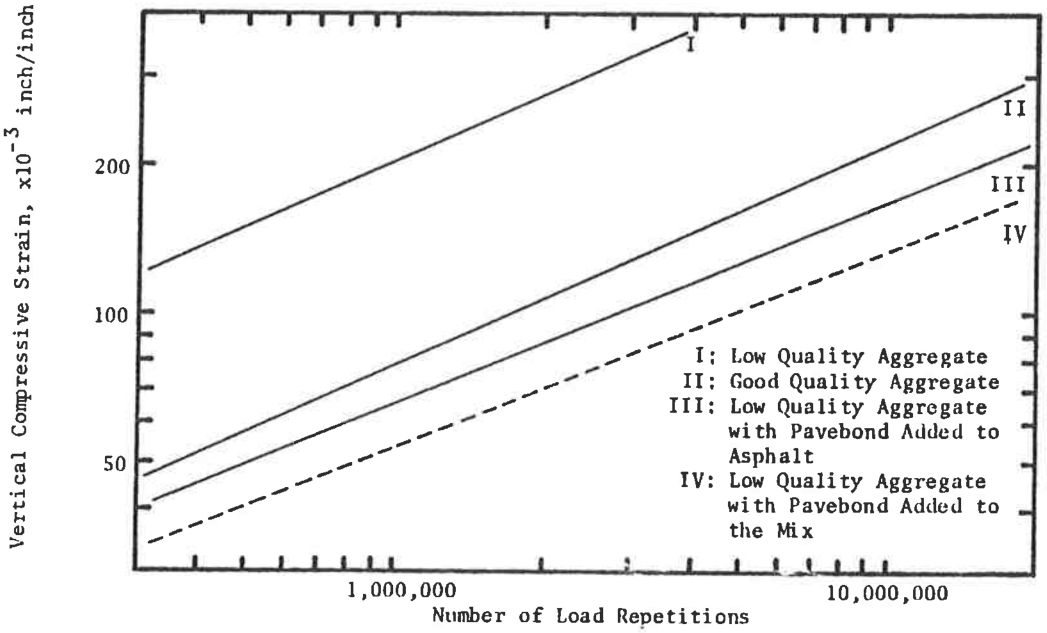


Figure 32a. Influence of Pavebond on Permanent Deformation as Compacted Samples

6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt - 96% Compaction

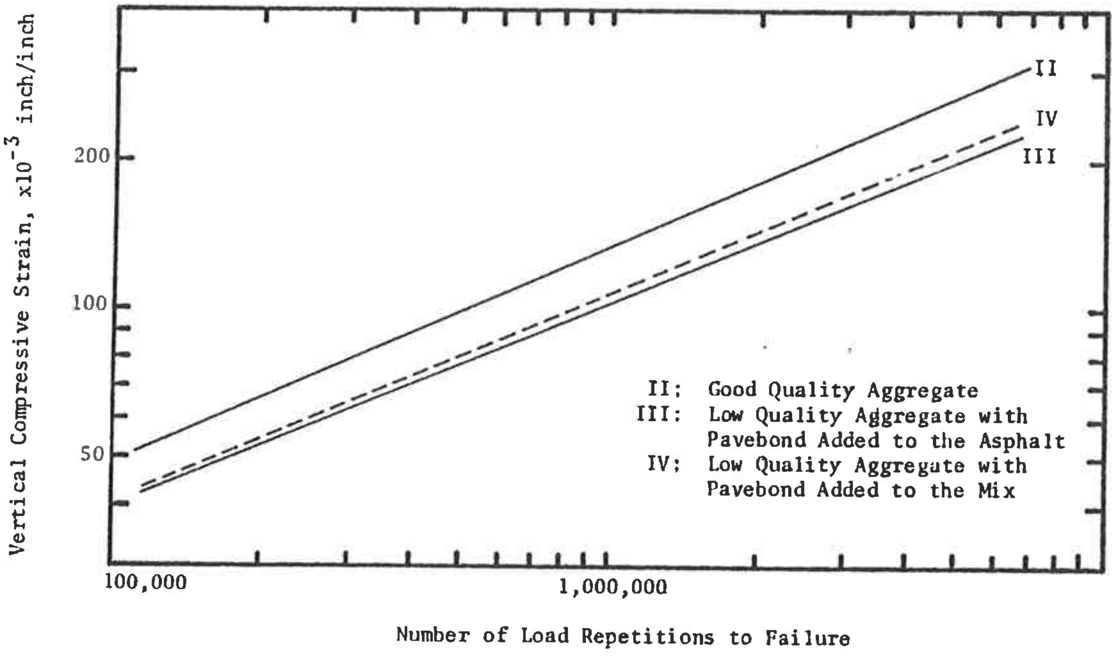


Figure 32b. Influence of Pavebond on Permanent Deformation Conditioned Samples

6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt - 96% Compaction

Table 24. Influence of Passing No. 10 on Permanent Deformation

Type of Mix	Permanent Deformation	
	As Compacted	Conditioned
Standard *	$\varepsilon_c = [2.60(\varepsilon_t) - 2.77 \times 10^{-5}]\ N^{.506}$	$\varepsilon_c = [2.22(\varepsilon_t) + 5.42 \times 10^{-5}]N^{.610}$
Mix with 30% Passing No. 10	$\varepsilon_c = [2.72(\varepsilon_t) - 5.16 \times 10^{-5}]N^{.495}$	$\varepsilon_c = [.523(\varepsilon_t) + 1.96 \times 10^{-4}]\ N^{.545}$
Mix with 35% Passing No. 10	$\varepsilon_c = [2.23\ (\varepsilon_t) + 2.32 \times 10^{-5}]\ N^{.464}$	$\varepsilon_c = [1.98(\varepsilon_t) + 1.54 \times 10^{-4}]\ N^{.469}$

* 6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt - 92% Compaction

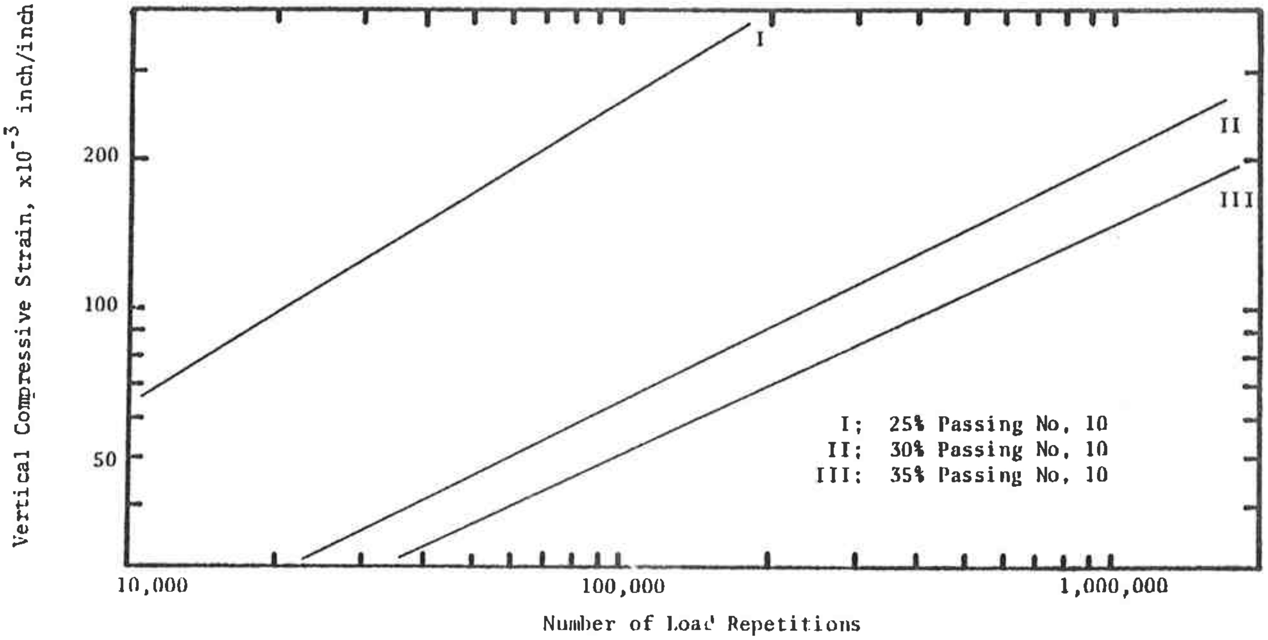


Figure 33a. Influence of Percent Passing No. 10 on Permanent Deformation
As Compacted Samples
6% Passing No. 200 - 6% Asphalt - 92% Compaction

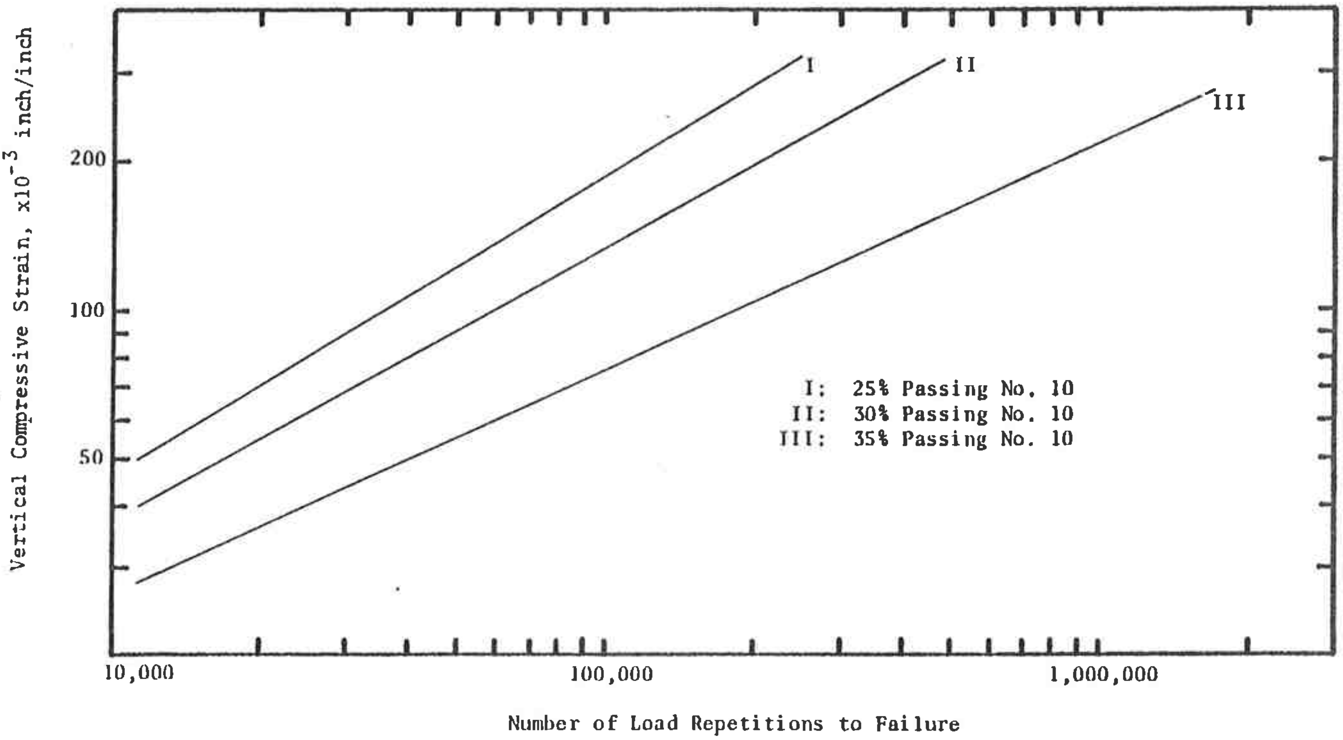


Figure 33b. Influence of Percent Passing No. 10 on Permanent Deformation Conditioned Samples

6% Passing No. 200 - 6% Asphalt - 92% Compaction

results have been summarized in Table 25. Resilient modulus was measured on all samples before running the indirect tensile test. Table 25a gives the average as-compacted resilient modulus, Bulk Specific Gravity and tensile strength. For the conditioned samples, resilient modulus was measured both before and after conditioning. These data are recorded in Table 25b along with the conditioned tensile strength.

The relationship between the resilient modulus and indirect-tensile-strength is shown in Figure 34 and can be expressed as

$$M_e = 2.31 \times 10^3 \, [S] = 9.93 \times 10^3 \qquad (19)$$

The coefficient of correlation was found to be equal to .613. This low value can be explained by two test series which results are out of range: 6% passing No. 200, 6% asphalt content and 10% passing No. 200, 5% asphalt content.

Table 25a. Indirect Tensile Strength Test Data as Compacted Samples

- Tensile Strength (PSI)
- Resilient Modulus (PSI)
- Bulk Specific Gravity

Level of Compaction	2% Passing #200			6% Passing #200			10% Passing #200		
	Asphalt Content			Asphalt Content			Asphalt Content		
	5	6	7	5	6	7	5	6	7
2nd Compaction					198.81 493,000 2.42				
1st Compaction					123.43 306,000 2.37				
95 Blows at 100 PSI 500 PSI Leveling Load	84.80 200,000 2.16		69.73 159,000 2.16	111.91 214,000 2.23	112.82 127,000 2.25	100.35 145,000 2.24	81.07 281,000 2.23		88.67 252,000 2.28
30 Blows at 100 PSI 300 PSI Leveling Load					95.05 191,000 2.21				

Table 25b. Indirect Tensile Strength Test Data Conditioned Samples

- Tensile Strength (PSI)
- Resilient Modulus Before Conditioning
- Resilient Modulus After Conditioning

Level of Compaction	2% Passing #200			6% Passing #200			10% Passing #200		
	Asphalt Content			Asphalt Content			Asphalt Content		
	5	6	7	5	6	7	5	6	7
2nd Compaction					108.77 328,000 262,000				
1st Compaction					84.31 268,000 254,000				
95 Blows at 100 PSI 500 PSI Leveling Load	33.70 191,000 N.A.		40.42 136,000 N.A.	58.76 194,000 158,000	50.37 146,000 103,000	53.48 138,000 123,000	16.05 304,000 N.A.		59.38 258,000 N.A.
30 Blows at 100 PSI 300 PSI Leveling Load					27.45 200,000 N.A.				

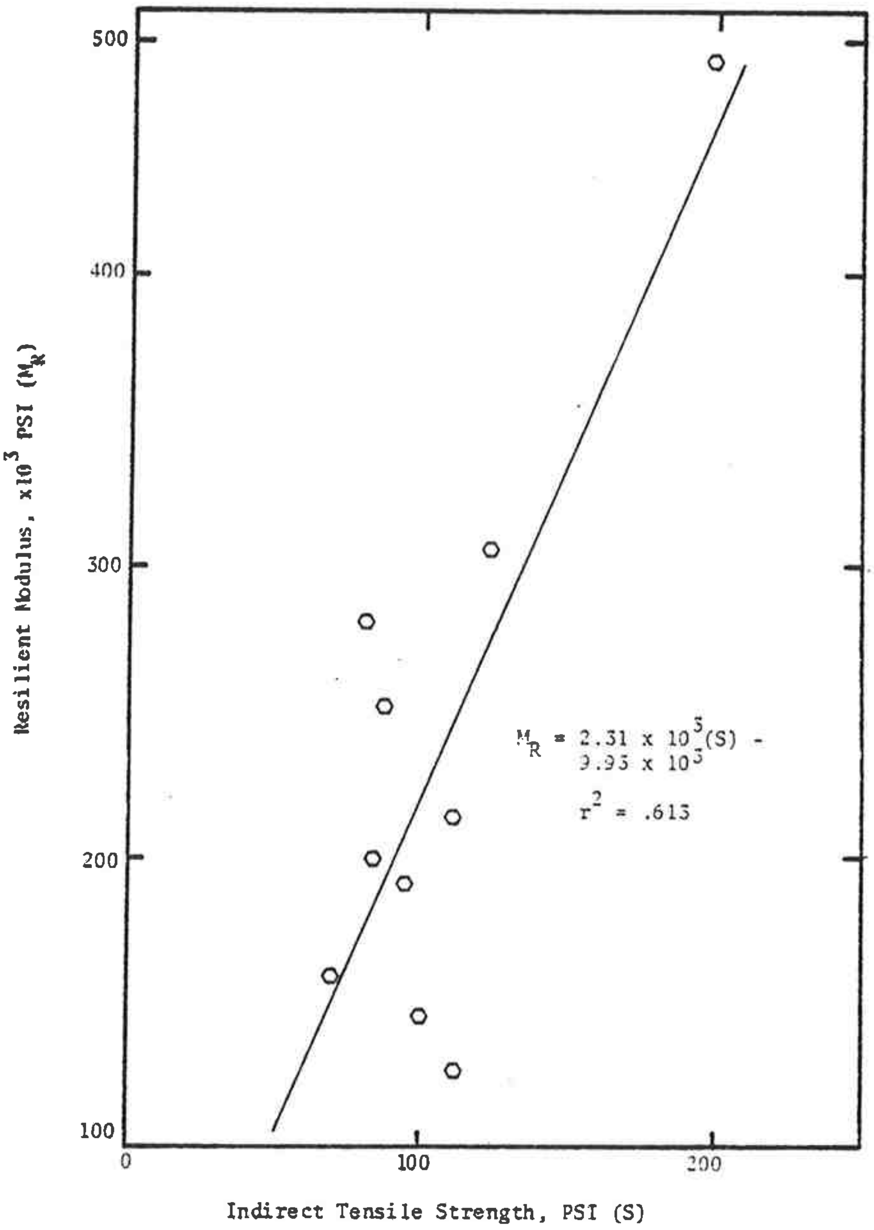


Figure 34. Relationship Between Resilient Modulus and Indirect Tensile Strength

5.0 DEVELOPMENT OF PAY ADJUSTMENT FACTORS

Fatigue

The testing program covered a wide range of mix variables. From this, it is possible to evaluate the reduction in pavement life when the design specifications are not met. Using the mix fulfilling the design specifications as a reference mix, the fatigue life of mixes not meeting specifications is determined and compared with the standard mix fatigue life. The resulting ratios of the fatigue lives can be used as one way of estimating corresponding pay factors.

This calculation will be accomplished for three strain levels: 125, 100 and 50 microstrain. The high strain level (125) is associated with low volume roads and the lower strain levels (50 and 100) are associated with primary and Interstate type facilities. The values corresponding to 100 microstrain were considered as a good average, since all tests were performed between 50 and 125 microstrains. Table 26 presents the estimated reduction in life when the design mix density is not achieved. The standard mix is composed of 6% passing No. 200 sieve, 25% passing No. 10 sieve, 6% asphalt content and is compacted at 96% of the maximum laboratory density. This standard mix is compared in Table 26 with mixes compacted at different levels: 100, 92 and 91% compaction, corresponding to the mix Bulk Specific Gravities shown in the table.

For a fixed value of the mix tensile strain, the standard mix has an estimated fatigue life equal to N_{sm}. The mix out of specification has an estimated fatigue life N_{os}. The percent reduction in life is given by

$$\frac{N_{os}}{N_{sm}} \times 100 \qquad (20)$$

The pay factors shown in Table 26 indicate that the effect of variations in mix density result in wider variations in life at low strain values than high strain values.

Table 27 gives the pay factors computed for mixes with low and high asphalt contents. The standard mix is composed of 6% passing No. 200 sieve, 25% passing No. 10, 6% asphalt and compacted at 92%. The standard level of compaction is fixed at 92% compaction because it is expected that variations in the mix behavior related to the asphalt content or the amount of fines will be emphasized at a relatively low level of compaction. As the fatigue

Table 26. Estimated Reduction in Pavement Life Fatigue Criteria Effect of Mix Density.

	Level of Compaction	Mix BSG	Test Condition*	STRAIN LEVEL		
				50 με	100 με	125 με
PAVEMENT LIFE	Standard 96%	2.31	B.C.	1.68×10^5	2.55×10^4	1.39×10^4
			A.C.	1.59×10^5	3.50×10^4	2.15×10^4
	100%	2.41	B.C.	1.37×10^7	1.29×10^5	2.86×10^4
			A.C.	6.80×10^5	7.75×10^4	3.97×10^4
	92%	2.22	B.C.	1.01×10^5	2.07×10^4	1.24×10^4
			A.C.	1.34×10^5	3.11×10^4	1.95×10^4
	91%	2.19	B.C.	7.91×10^4	1.94×10^4	1.23×10^4
			A.C.	2.80×10^4	1.26×10^4	9.76×10^3
PAY FACTOR	Standard 96%	2.31	B.C.	1.0	1.0	1.0
			A.C.	1.0	1.0	1.0
	100%	2.41	B.C.	81.5	5.06	2.06
			A.C.	3.91	2.21	1.84
	92%	2.22	B.C.	.601	.812	.892
			A.C.	.842	.890	.906
	91%	2.19	B.C.	.471	.761	.885
			A.C.	.177	.361	.456

*B.C. - Before Conditioning
A.C. - After Conditioning

Table 27. Estimated Reduction in Pavement Life Fatigue Criteria Effect of Asphalt Content 6% Passing No. 200 - 92% Compaction

	Asphalt Content	Mix BSG	Test Condition*	STRAIN LEVEL		
				50 με	100 με	125 με
PAVEMENT LIFE	Standard 6%	2.22	B.C.	1.01×10^5	2.07×10^4	1.24×10^4
			A.C.	1.34×10^5	3.11×10^4	1.95×10^4
	5% Asphalt	2.24	B.C.	1.07×10^5	2.13×10^4	1.27×10^4
			A.C.	8.14×10^4	2.31×10^4	1.54×10^4
	7% Asphalt	2.24	B.C.	1.40×10^5	2.39×10^4	1.35×10^4
			A.C.	1.53×10^5	3.34×10^4	2.05×10^4
PAY FACTOR	Standard 6%	2.22	B.C.	1.0	1.0	1.0
			A.C.	1.0	1.0	1.0
	5% Asphalt	2.24	B.C.	1.06	1.03	1.02
			A.C.	.610	.740	.788
	7% Asphalt	2.24	B.C.	1.39	1.16	1.09
			A.C.	1.14	1.07	1.05

*B.C. - Before Conditioning
A.C. - After Conditioning

results indicated, the effects of a change in the asphalt content on fatigue life are less than density for the asphalt content range considered in this analysis.

The impact of amount of fines on fatigue life is shown in Table 28 for mix composed of 5% asphalt, and Table 29 for mixes composed of 7% asphalt. The complex asphalt-fines interaction substantially affect the fatigue life of the mix. A comparison between Tables 28 and 29 shows that the influence of the amount of fines is more sensitive at the low asphalt content. Increasing the amount of fines from 2 to 10% increase the pay factor by a factor of approximately 10 at 5% asphalt content and 4 at 7% asphalt content.

Pay factors corresponding to different percent passing No. 10 are presented in Table 30. As-compacted and conditioned results both indicate a reduction in pavement life when the percent passing the No. 10 sieve is increased above optimum.

In summary, the fatigue life of the mix considered in this project is affected substantially by the mix density and the amount of fines. The influence of the asphalt content is relatively low within ±1% from the design optimum asphalt content.

Permanent Deformation

The reduction in pavement life for out of specification mixes can also be estimated from the permanent deformation data. Pavement failure in the permanent deformation mode is defined for a rut depth of 3/4 inch (1.9 cm). Assuming that most of the permanent deformation takes place in the asphalt concrete layer, a 3/4 inch (1.9 cm) rut depth corresponds to a cumulative permanent strain equal to:

$$\varepsilon_F = \frac{3/4}{T} \times 100 \tag{21}$$

where ε_F = Cumulative permanent strain at failure

T = Asphalt concrete layer thickness, inches.

If ε_F is set, it is possible to calculate the corresponding number of load repetitions from the permanent deformation data. As indicated earlier, the

Table 28. Estimated Reduction in Pavement Life Fatigue Criteria
Effect of P_{200} at 5% Asphalt

	Percent P_{200}	Mix BSG	Test Condition*	STRAIN LEVEL		
				50 με	100 με	125 με
PAVEMENT LIFE	Standard 6% P_{200}	2.24	B.C.	1.07×10^{5}	2.13×10^{4}	1.27×10^{4}
			A.C.	8.14×10^{4}	2.31×10^{4}	1.54×10^{4}
	2% P_{200}	2.17	B.C.	2.90×10^{4}	1.31×10^{4}	1.01×10^{4}
			A.C.	1.08×10^{4}	7.16×10^{3}	6.27×10^{3}
	10% P_{200}	2.27	B.C.	2.06×10^{5}	2.73×10^{4}	1.42×10^{4}
			A.C.	2.10×10^{5}	4.12×10^{4}	2.44×10^{4}
PAY FACTOR	Standard 6% P_{200}	2.24	B.C.	1	1	1
			A.C.	1	1	1
	2% P_{200}	2.17	B.C.	.272	.612	.794
			A.C.	.133	.311	.493
	10% P_{200}	2.27	B.C.	1.94	1.28	1.12
			A.C.	2.54	1.79	1.58

*B.C. - Before Conditioning
A.C. - After Conditioning

Table 29. Estimated Reduction in Pavement Life-Fatigue Criteria
Effect of P_{200} at 7% Asphalt

	Percent P_{200}	Mix BSG	Test Condition*	STRAIN LEVEL		
				50 με	100 με	125 με
PAVEMENT LIFE	Standard 6%	2.24	B.C.	1.40×10^{5}	2.39×10^{4}	1.35×10^{4}
			A.C.	1.53×10^{5}	3.34×10^{4}	2.05×10^{4}
	2% P_{200}	2.19	B.C.	3.87×10^{5}	1.50×10^{4}	1.10×10^{4}
			A.C.	2.39×10^{4}	1.13×10^{4}	8.87×10^{3}
	10% P_{200}	2.30	B.C.	3.94×10^{5}	4.66×10^{4}	2.34×10^{4}
			A.C.	4.38×10^{5}	6.24×10^{4}	3.33×10^{4}
PAY FACTOR	Standard 6%	2.24	B.C.	1	1	1
			A.C.	1	1	1
	2% P_{200}	2.19	B.C.	.276	.626	.814
			A.C.	.156	.33	.432
	10% P_{200}	2.30	B.C.	2.81	1.95	1.73
			A.C.	2.87	1.87	1.62

*B.C. - Before Conditioning
A.C. - After Conditioning

Table 30. Estimated Pavement Life and Associated Pay Factors for Three Levels of Percent Passing No. 10 - Fatigue Basis

	Percent P_{10}	Mix BSG	Test Condition*	STRAIN LEVEL		
				50με	100με	125με
PAVEMENT LIFE	Standard 25% P_{10}	2.22	B.C.	1.01×10^5	2.07×10^4	1.24×10^4
			A.C.	1.34×10^5	3.11×10^4	1.35×10^4
	30% P_{10}	2.23	B.C.	5.50×10^4	1.67×10^4	1.14×10^4
			A.C.	2.23×10^4	1.09×10^4	8.67×10^3
	35% P_{10}	2.21	B.C.	5.57×10^4	1.70×10^4	1.16×10^4
			A.C.	4.64×10^4	1.71×10^4	1.24×10^4
PAY FACTOR	Standard 25% P_{10}	2.22	B.C.	1.0	1.0	1.0
			A.C.	1.0	1.0	1.0
	30% P_{10}	2.23	B.C.	.55	.81	.92
			A.C.	.17	.35	.45
	35% P_{10}	2.21	B.C.	.55	.82	.93
			A.C.	.35	.55	.64

***B.C. - Before Conditioning**
A.C. - After Conditioning

cumulative permanent strain is expressed in function of the mix tensile strain and the number of load repetition:

$$\varepsilon_c = (Ax(\varepsilon_T) + B)\ N^s \tag{22}$$

where ε_c = Cumulative permanent strain

ε_T = Initial tensile strain

A,B,s = Regression constraints

N = Number of load repetitions

Knowing A, B and s, it is possible to determine N as a function of ε_T:

$$N = \left[\frac{\varepsilon_c}{(A(\varepsilon_T) + B)}\right]^{1/s} \tag{23}$$

The average pavement thickness on the North Oakland-Sutherlin project is 5 inches (12.7 cm). From equation (17), the cumulative permanent strain at failure is:

$$\varepsilon_F = \frac{3/4}{5} \times 100 = 15\%$$

Equation (16) becomes:

$$N = \left[\frac{.15}{(A(\varepsilon_T) + B}\right]^{1/s} \tag{24}$$

Estimated pavement lives have been calculated using equation (17) for three values of the initial tensile strain: 50, 100 and 125 microstrain.

Table 31 illustrates the importance of the mix density in the development of rutting within the asphalt concrete layer. Increasing the mix Bulk Specific Gravity from 2.31 to 2.41 improves the resistance to rutting by a factor of over 1000. The influence of the asphalt content presented in Table 32 indicates better performance for 5 and 7% asphalt content. This is not realistic and should be checked with more test results before any conclusion can be drawn.

The reduction in pavement life resulting from a change percent passing No. 200 sieve is shown in Tables 33 and 34. As discussed previously, 6% passing No. 200 sieve appears to be the optimum for minimum susceptibility to permanent deformation.

Table 31. Estimated Reduction in Pavement Life-Permanent Deformation Criteria
Effect of Density

	Level of Compaction	Mix BSG	Test Condition*	STRAIN LEVEL		
				50 με	100 με	125 με
PAVEMENT LIFE	Standard 96%	2.31	B.C.	3.24×10^{8}	3.71×10^{6}	1.66×10^{6}
			A.C.	N.A.	N.A.	N.A.
	100%	2.41	B.C.	1.66×10^{11}	7.25×10^{8}	2.30×10^{8}
			A.C.			
	92%	2.22	B.C.	1.81×10^{6}	3.58×10^{5}	2.20×10^{5}
			A.C.			
	91%	2.19	B.C.	7.47×10^{5}	2.56×10^{5}	1.75×10^{5}
			A.C.			
PAY FACTOR	Standard 96%	2.31	B.C.	1.0	1.0	1.0
			A.C.	1.0	1.0	1.0
	100%	2.41	B.C.	513.0	196.0	139.0
			A.C.	N.A.	N.A.	N.A.
	92%	2.22	B.C.	.006	.10	.133
			A.C.	N.A.	N.A.	N.A.
	91%	2.19	B.C.	.002	.069	.106
			A.C.	N.A.	N.A.	N.A.

*B.C. - Before Conditioning
A.C. - After Conditioning

Table 52. Estimated Pavement Life and Associated Pay Factors for Three Levels of Asphalt Content - Permanent Deformation Basis

	Asphalt Content	Mix BSG	Test Condition*	STRAIN LEVEL		
				50 με	100 με	125 με
PAVEMENT LIFE	Standard	2.22	B.C.	1.81×10^6	3.58×10^5	2.20×10^5
			A.C.	4.07×10^5	6.89×10^4	4.31×10^4
	7% Asphalt	2.24	B.C.	9.53×10^6	3.37×10^6	2.26×10^6
			A.C.	4.21×10^6	9.13×10^5	5.47×10^5
	5% Asphalt	2.24	B.C.	4.37×10^6	1.07×10^6	6.72×10^5
			A.C.	1.44×10^6	6.17×10^5	4.44×10^5
PAY FACTOR	Standard	2.22	B.C.	1.0	1.0	1.0
			A.C.	1.0	1.0	1.0
	7% Asphalt	2.24	B.C.	5.27	9.42	10.3
			A.C.	10.4	13.3	12.7
	5% Asphalt	2.24	B.C.	2.42	2.99	3.06
			A.C.	3.53	8.96	10.3

*B.C. - Before Conditioning
A.C. - After Conditioning

Table 53. Estimated Pavement Life and Associated Pay Factors for Three Levels of Percent Passing No. 200 at 5% Asphalt Content - Permanent Deformation Basis

	Percent P_{200}	Mix BSG	Test Condition*	STRAIN LEVEL		
				50 με	100 με	125 με
PAVEMENT LIFE	Standard 6% P_{200}	2.24	B.C.	4.37×10^6	1.07×10^6	6.72×10^5
			A.C.	1.44×10^6	6.17×10^5	4.44×10^5
	10% P_{200}	2.17	B.C.	8.59×10^5	5.48×10^5	4.52×10^5
			A.C.	1.27×10^6	5.11×10^5	3.65×10^5
	2% P_{200}	2.27	B.C.	3.19×10^6	8.26×10^5	5.22×10^5
			A.C.	1.71×10^5	1.26×10^5	1.10×10^5
PAY FACTOR	Standard 6% P_{200}	2.24	B.C.	1.0	1.0	1.0
			A.C.	1.0	1.0	1.0
	10% P_{200}	2.17	B.C.	.20	.51	.67
			A.C.	.88	.83	.82
	2% P_{200}	2.27	B.C.	.73	.77	.78
			A.C.	.12	.20	.25

*B.C. - Before Conditioning
A.C. - After Conditioning

Table 35 gives the pay factors computed for 30 and 35% passing the No. 10 sieve. Results for 100 microstrain indicate increasing pavement performance with increasing amount passing the No. 10 sieve. The amount passing the No. 10 sieve is therefore a variable to be considered to limit permanent deformation.

It is interesting that the pay adjustment factors increase with increasing mix elastic tensile strain. Higher strain values would normally be associated with low volume roads while lower strain values would normally be associated with high volume (or Interstate) facilities. For a given mix variation, therefore, the results indicate a greater potential reduction in life for high volume type facilities.

Table 34. Estimated Pavement Life and Associated Pay Factors for Three Levels of Percent Passing No. 200 at 7% Asphalt Content - Permanent Deformation Basis

	Percent P_{200}	Mix BSG	Test Condition*	STRAIN LEVEL		
				50 με	100 με	125 με
PAVEMENT LIFE	Standard 6% P_{200}	2.24	B.C.	9.53×10^6	3.37×10^6	2.26×10^6
			A.C.	4.21×10^6	9.13×10^5	5.47×10^5
	10% P_{200}	2.19	B.C.	2.01×10^6	1.29×10^6	1.07×10^6
			A.C.	1.05×10^7	1.78×10^6	1.01×10^6
	2% P_{200}	2.30	B.C.	6.07×10^5	3.17×10^5	2.45×10^5
			A.C.	3.37×10^5	7.89×10^5	1.49×10^5
PAY FACTOR	Standard 6% P_{200}	2.24	B.C.	1.0	1.0	1.0
			A.C.	1.0	1.0	1.0
	10% P_{200}	2.19	B.C.	.211	.38	.47
			A.C.	2.50	1.95	1.84
	2% P_{200}	2.30	B.C.	.06	.09	.11
			A.C.	.08	.21	.272

*B.C. - Before Conditioning
A.C. - After Conditioning

Table 35. Estimated Pavement Life and Associated Pay Factors for Three Levels of Percent Passing No. 10 - Permanent Deformation Basis

Effect of Passing No. 10 Sieve
6% Asphalt
92% Compaction

	Percent P_{10}	Mix BSG	Test Condition*	STRAIN LEVEL		
				50με	100με	125με
PAVEMENT LIFE	Standard 25% P_{10}	2.22	B.C.	1.81×10^6	3.58×10^5	2.20×10^5
			A.C.	4.07×10^5	6.89×10^4	4.31×10^4
	30% P_{10}	2.23	B.C.	3.67×10^6	5.28×10^5	3.07×10^5
			A.C.	1.55×10^5	1.27×10^5	1.15×10^5
	35% P_{10}	2.21	B.C.	3.68×10^6	1.00×10^6	6.47×10^5
			A.C.	8.18×10^5	4.04×10^5	3.05×10^5
PAY FACTOR	Standard 25% P_{10}	2.22	B.C.	1.0	1.0	1.0
			A.C.	1.0	1.0	1.0
	30% P_{10}	2.23	B.C.	2.03	1.48	1.40
			A.C.	.382	1.84	2.67
	35% P_{10}	2.21	B.C.	2.04	2.81	2.94
			A.C.	2.01	5.87	7.08

*B.C. - Before Conditioning
A.C. - After Conditioning

6.0 CONCLUSIONS AND RECOMMENDATIONS

Performance of the mix used in the construction of the North Oakland-Sutherlin project was evaluated from dynamic testing of laboratory compacted samples. Mix resilient modulus, fatigue life and permanent deformation characteristics were determined for samples prepared within the following range of variables:

(1) Mix level of compaction: 100%, 96%, 92% and 91%
(2) Asphalt content: 5, 6 and 7%
(3) Percent passing No. 200: 2, 6 and 10%
(4) Percent passing No. 10: 25, 30 and 35%
(5) Aggregate quality: satisfactory and borderline quality

Study of the first three variables presented above was done using the same type of aggregate as used for the construction of the North Oakland-Sutherlin project.

It was found that the mix level of compaction is the controlling factor for all mix dynamic properties. Increasing the mix density increases the mix stiffness, fatigue life and resistance to permanent deformation. High mix density also reduces substantially the damaging action of water and other environmental factors. One percent change in asphalt content from the design optimum did not change significantly the fatigue life of the mix, but a slight increase in fatigue life was noted when the asphalt content was increased to 7%. Permanent deformation results showed minimum mix performance at 6% asphalt content, and increasing resistance to permanent deformation when the asphalt content was raised to 7%. Six percent passing No. 200 sieve appeared to be an optimum value for mix resistance to permanent deformation, although fatigue life improved substantially when the amount passing No. 200 was increased to 10%. The improved fatigue performance due to the higher percent of fines may be related to the fact that the primary evaluation was conducted at a low level of compaction. Thus the higher the percent fines the lower the air voids. Increasing the amount passing the No. 10 sieve decreased slightly the mix fatigue life but increased substantially the mix resistance to permanent deformation.

These results seem to indicate that the optimum design asphalt content (6%) is more likely to be a minimum acceptable value, whereas 6% passing No. 200 sieve is the optimum fines content according to the permanent deformation criteria.

Samples prepared with good quality aggregate were stiffer than the standard samples, but did not show any significant improvement in fatigue life or resistance to permanent deformation. Based on fatigue curves, pay factors have been developed to show variations in mix performance resulting from changes in mix density, asphalt content and percent passing No. 200. These data, shown in detail earlier, have been summarized in Table 36. The values presented were calculated for a mix tensile strain of 100 microstrain. Pavement life data indicate that fatigue life is generally shorter than permanent deformation life. This is primarily due to the temperatures used for the tests. Had higher temperatures been used the life for permanent deformation would probably have been shorter than that for fatigue. At this time, the permanent deformation pay factors are not included in this summary.

Only the conditioned data were considered in Table 22, since conditioned data are assumed to be more closely duplicating a typical pavement condition. Pay factors developed at 2 and 10% passing the No. 200 sieve are the average pay factors calculated at 5 and 7% asphalt. The results corroborate earlier remarks:

(1) Lowering the mix density decreases the mix resistance to fatigue life.

(2) Fatigue life improves with increasing asphalt content.

(3) Percent passing No. 200 sieve affects primarily the mix permanent deformation. Fatigue life improves when increasing the amount of fines for the low level of compaction evaluated.

However, it should be emphasized that the findings are applicable only for the ambient test temperatures employed. The results should be expected to change at higher temperature (critical for permanent deformation) and at lower temperatures (critical for cracking). Therefore, recommendations for further research include:

(1) Test results indicate that the optimum asphalt content was not in the range of values considered in this study. It would therefore be of value to run tests at higher asphalt content (8 or 9%) to find the optimum for the fatigue and permanent deformation criteria.

(2) The permanent deformation study indicated that increasing the amount of fines increases the mix resistance to permanent deformation at ambient temperatures. Fatigue life also increases substantially when the percent fines is increased. Tests run on samples prepared with more than 10% passing No. 200 sieve and at higher temperatures are needed to verify these results.

Table 36. Summary of Most Critical Pay Adjustment Factors* (at 100 microstrain)

	Percent Level of Compaction			
	96	100	92	91
Pay Factor	1.0	2.21	.89	.36

	Percent Asphalt Content		
	6	5	7
Pay Factor	1.0	.74	1.07

	Percent Fines		
	6	2	10
Pay Factor	1.0	.32	1.83

	Percent Passing No. 10		
	25	30	35
Pay Factor	1.0	.35	.55

*These values are based on an analysis of this one project. They will be combined with the results of two other projects to formulate recommendations for pay adjustment factors for use in Oregon.

7.0 REFERENCES

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Appendix A - Summary of Core Data

This appendix summarizes core data taken across the panel at selected locations on the North Oakland-Sutherlin Project. Table A-1 summarizes the average results of tests. Ten cores were taken from Station 1655+85, 5 cores from Station 1753+95, 2 cores from Station 1700+00 and 1705+00 and 2 cores from Station 1740+00 and 1781+00.

Table A-1. Summary of Average Test Results

*Station 1655+85

**Station 1700 and 1705

Paving Tested:	*Wearing Surface Class "E"	*Top Lift Class "B"	**Bottom Lift Class "B"
Mix Property:			
Bulk Sp. Gr. In place	2.09	2.24	2.17
Bulk Sp. Gr. R. Compacted	2.29	2.39	2.45
Max. Sp. Gravity	2.52	2.51	2.51
% Voids, In place	17.2	10.0	15.7
Gradation, % passing			
1/4 "	61	62	60
No. 10	16	30	31
No. 200	3.9	6.3	8.2
Asphalt Content	6.5	7.2	7.2
Modulus, ksi Lab. Compacted	218	468	-

Table A-1. Con't. Summary of Average Test Results

*Station 1753+95

**Station 1740 and 1781

Paving Tested:	*Wearing Surface Class "E"	*Top Lift Class "B"	**Bottom Lift Class "B"
Mix Property:			
Bulk Sp. Gr. In place	2.09	2.28	2.11
Bulk Sp. Gr. R. Compacted	2.24	2.40	2.42
Max. Sp. Gravity	2.52	2.47	2.49
% Voids, In place	17.2	8.0	15.5
Gradation, % passing			
1/4 "	64	67	65
No. 10	18	34	31.5
No. 200	3.6	6.4	8.7
Asphalt Content	7.0	7.1	7.7
Modulus, ksi Lab. Compacted	220	476	1232