IMPACT OF VARIATION IN MATERIAL PROPERTIES ON ASPHALT PAVEMENT LIFE

EVALUATION OF CASTLE ROCK-CEDAR CREEK PROJECT

HP & R Study: 0815157

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 issued from a recent project built in 1979, Oregon State Highway Department and Oregon State University conducted a laboratory study to determine the relationship between asphalt concrete pavement performance and mix level of compaction, asphalt content, and mix gradation.

Conventional tests and improved dynamic tests were run 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 specimens prepared at the design optimum with the performance of mix out of specifications. It was found that performance is primarily affected by the mix level of compaction. Fatigue data corroborated the design optimum asphalt content (6%), and showed a strong interaction between the asphalt content and the amount of fines. Mix susceptibility to permanent deformation decreased when increasing the amount of fines and decreasing the asphalt content. A summary table giving the most critical pay adjustment factors between the fatigue and the permanent deformation criteria is developed in the conclusions and recommendations chapter.

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This report is the third of a series of reports concerned with the impact of variations in material properties of asphalt pavement life. The data developed in this report will be combined with that developed for two other projects (North Oakland-Sutherlin and Warren-Scappose). All projects 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 (dryer 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 the Three Rivers Highway, between Hebo and Valley Junction, referred to as the Castle Rock-Cedar Creek project, was built in 1979. Progressive pavement raveling and potholing were noticed during the months following construction of this project. Evaluation of the reduction in pavement life resulting from changes in the design specifications (e.g., aggregate quality, gradation, density, asphalt content) requires a study of the mix dynamic properties under controlled conditions. A rational approach is needed to assess the effects of these mix variables on pavement life.

Purpose

The purpose of this report is to obtain a better understanding of the causes of the pavement problems noticed in the past years, and to develop relationships between pavement performance and the different 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 Castle Rock-Cedar Creek project. Three mix variables were considered in the study:

(1) Asphalt content

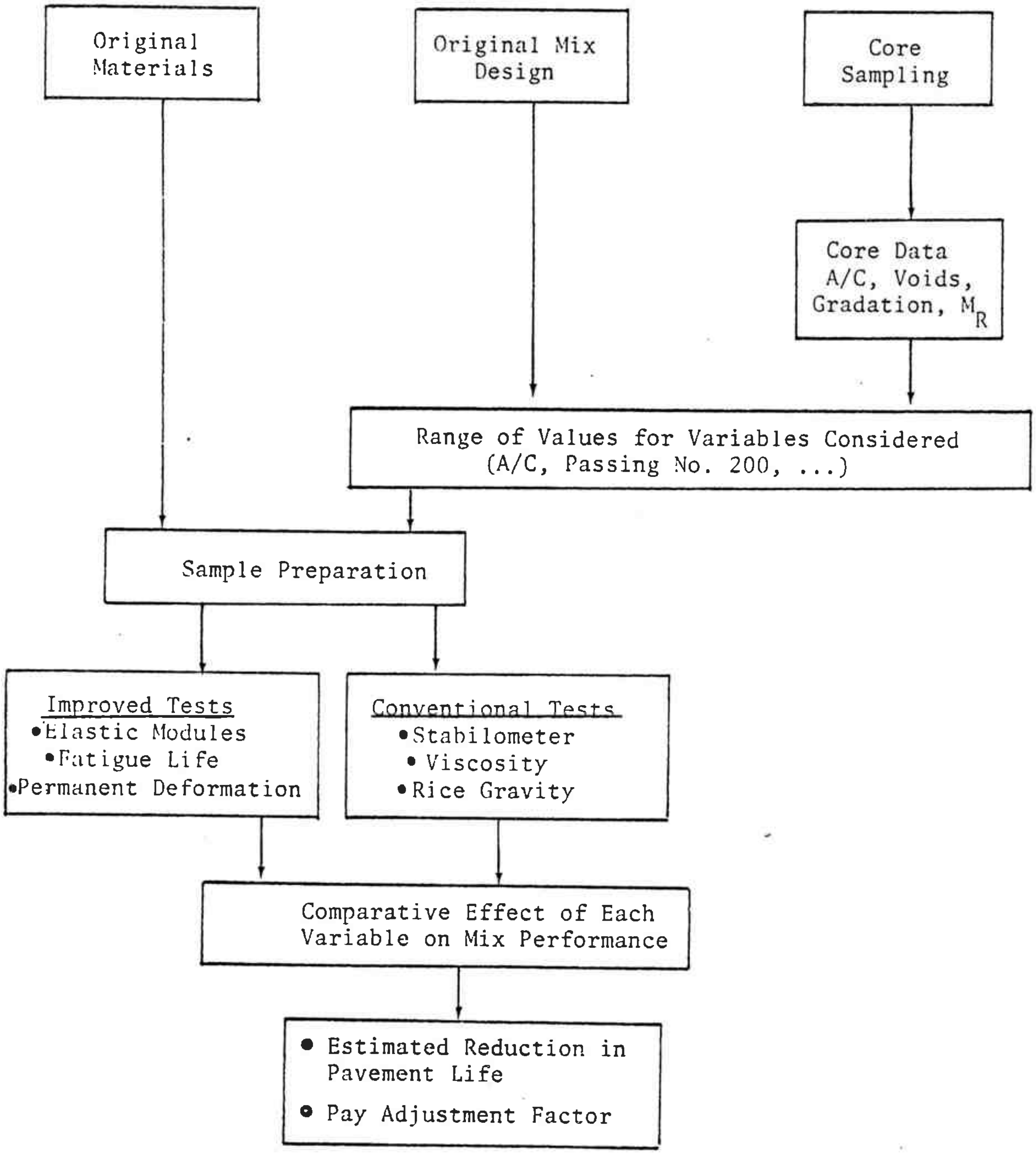
(2) Percent Passing No. 200 sieve (.074 mm).

(3) Mix density.

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 (See Appendix A). These are as follows:

(1) Asphalt content: 5% - 6% - 7%

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

(3) Mix level of compaction: 100% - 97% - 92% - 90%

Following the standard ODOT procedure, 4 inches (10 cm) in diameter by 2.5 inches (6 cm) high samples were fabricated for each set of conditions, using the same materials (asphalt and aggregate) as used during construction of the Castle Rock-Cedar Creek project.

The main types of pavement failure considered during the test program include fatigue cracking and rutting. All samples are 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 (stabilometer, void content, index of retained strength).

To identify the potential for stripping and raveling, elastic modulus, fatigue life and permanent deformation tests are performed both before and after vacuum saturation of the samples, followed by a freeze-thaw cycle.

Scope of Report

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

2.0 PROJECT DESCRIPTION

Location

The Castle Rock-Cedar Creek project is a section of the Hebo-Valley Junction Highway, located in Tillamook and Yamhill counties (Figure 1). Precise location of the project is shown on Figure 2. The project overall length is 11.7 miles (18.7 km).

Cross-Section

Reconstruction of this section of the Hebo-Valley Junction highway included an asphalt concrete base course and an asphalt concrete wearing surface, on top of the existing bituminous surface. Both layers were built using an ODOT class B mix. The average for the as constructed thickness is 2.0" (5.1 cm) for the base and 1.7" (4.3 cm) for the wearing surface.

Mix Design

A summary of the original mix design is presented in Table 3. This mix design was used for both the base and the top layers. The aggregate gradation was also the same for both layers, and correspond to a type B mix (Table 4). The recommended asphalt content was 6.1 percent for the wearing surface and 6.7 percent for the base course. The asphalt grade recommended was an AR 4000, from Chevron. The recommended mix temperature at time of placement was 270°F (132°C).

Project Data

Pavement raveling, potholing, variation in mix gradation and asphalt content were noticed during construction of the Castle Rock-Cedar Creek project in 1979.

Inspection of the mix showed that the coarse aggregate were 40 to 95 percent coated. Aggregates were dirty and contained soft materials (AASHTO T-112), and the asphalt was not uniformly mixed with the aggregates.

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 and the specified job mix tolerances (Table 6), it appears from the average field

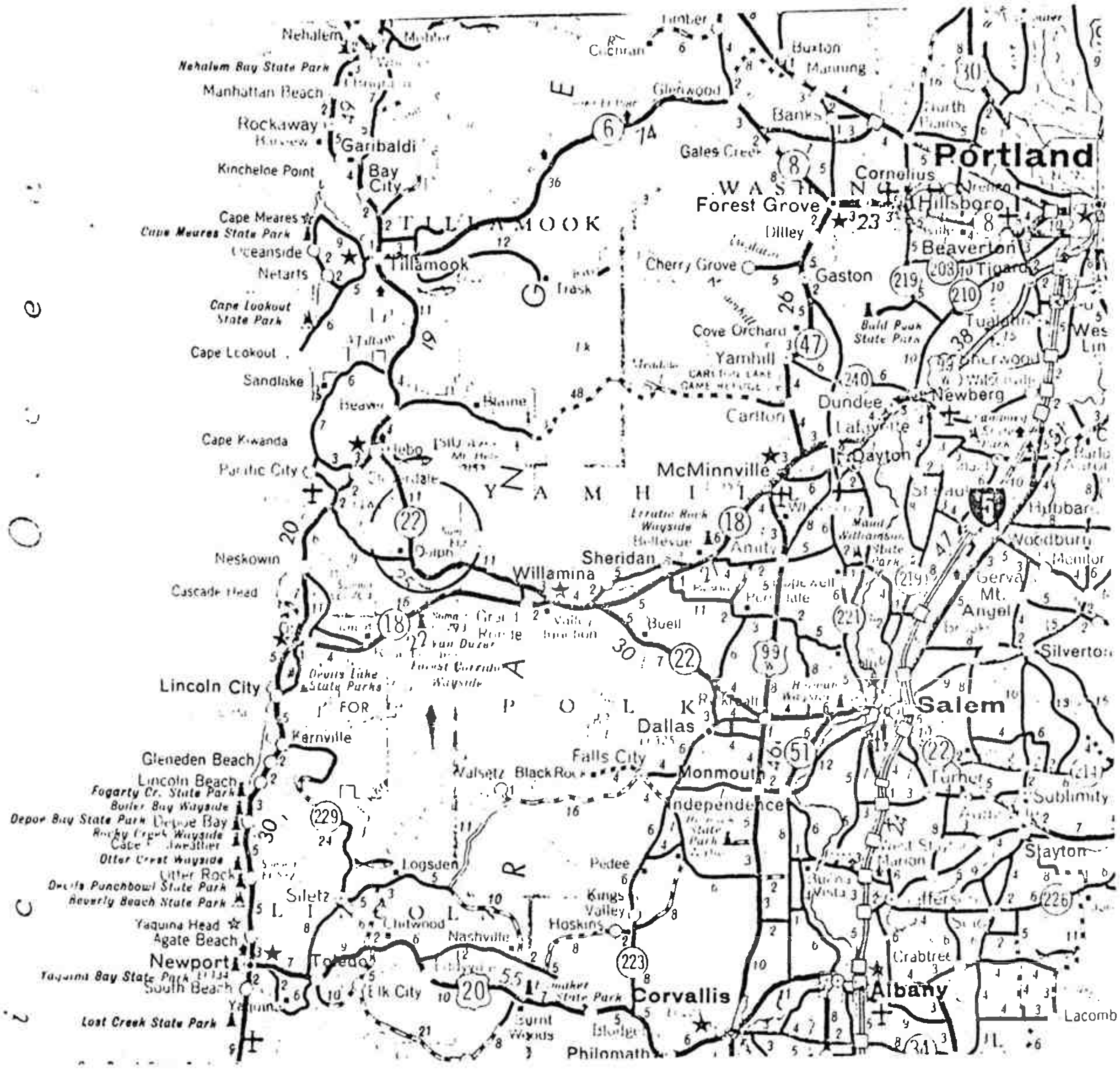


Figure 1. Map of Northwestern Oregon
Project Location

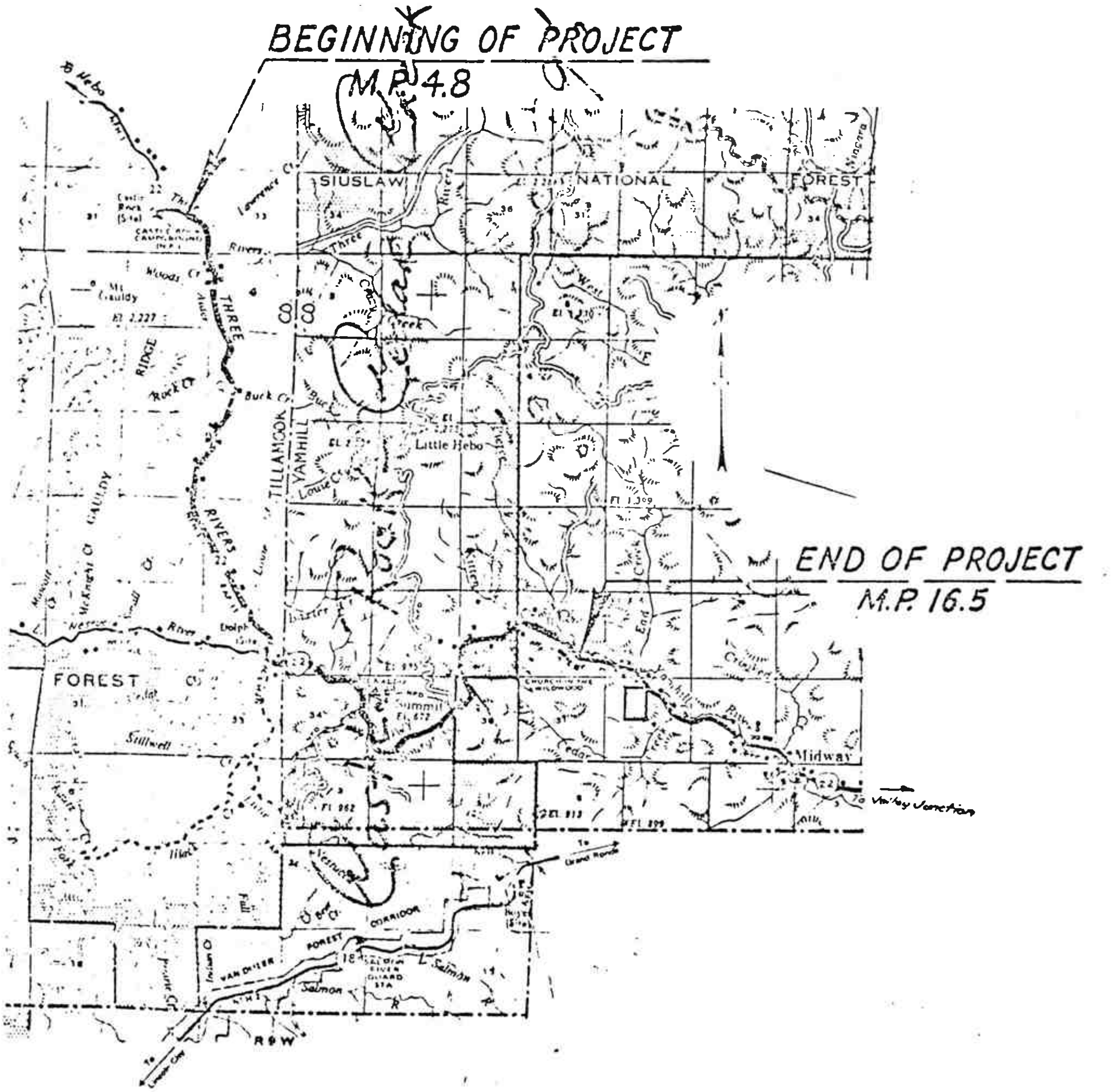


Figure 2. Castle Rock - Cedar Creek
Detail of Project Location

Table 3. Original Mix Design Test Results (ODOT)
Hebo Valley Junction Project

Asphalt Content - AR 4000	5.0	5.5	6.0	6.5	7.0
Asphalt Film Thickness	Suff.	Suff.	Suff. Thick.	Thick	V. Thick
Stability Value, 1st Compaction	41	41	37	37	39
Bulk Spe. Gravity, 1st Compaction	2.22	2.24	2.26	2.28	2.29
Percent Voids, 1st Compaction	11.2	9.7	8.1	6.6	5.4
Stability Value, 2nd Compaction	49	51	51	51	50
Bulk Spe. Gravity, 2nd Compaction	2.31	2.33	2.35	2.37	2.38
Percent Voids, 2nd Compaction	7.6	6.0	4.5	2.9	1.7
Rice Gravity	2.50	2.48	2.46	2.44	2.42
Index of Retained Strength AASHTO-T-165	56	-	79	-	87

Table 4. Mix Design: Aggregate Gradation, Class B

SIEVE SIZE	COMBINED AGGREGATE AASHTO-T-27	WASHED SIEVE AASHTO-T-11	JOB MIX TOLERANCE	SPECIFICATION
1"	100	100	100	100
3/4"	99	99	95 - 100	95 - 100
1/2"	90	90	81 - 93	-
1/4"	63	65	57 - 69	52 - 72
# 10	26	28	22 - 30	21 - 41
# 40	10	11	8 - 16	8 - 24
#200	3.6	4.6	3 - 7	3 - 7

Table 5. Summary of Construction Reports - 1979
Base and Top Lift Bituminous Mix Class "B"

IN PLACE MIX DATA	TOP LIFT		BASE LIFT	
	Average Value	Max. and Min. Values	Average Value	Max. and Min. Values
Mix Bulk Specific Gravity	2.15 ± .03 (10 tests)	2.11 - 2.19	2.21 ± .02 (5 tests)	2.17 - 2.23
Asphalt Content	6.04 ± .28 (27 tests)	5.4 - 6.5	6.56 ± .51 (30 tests)	5.1 - 7.2
Percent Passing No. 200	5.54 ± .47 (28 tests)	4.4 - 6.3	5.75 ± .42 (30 tests)	4.7 - 6.4
Percent Passing No. 10	27.23 ± 3.48 (28 tests)	20.6 - 35.8	27.87 ± 2.93 (30 tests)	22 - 34

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

	TOP LIFT			BASE LIFT		
	Daily Plant Test Results	Core Data	Job Mix Tolerance	Daily Plant Test Results	Core Data	Job Mix Tolerance
Mix Bulk Specific Gravity	2.15	2.12	-	2.21	2.14	-
Asphalt Content	6.04	6.0	5.6 - 6.6	6.56	7.0	6.2 - 7.2
Percent Passing No. 200	5.54	6.2	3 - 7	5.75	6.6	3 - 7
Percent Passing No. 10	27.23	27	22 - 30	27.87	29	22 - 30

data that the asphalt content and the amount passing No. 10 sieve were high, but the amount passing No. 200 was reasonably within specifications.

Table 5 also indicates 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 ODOT inspector's report which noted that the contractor's quality control was nonexistent, production erratic and workmanship sloppy.

Consequently, the pavement quality was largely reduced. ODOT field tests for production control of the mix indicated that the pavement raveling and potholing are the result of inadequate asphalt coating of aggregate, excess variation in mix gradation and asphalt content and excess soft material contained in the aggregate.

3.0 TEST RESULTS - ODOT

The Oregon State Highway Division testing program included the conventional tests such as standard mix design for each mix variable, gradation, asphalt content, void percent compaction content and resilient modulus of pavement samples from five locations and recovered asphalt properties. This chapter presents the results of their 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 on Table 7. Modulus and bulk specific gravities shown on this table were used as reference values during sample preparation at Oregon State University.

Core Data

Five core sampling sites were selected on the Castle Rock-Cedar Creek project. For each site, asphalt concrete cores were sampled across each panel at two foot intervals starting from the road centerline (See Appendix A for details). Table 8 summarize the results of the tests run on a total of 25 core samples. The mix density was low in both lifts which resulted in unusually high voids. The gradation limits shown for each aggregate size are minimum and maximum values. Shown on Figures 3 and 4 are the aggregate gradation for the surfacing and the base mixes, along with the job mix tolerances. Both mix gradations are out of specification below the No. 10 sieve size, with an excess amount of fines for both the top and the base layers. In both cases, the specified amount of passing No. 10 sieve has been respected. Above the No. 10 sieve size, the wearing surface gradation shows wide variability, with a tendency toward a fine gradation. Less variability can be noticed in the coarse part of the base gradation, which is within the specified gradation range. The excess amount of fines and the relatively finer gradation of the surface course can be partially explained by the presence of some soft materials in the mix aggregate.

Study of the cores also showed insufficient asphalt coating of the aggregate. From the inspection of the cores, it appears that the coarse aggregate is 40 to 95 percent coated. Tests run on the asphalt recovered from the mix

Table 7. Summary of OSHD Mix Designs for Variables Evaluated: Castle Rock - Cedar Creek

MIX TYPE	2% PASSING NO. 200			6% PASSING NO. 200			10% PASSING NO. 200			92% COMPACTION			88% COMPACTION		
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 Grav. T 209	2.46	2.41	2.37	2.47	2.43	2.39	2.46	2.42	2.38	2.43	2.39	2.36	2.44	2.40	2.38
1st Bulk Spe. Grav.	2.17	2.19	2.21	2.23	2.27	2.29	2.24	2.28	2.30	2.08	2.10	2.12	2.05	2.07	2.08
Voids, %	11.8	9.1	6.8	9.7	6.6	4.2	8.9	5.8	3.4	14.4	12.1	10.2	16.0	13.8	12.6
2nd Bulk Spe. Grav.	2.23	2.27	2.31	2.32	2.35	2.37	2.30	2.35	2.37	-	-	-	-	-	-
Voids, %	9.3	5.8	2.5	6.1	3.3	0.8	6.5	2.9	0.4	-	-	-	-	-	-
Modulus x 10^3 psi	353	305	310	381	420	355	757	479	401	236	191	189	173	132	131
Stability 1st	37	39	34	32	34	31	40	32	26	25	25	24	24	20	22
Stability 2nd	49	49	47	44	51	34	55	45	28	-	-	-	-	-	-
Indi. Tens. Test: Dry	248	237	237	318	302	276	308	327	329	170	178	202	185	186	172
Indi. Tens. Test: Wet	127	143	170	160	196	231	157	224	257	111	154	170	124	168	161
Indi. Tens. Test: Index, %	51	60	72	50	65	84	51	69	78	65	87	84	67	90	94
P 200 Batched	-	1.6	-	-	4.8	-	-	8.0	-	-	-	-	-	-	-
P 200 Extracted	-	2.9	-	-	5.8	-	-	8.2	-	-	-	-	-	-	-
Asphalt Extracted	-	5.4	-	-	5.3	-	-	5.5	-	-	-	-	-	-	-

Table 8. Core Data - Average Values and Standard Deviations

		CORE DATA		JOB MIX TOLERANCE (Class "B" Mix)	
		TOP LAYER	BASE LAYER	TOP LAYER	BASE LAYER
Thickness		1.7" ± .3	2.0" ± .5	-	-
Bulk S.G.		2.12 ± .04	2.14 ± .05	-	-
% Voids In Place		13.4 ± 2.5	11.5 ± 2.1	-	-
Modulus x 10^3 psi		197 ± 46	214 ± 37	-	-
GRADATION RANGES, % PASSING	3/4	96 - 100	98 - 100	95 - 100	
	1/2	69 - 95	84 - 96	81 - 96	
	3/8	50 - 86	66 - 87		
	1/4	31 - 68	51 - 74	57 - 69	
	10	14 - 33	22 - 38	22 - 30	
	40	7 - 16	11 - 17	8 - 16	
	200	3.5 - 7.6	5.2 - 7.6	3 - 7	
% A.C.		6.0 ± .7	7.0 ± .5	5.6-6.6	6.2-7.2
RECOVERED ASPHALT PROPERTIES AASHTO-T-170	Penetration At 70°F(21°C) (cm/100)	55 ± 8	58 ± 5	---	
	Kinematic Viscosity At 275°F(135°C) (cs.)	392 ± 69	396 ± 21	---	
	Absolute Viscosity At 140°F(60°C) (Poise)	2379 ± 669	2090 ± 247	---	

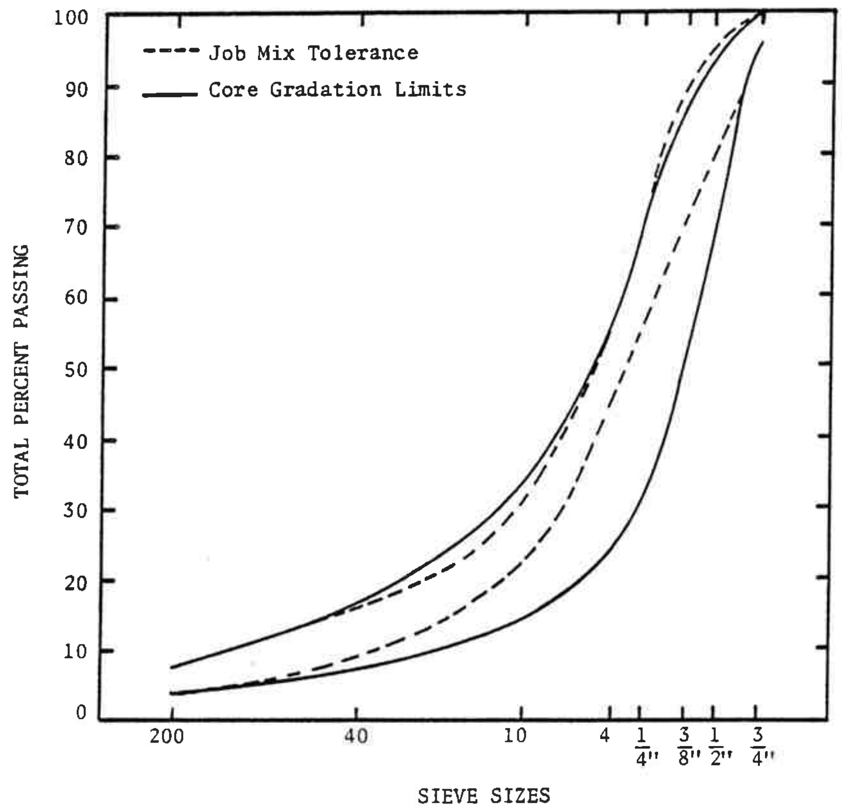


Figure 3. Core Gradation, Top Layer

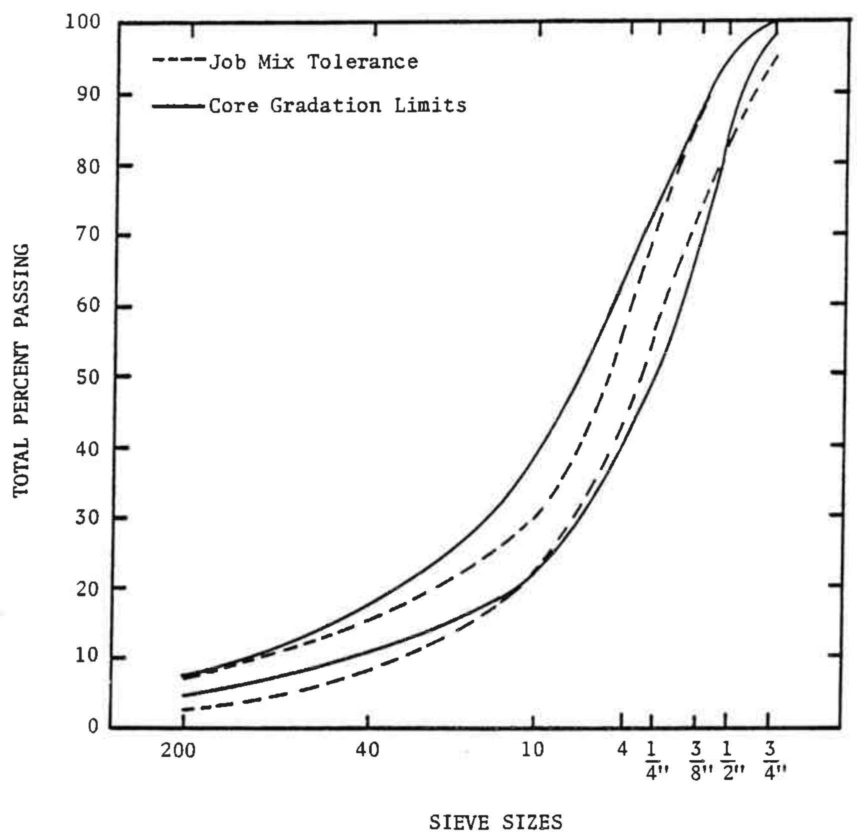


Figure 4. Core Gradation, Base Layer

indicate a softer consistency than expected for an AR4000 grade, which indicated a low mixing temperature, less than 300°F (149°C) to 325°F (163°C). The average asphalt content is 6.0 percent for the surfacing course and 7.0 for the base course, which is relatively close to the design values: 6.1 percent for the surfacing course and 6.7 percent for the base course. However, as indicated by the standard deviation, the variation in mix asphalt content exceeds the job mix tolerance.

In summary, analysis of the core samples brought the following information:

1. Excess variability in aggregate gradation and asphalt content;
2. Excess amount of fines (excess soft material in aggregate);
3. Inadequate asphalt coating of aggregate, and
4. Low recovered asphalt consistency (probably related to a low mixing temperature);
5. Low mix density (or high voids).

These data were used to establish the laboratory program 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 are tested as compacted, and eight samples are tested after conditioning.* Table 9 gives the flow chart for the test program followed. The principal variables studied included:

(1) Mix level of compaction: 100% - 97% - 92% - 90%

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

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

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 97%, while a 92% compaction standard mix was selected to study the influence of the amount of fines, asphalt content and aggregate quality. Detail of the combination of variables used in this analysis are shown in Table 10.

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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 three hours.
6. Run the mechanical property tests, along the same sample axis as the as-compacted modulus was measured (Step 1).

Table 9. Test Program - OSU

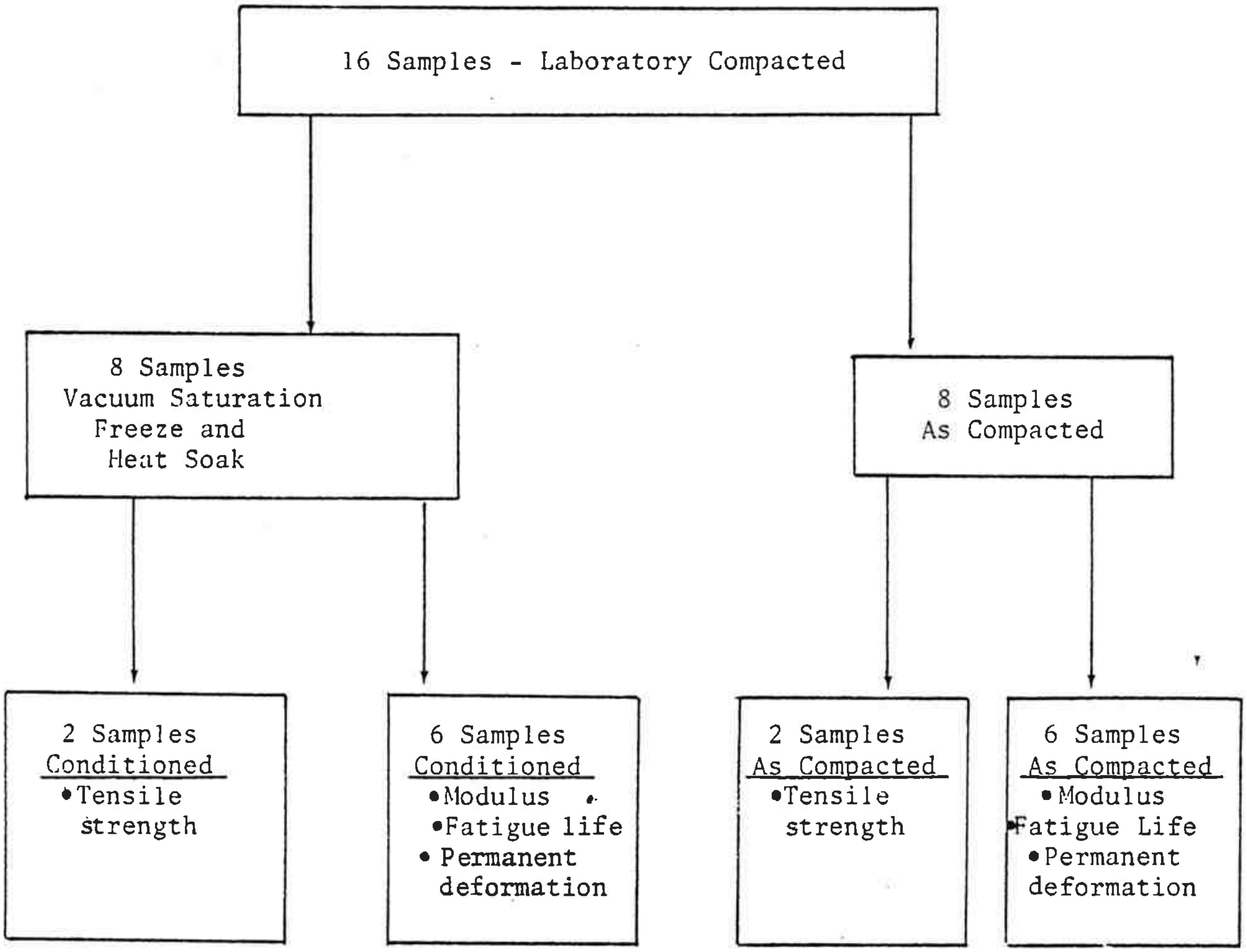


Table 10. Range of Mix Variables Considered in This Study
(Crossed Boxes)
North Oakland - Sutherlin Project

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					X				
1st Compaction					X				
95 Blows at 100 psi 500 psi Leveling Load	X		X	X	X	X	X		X
30 Blows at 100 psi 300 psi Leveling Load					X				

Test Equipment

Figure 5 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" (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 6).

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):

$$M_R = \frac{P}{\Delta H \times h} \left[.2692 + .9974\nu \right] \quad (1)$$

where

M_R = Resilient modulus, psi

ΔH = Horizontal elastic tensile deformation, inches

P = Dynamic load, pounds

h = Sample thickness, inches

ν = Poisson's ratio

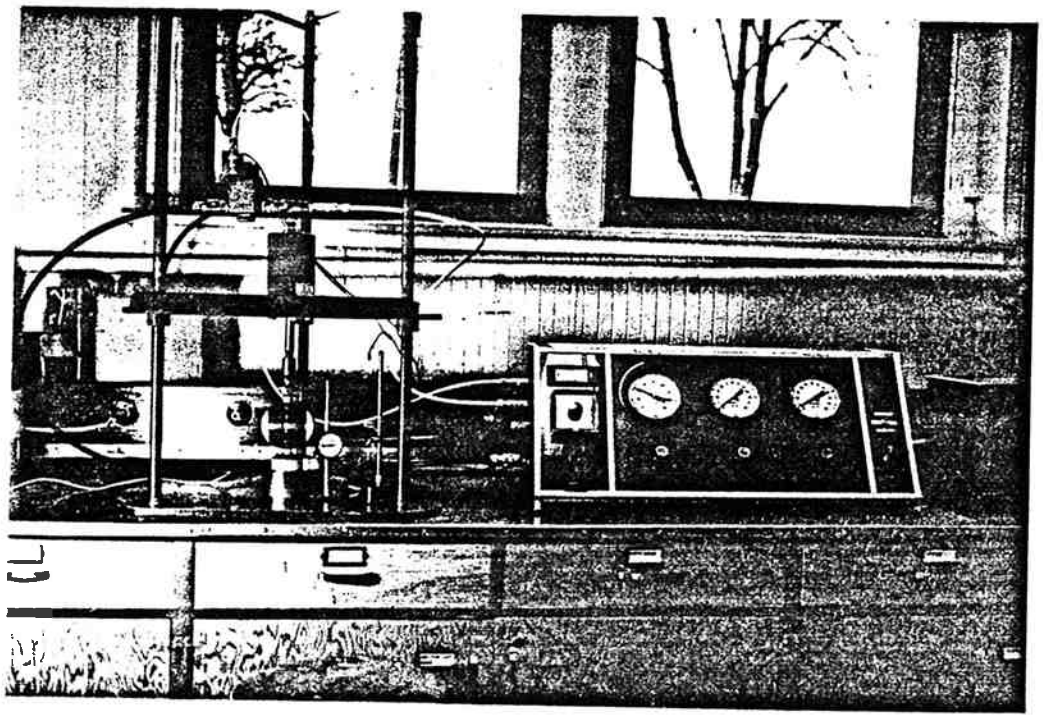


Figure 5. Diametral Test Apparatus

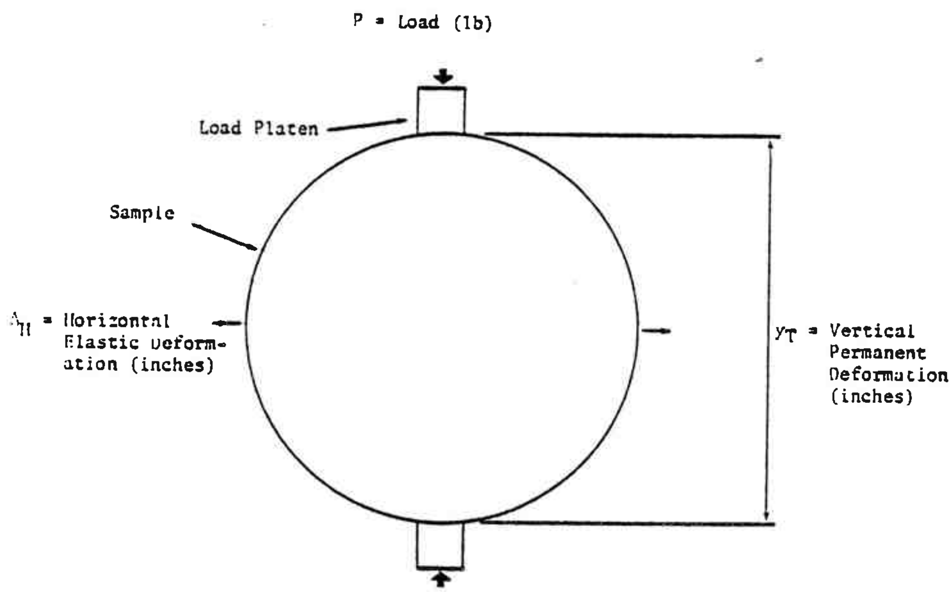


Figure 6. Diametral Test - Variables Recorded

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

$$M_R = \frac{P \times .6183}{\Delta H \times h} \qquad (2)$$

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):

$$N_f = K(\frac{1}{\varepsilon_t})^m \qquad (3)$$

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):

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

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:

$$\varepsilon_t = \Delta H \times .5203 \qquad (5)$$

The number of load repetitions to fatigue failure was defined as the number of repetitions required to get a vertical crack approximately 1/4" (.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 plan perpendicular to the plane formed by the load platen (see Figure 7). 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 8).

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

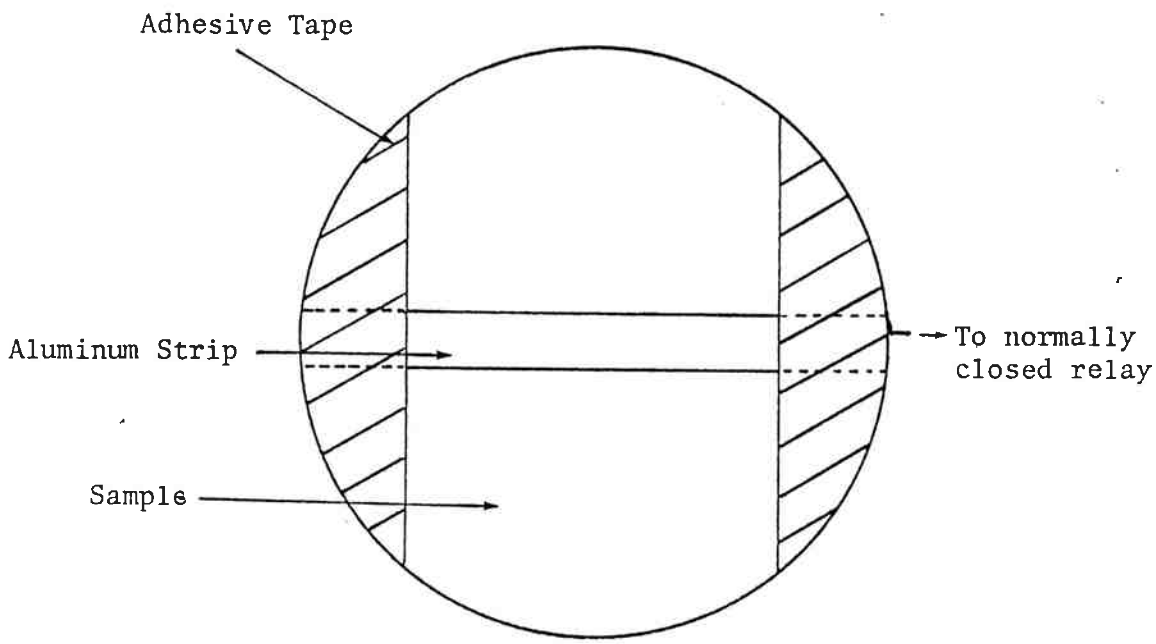
If the Poisson's ratio is assumed constant and equal to .35, equation (6) becomes:

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

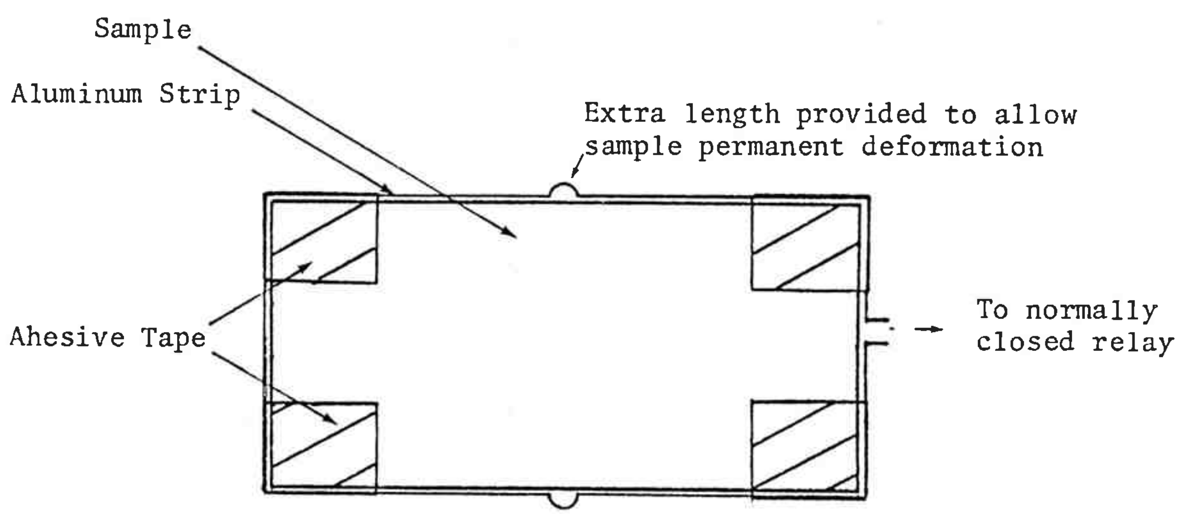
Resilient modulus, fatigue and permanent deformation models and tensile strength values have been determined for each set of variables considered in this study. The significance of these results and their correlation with other mix properties are developed later in this chapter.

Resilient Modulus Data

The resilient modulus of the as compacted samples are presented in Table 11, along with the mix Bulk Specific Gravity and percent voids. The influence of the Bulk Specific Gravity on the resilient modulus obviously predominates the other factors considered in this study. Plotted on Figure 9, the relationship between the as compacted mix resilient modulus and the Bulk Specific Gravity appears linear and affected by the asphalt and fines content only when both the asphalt and fines exceed the design optimum (example: 7% asphalt - 10% fines). Excess fines and asphalt results in a decrease in



(a) Side view



(b) Top view

Figure 7. Schematic View of the Automatic Shut-Off Device for Fatigue Testing

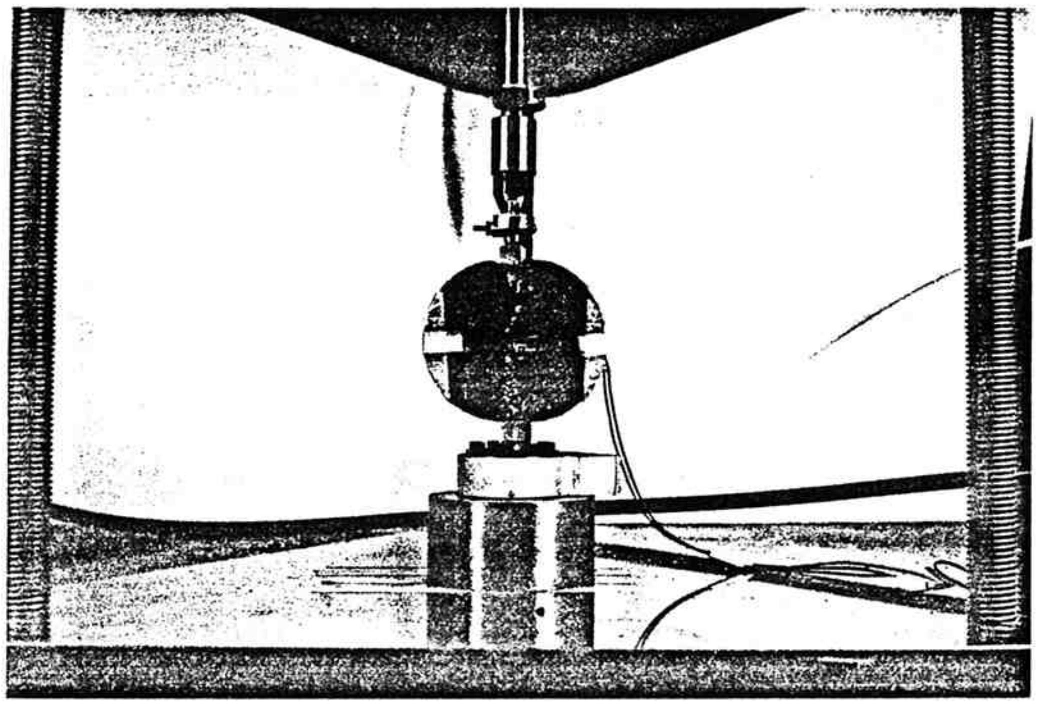


Figure 8. Sample at End of Fatigue Test

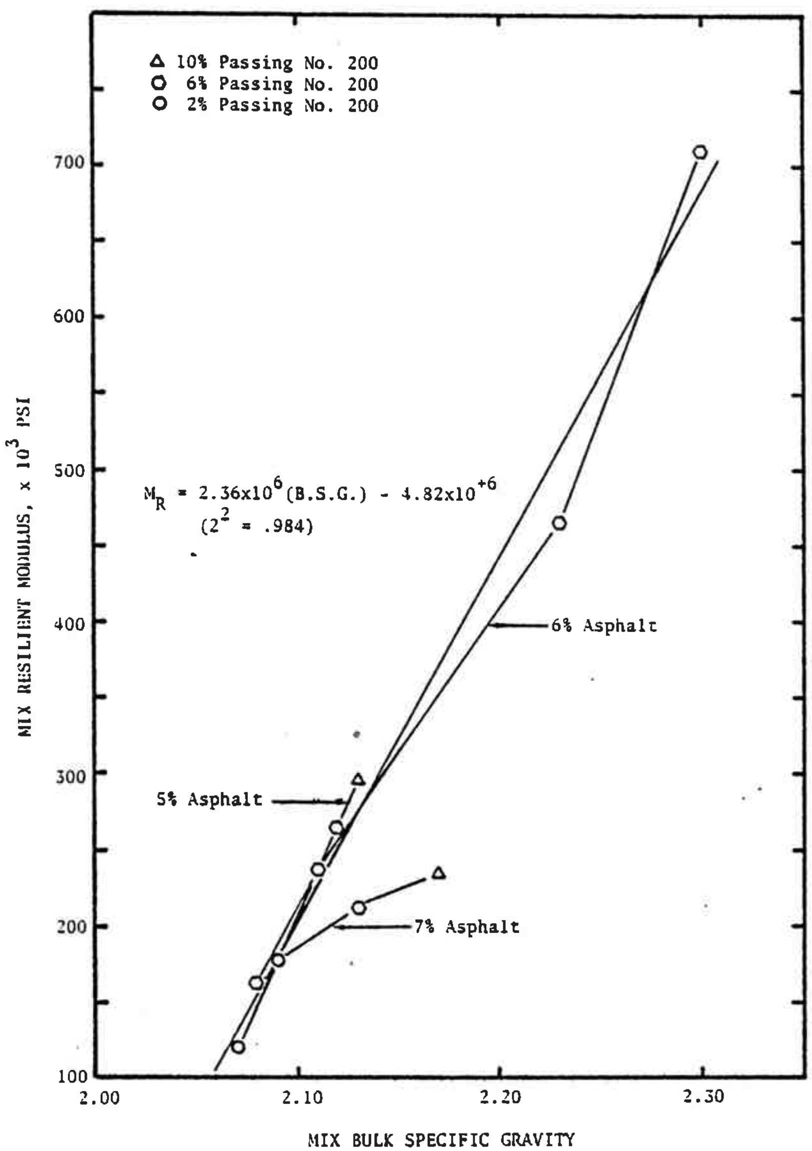


Figure 9. Influence of Bulk Specific Gravity on Resilient Modulus, As Compacted Samples.

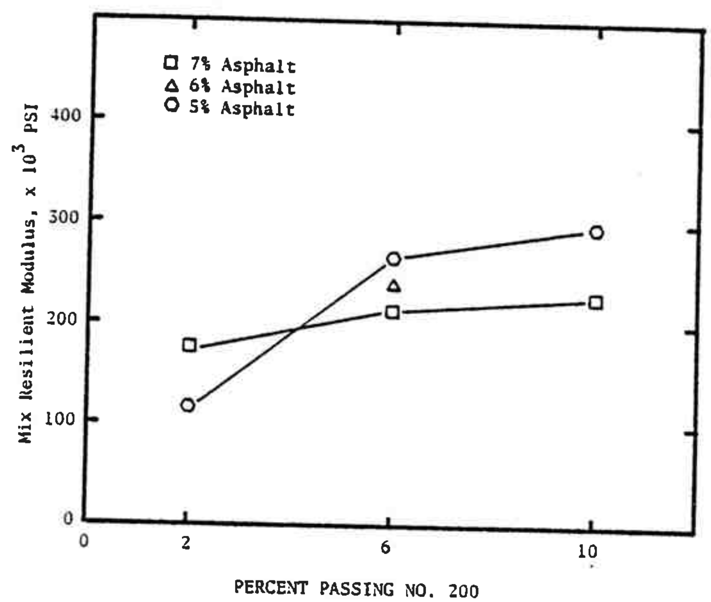


Figure 10. Influence of Amount of Fines on Resilient Modulus - As Compacted Samples.

the mix resilient modulus. Linear regression analysis of the relationship between resilient modulus and Bulk Specific Gravity gave a coefficient of correlation equal to .984, when the two points corresponding to the 7 percent asphalt, 6 and 10 percent fines samples are not included in the calculation. The influence of the amount of fines and asphalt content at a constant level of compaction is illustrated on Figure 10, which shows that mixes composed of 5% asphalt have higher modulus than mixes with 6 or 7% asphalt. However, decreasing the asphalt and fines content causes a substantial decrease in the mix stiffness. Similar trends can be observed with the conditioned samples. Moduli values of conditioned samples are presented in Table 11b together with their percentage of retained stiffness (compared with the mix modulus measured before sample conditioning). These data indicate wide variations in the percentage of retained stiffness, with no clear correlation with the amount of asphalt and fines or with the mix Bulk Specific Gravity. The relationship between conditioned modulus and the mix Bulk Specific Gravity (Figure 11) presents slightly more scatter than for the as-compacted modulus, but shows predominant effect of the Bulk Specific Gravity over the other factors, at least within the range of asphalt and amount of fines considered in this study. Linear regression analysis gave a coefficient of correlation equal to .885, all points being included. The samples prepared with 7 percent asphalt content have been slightly affected by the conditioning process, but the samples prepared with 5 percent asphalt content show a significant loss in stiffness after conditioning. Increasing the asphalt content therefore improves the mix resistance to water. A peak value of the mix modulus at 6 percent passing the No. 200 sieve for the 5 percent asphalt content mix seems to indicate that 6 percent fines is optimum when using 5 percent asphalt content. (see Fig. 12)

In summary, the mix modulus is primarily affected by the mix Bulk Specific Gravity. For a constant amount of passing the No. 200 and after conditioning, the mix modulus increases when the asphalt content increases. A peak modulus value appeared at 5 percent asphalt content for 6 percent passing the No. 200 sieve.

Fatigue Data

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

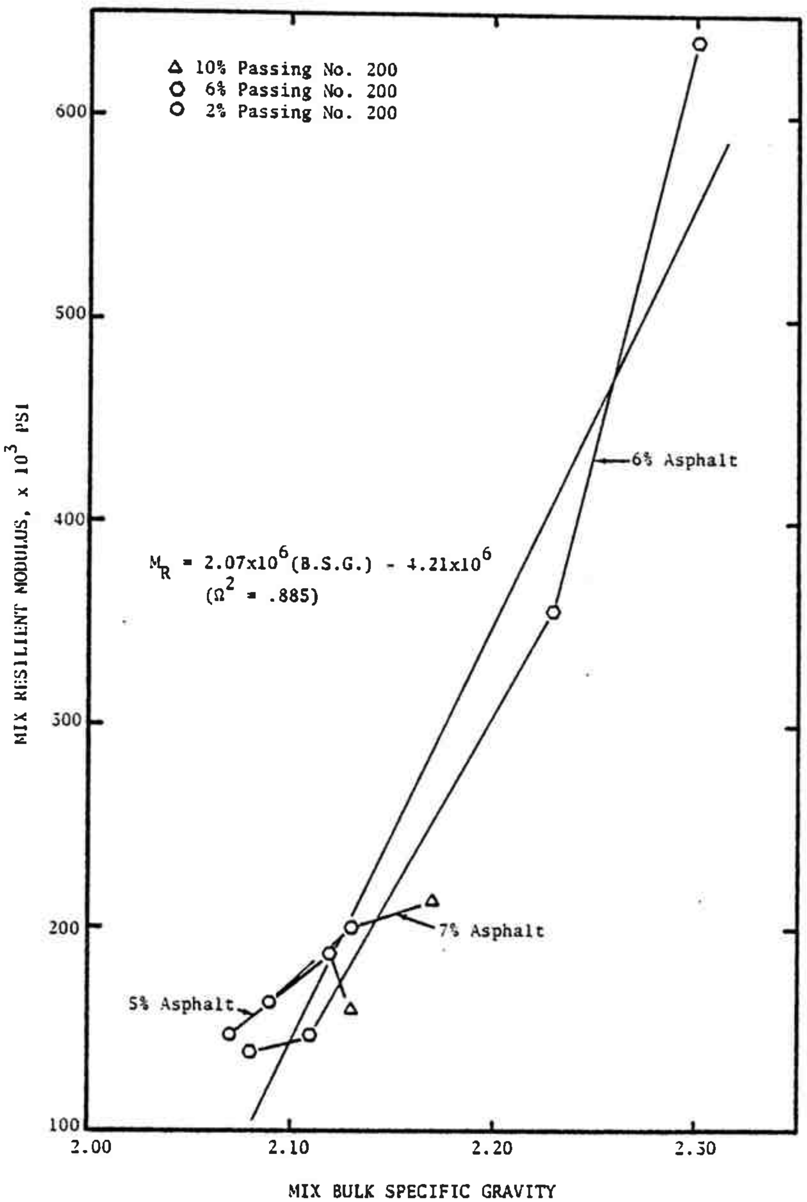


Figure 11. Influence of Bulk Specific Gravity on Resilient Modulus, Conditioned Samples.

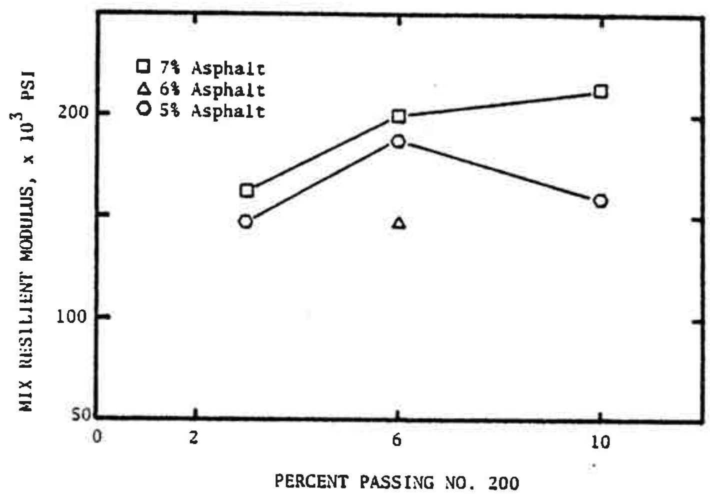


Figure 12. Influence of Amount of Fines on Resilient Modulus - Conditioned Samples.

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
100%					710,000 5.3 2.30				
97%					466,000 8.2 2.23				
92%	117,000 15.9 2.07		176,000 11.8 2.09	265,000 14.2 2.12	238,000 13.2 2.11	212,000 10.9 2.13	297,000 13.4 2.13		227,000 8.8 2.17
90%					163,000 14.4 2.08				

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

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
100%					638,000 89.9%				
97%					357,000 76.6%				
92%	148,000 126%		163,000 92.6%	188,000 70.9%	147,000 61.8%	200,000 94.3%	159,000 53.5%		213,000 93.8%
90%					139,000 85.9%				

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

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 microstrain. The coefficients K and m were then determined by linear regression analysis. Table 12a shows the as-compacted 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 13. The relationship m versus ℓn K follows the equation:

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

Linear regression run on the as-compacted data gave:

A_1 = .124

A_o = -.875

Coefficient of correlation: r^2 = .982

It can be deduced from this relationship between m and $\ell n(K)$ that fatigue curves, expressed in Number of Load Repetitions versus Mix Tensile strain, should intercept at a common point, called focus point (11, 12, 13). The coordinates of this focus point (ε_o, N_o) can be deduced from equation (8) and (9):

$$m = A_o + A_1 \ell n(K) \quad (10)$$

$$N_F = K(\varepsilon)^m \iff \ell n(N_F) = \ell n(K) + m\ \ell n(\varepsilon) \quad (11)$$

(11) is also true at the focus point

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

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

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

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

Coordinates of the as-compacted samples focus point gave:

$$\varepsilon_o = 316 \times 10^{-6}$$

$$N_o = 1.15 \times 10^3$$

Knowing the coordinates of the focus point, linear regression analysis were rerun for each set of samples, and fitted through the focus point. Table 13a gives the new K and m values (noted k' and m').

The same approach was followed for the conditioned test results. Table 12b gives the K and m values computed from the test results. The relationship found between K and m is:

$$m = .116 \ \ell n(K) - 1.01 \qquad (13)$$

The coordinates of the focus point for the conditioned samples are $\varepsilon_o = 184 \times 10^{-6}$ microstrain and $N_o = 5.97 \times 10^3$ load repetitions. The corrected K' and m' for the conditioned samples are given in Table 13b.

The effect of asphalt content, 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 fatigue curves for 6% asphalt content and 6% passing the No. 200 sieve are presented in Figure 14a for the as-compacted samples and Figure 14b for the conditioned samples. As-compacted and conditioned results show a substantial decrease in fatigue life when the mix density drops from 97% to 92%. The influence of asphalt content is illustrated in Figure 15. The optimum asphalt content (6%) is noticeable with the conditioned samples. The as-compacted samples show a continuous increase in fatigure life when the asphalt content is increased.

The influence of the percent passing No. 200 on the mix fatigue life was studied at 5 percent asphalt content (Figure 16) and 7 percent asphalt content (Figure 17). At both asphalt contents the mix fatigue life decreases when the amount of fines is reduced to 2 percent. The optimum amount of fines is more apparent when using 5 percent asphalt in the mix, Figure 16a and 16b showing a longer fatigue life for the mix with 6 percent passing the No. 200 sieve.

The mix fatigue life is, therefore, primarily affected by the mix density. Optimum asphalt content (6%) and optimum percent passing No. 200 (6%) are both apparent in the test results of the conditioned samples.

Table 12a. 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
100%					$k=1.15 \times 10^{-7}$ $m=-2.95$ $\log k=-6.94$				
97%					$k=1.76 \times 10^{-8}$ $m=-3.10$ $\log k=-7.75$				
92%	$k=2.13 \times 10^{-2}$ $m=-1.30$ $\log k=-1.67$		$k=6.28 \times 10^{-6}$ $m=-2.34$ $\log k=-5.48$	$k=7.98 \times 10^{-4}$ $m=-1.77$ $\log k=-3.10$	$k=4.99 \times 10^{-4}$ $m=-1.83$ $\log k=-3.30$	$k=1.95 \times 10^{-4}$ $m=-2.03$ $\log k=-3.71$	$k=3.65 \times 10^{-7}$ $m=-2.58$ $\log k=-6.44$		$k=3.75 \times 10^{-5}$ $m=-2.21$ $\log k=-4.43$
90%					$k=3.28 \times 10^{-5}$ $m=-2.08$ $\log k=-4.48$				

* N_F : Number of Load Repetitions to Failure
ε_T : Mix Elastic Tensile Strain
k,m: Regression Constants

Table 12b. 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
100%					$k=6.67 \times 10^{-10}$ $m=-3.51$				
97%					$k=8.92 \times 10^{-5}$ $m=-2.22$				
92%	$k=1.28 \times 10^{-1}$ $m=-1.21$		$k=7.52$ $m=-.821$	$k=4.36 \times 10^{-6}$ $m=-2.38$	$k=1.86 \times 10^{-2}$ $m=-1.46$	$k=3.00 \times 10^{-4}$ $m=-1.95$	$k=1.43 \times 10^{-3}$ $m=-1.76$		$k=4.99 \times 10^{-7}$ $m=-2.67$
90%					$k=2.88 \times 10^{-5}$ $m=-2.18$				

* N_F : Number of Load Repetitions to Failure
ε_T : Mix Elastic Tensile Strain
k,m: Regression Constants

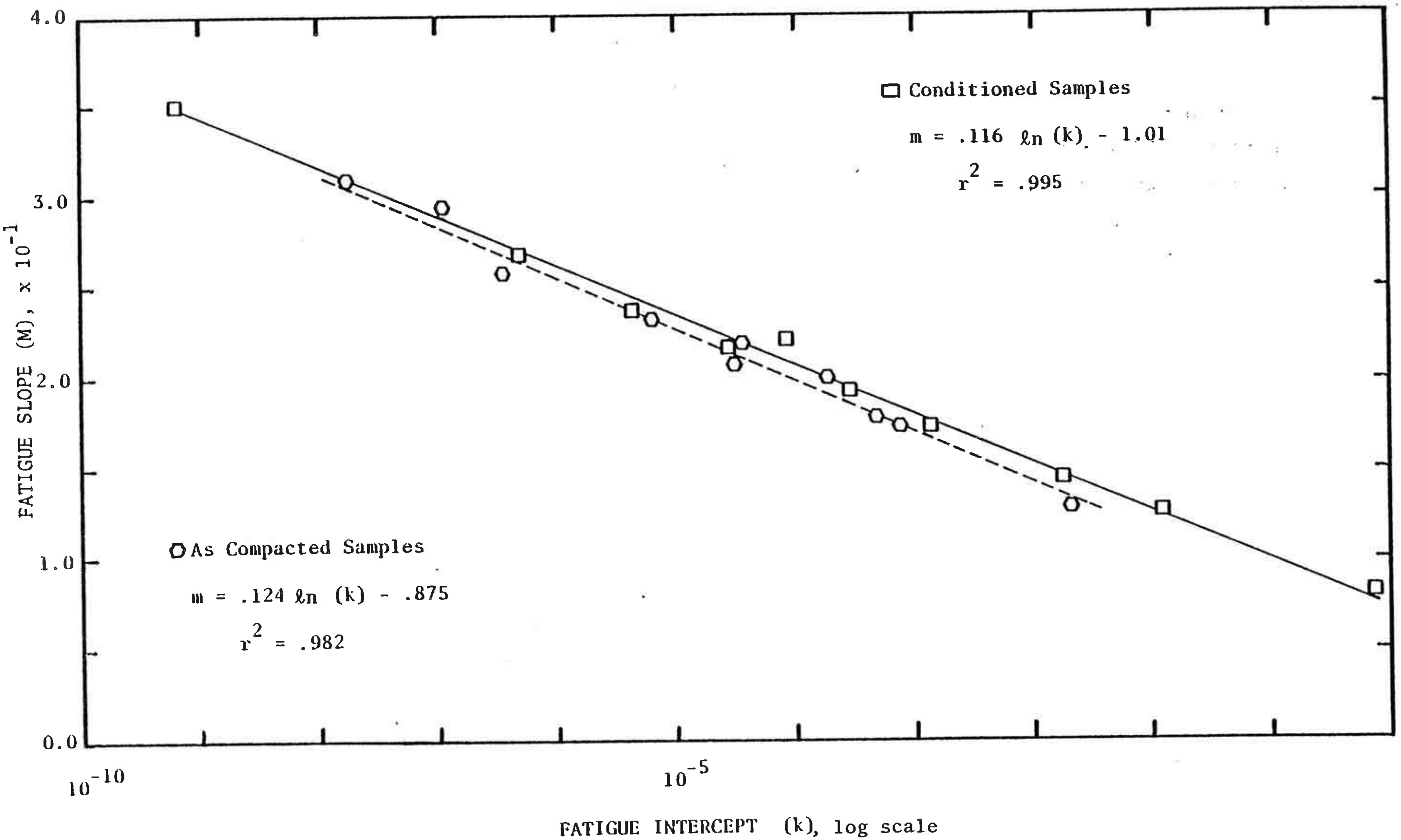


Figure 13. Relationship Between k and m.

Table 13a. Corrected 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
100%					$k'=1.06\times10^{-9}$ $m'=-3.44$				
97%					$k'=8.59\times10^{-9}$ $m'=-3.18$				
92%	$k'=4.77\times10^{-1}$ $m'=-.966$		$k'=2.61\times10^{-5}$ $m'=-2.18$	$k'=3.98\times10^{-4}$ $m'=-1.85$	$k'=1.98\times10^{-4}$ $m'=-1.93$	$k'=1.79\times10^{-6}$ $m'=-2.52$	$k'=5.24\times10^{-4}$ $m'=-1.81$		$k'=6.00\times10^{-7}$ $m'=-2.65$
90%					$k'=2.04\times10^{-3}$ $m'=-1.64$				

* N_f : Number of Load Repetitions to Failure

ε_T : Mix Elastic Tensile Strain

k', m': Regression Constants, with $m' = .124\ I_n(k') = .875$

Table 13b. Corrected 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
100%					$k'=9.72\times10^{-12}$ $m'=-3.96$				
97%					$k'=3.94\times10^{-10}$ $m'=-3.53$				
92%	$k'=8.57$ $m'=-.761$		$k'=1.73\times10^{-1}$ $m'=-1.22$	$k'=1.58\times10^{-3}$ $m'=-1.76$	$k'=5.38\times10^{-5}$ $m'=-2.15$	$k'=6.37\times10^{-4}$ $m'=-1.87$	$k'=3.89\times10^{-3}$ $m'=-1.66$		$k'=6.46\times10^{-6}$ $m'=-2.40$
90%					$k'=3.47\times10^{-3}$ $m'=-1.67$				

* N_f : Number of Load Repetitions to Failure

ε_T : Mix Elastic Tensile Strain

k', m': Regression Constants, with $m' = .116\ I_n(k') = 1.01$

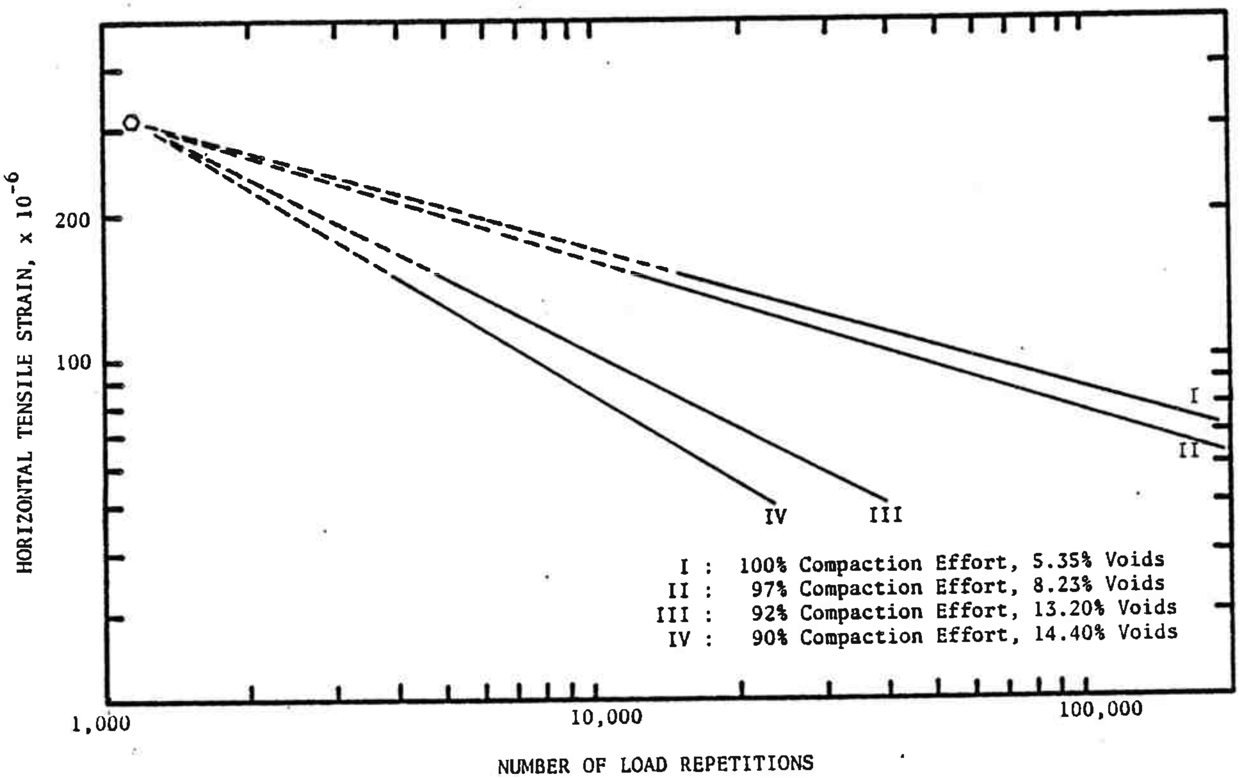


Figure 14a. Influence of Mix Density on Fatigue Life, As Compacted Samples.
6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt

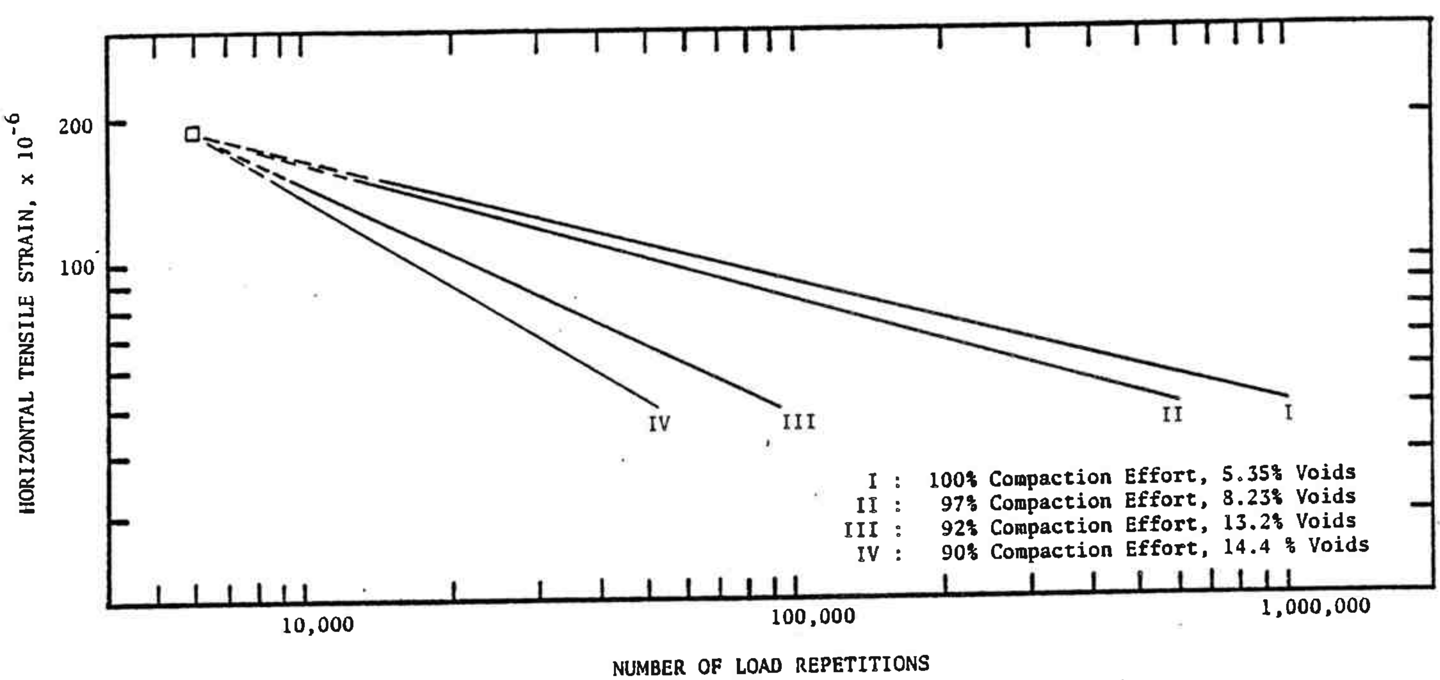


Figure 14b. Influence of Mix Density on Fatigue Life, Conditioned Samples.
6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt

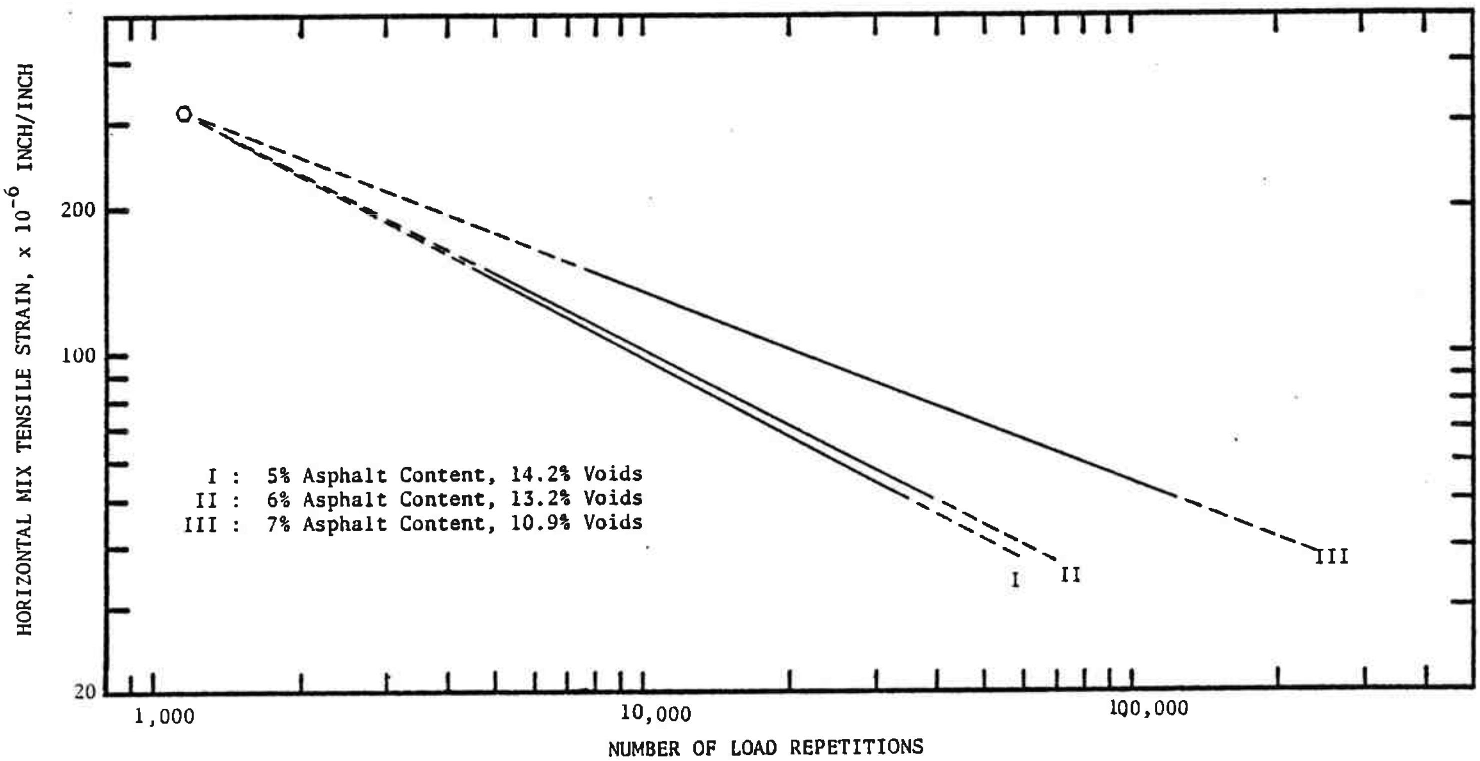


Figure 15a. Influence of Asphalt Content on Fatigue Life, As Compacted Samples
6% Passing No. 200 - 25% Passing No. 10 - 92% Compaction

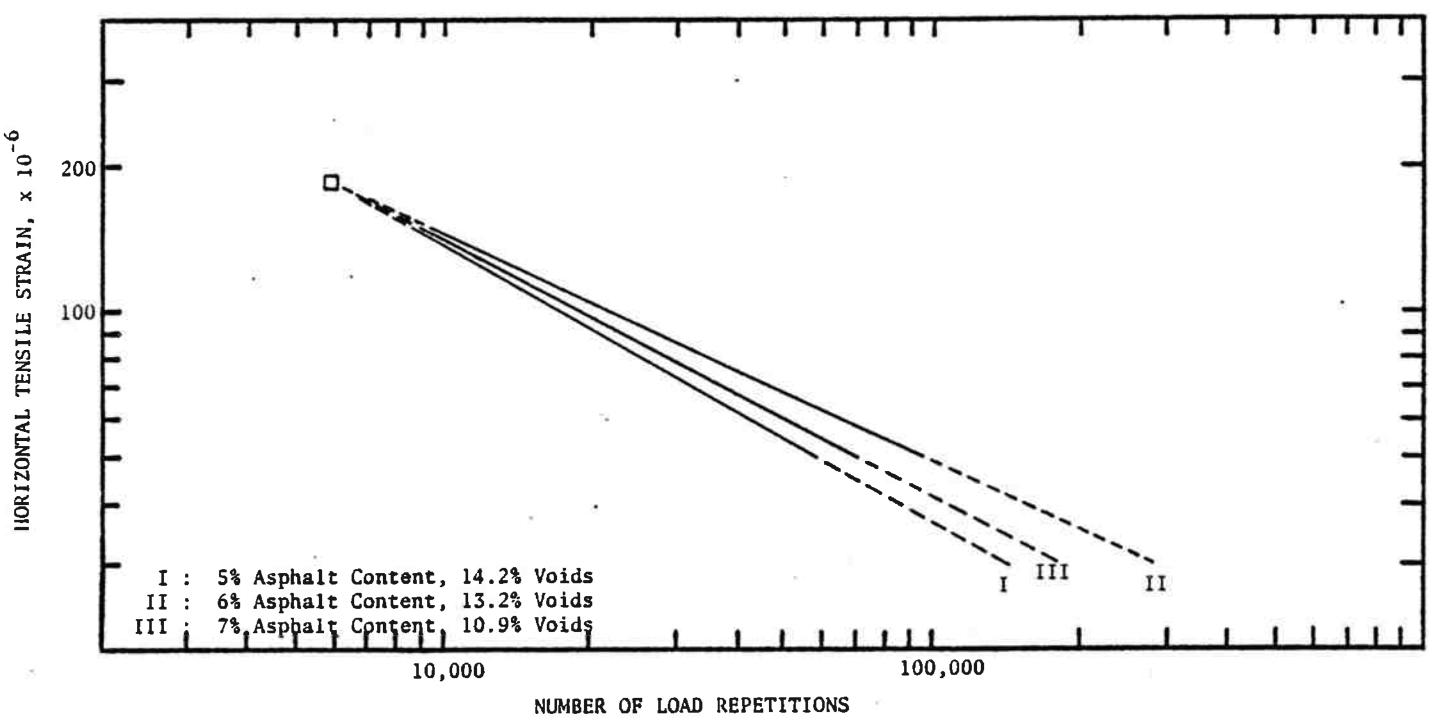


Figure 15b. Influence of Asphalt Content on Fatigue Life, Conditioned Samples
6% Passing No. 200 - 25% Passing No. 10 - 92% Compaction

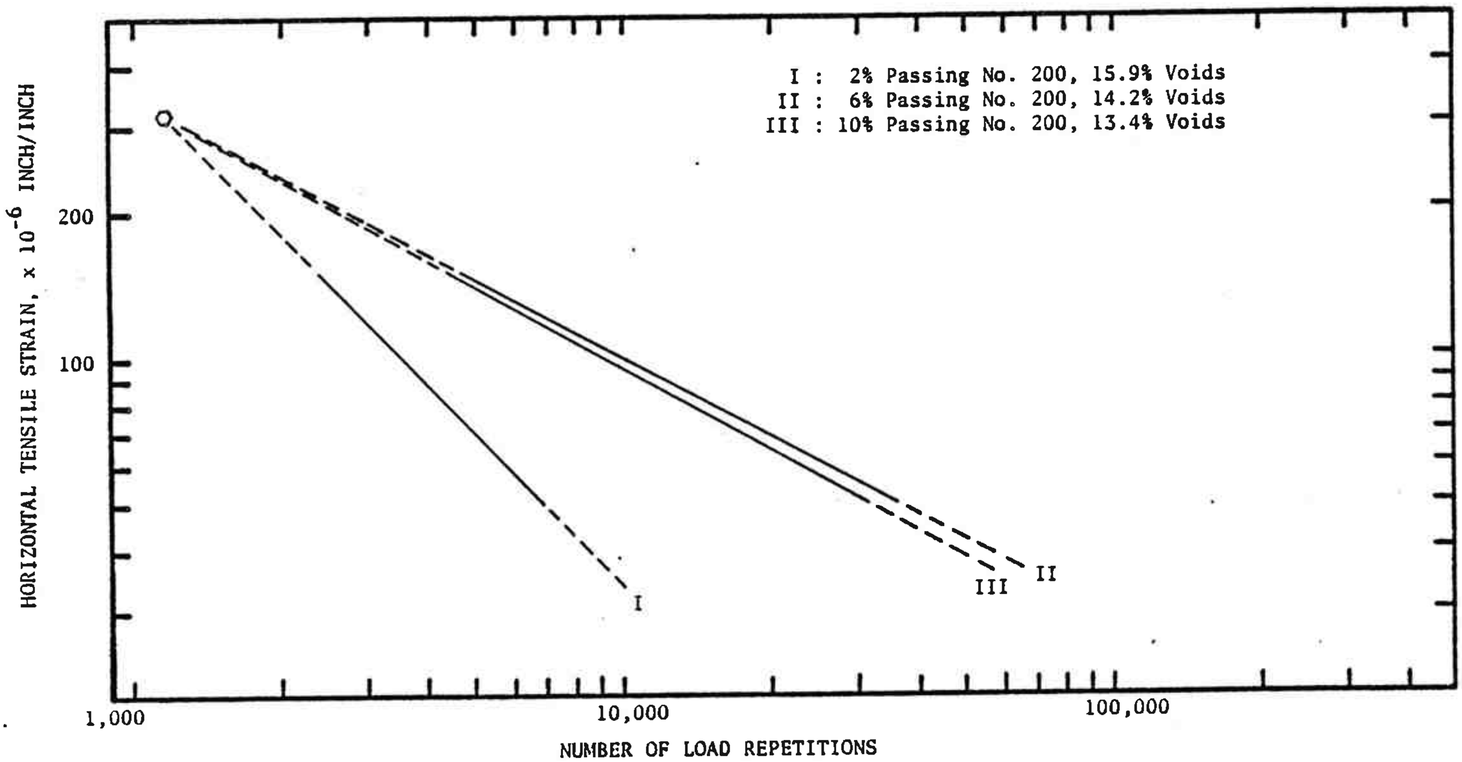


Figure 16a. Influence of Passing No. 200 on Fatigue Life, As Compacted Samples
5% Asphalt Content - 92% Compaction

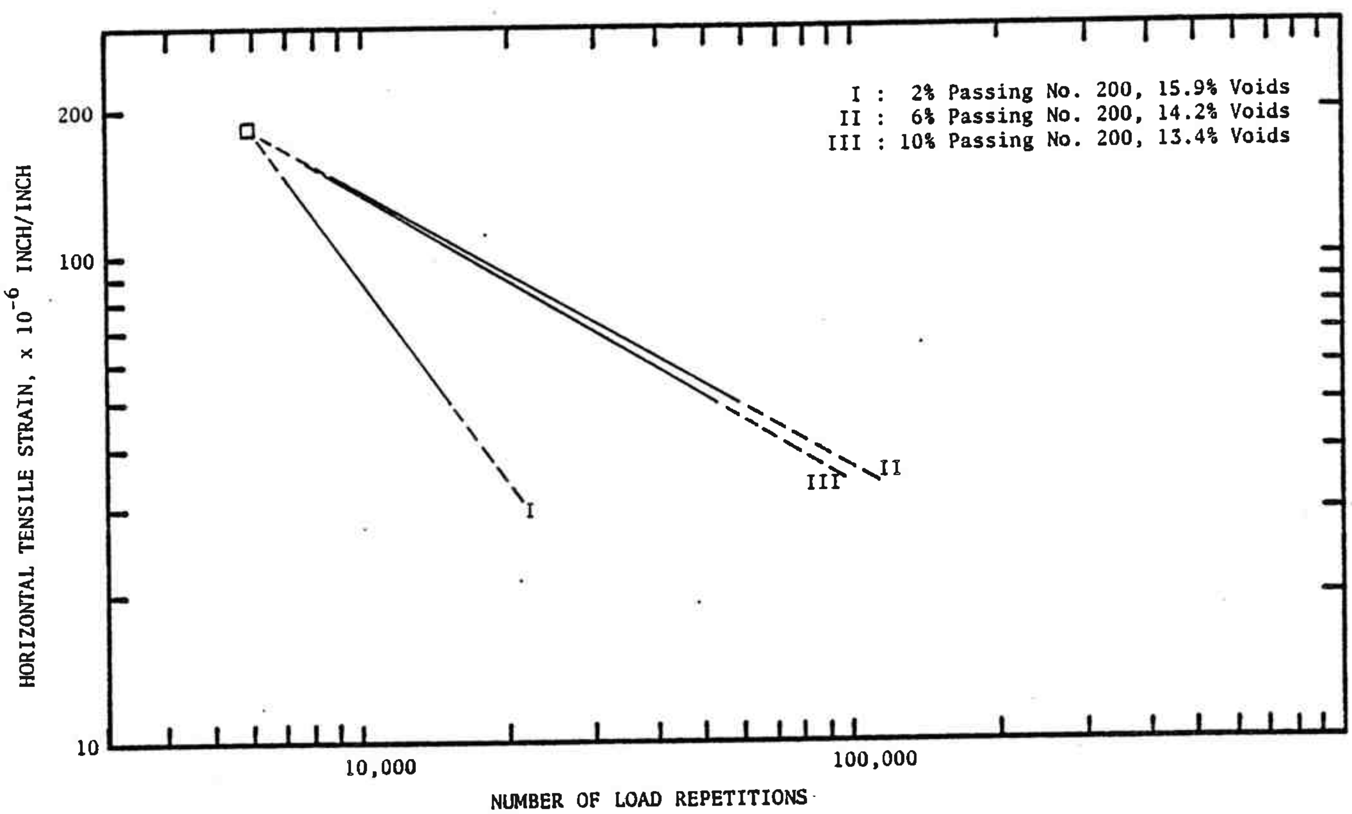


Figure 16b. Influence of Passing No. 200 on Fatigue Life, Conditioned Samples.
5% Asphalt - 92% Compaction

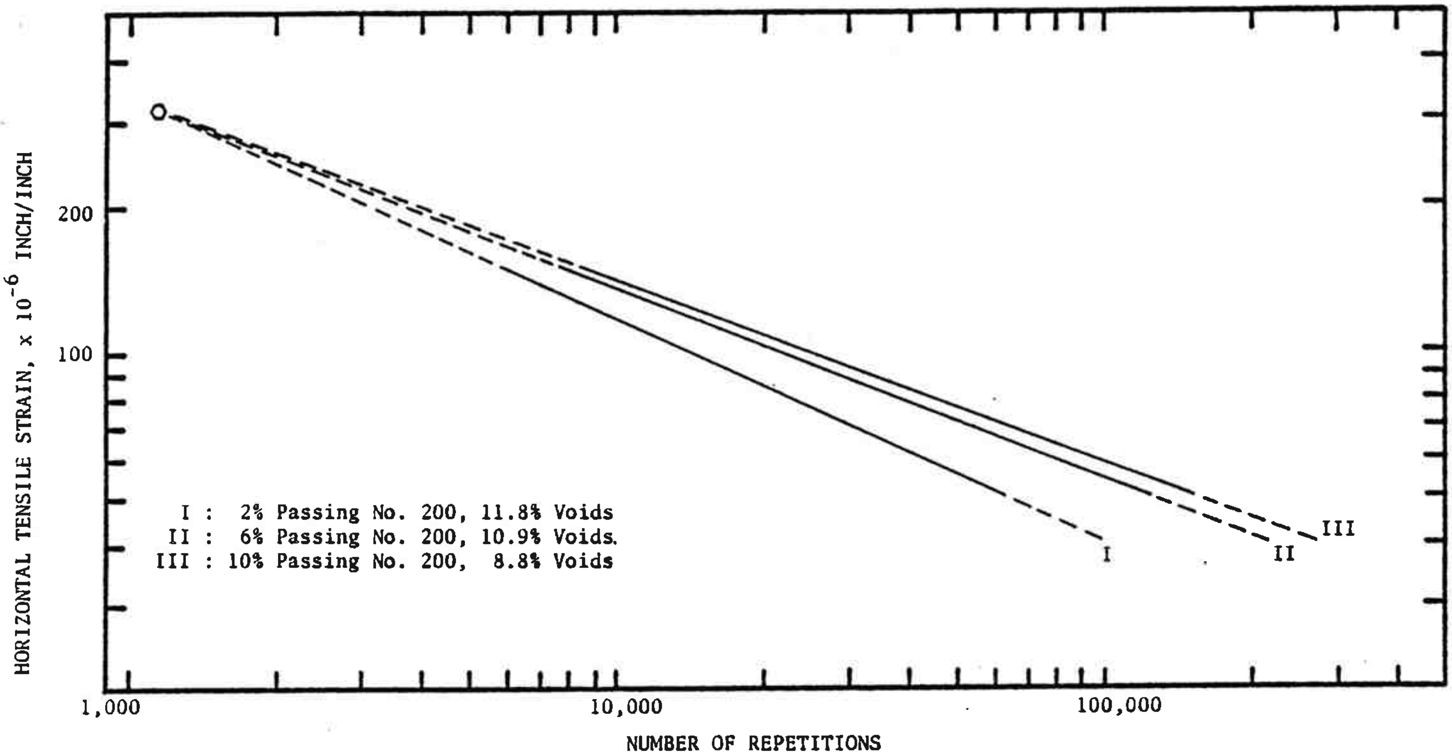


Figure 17a. Influence of Passing No. 200 on Fatigue Life, As Compacted Samples. 7% Asphalt - 92% Compaction.

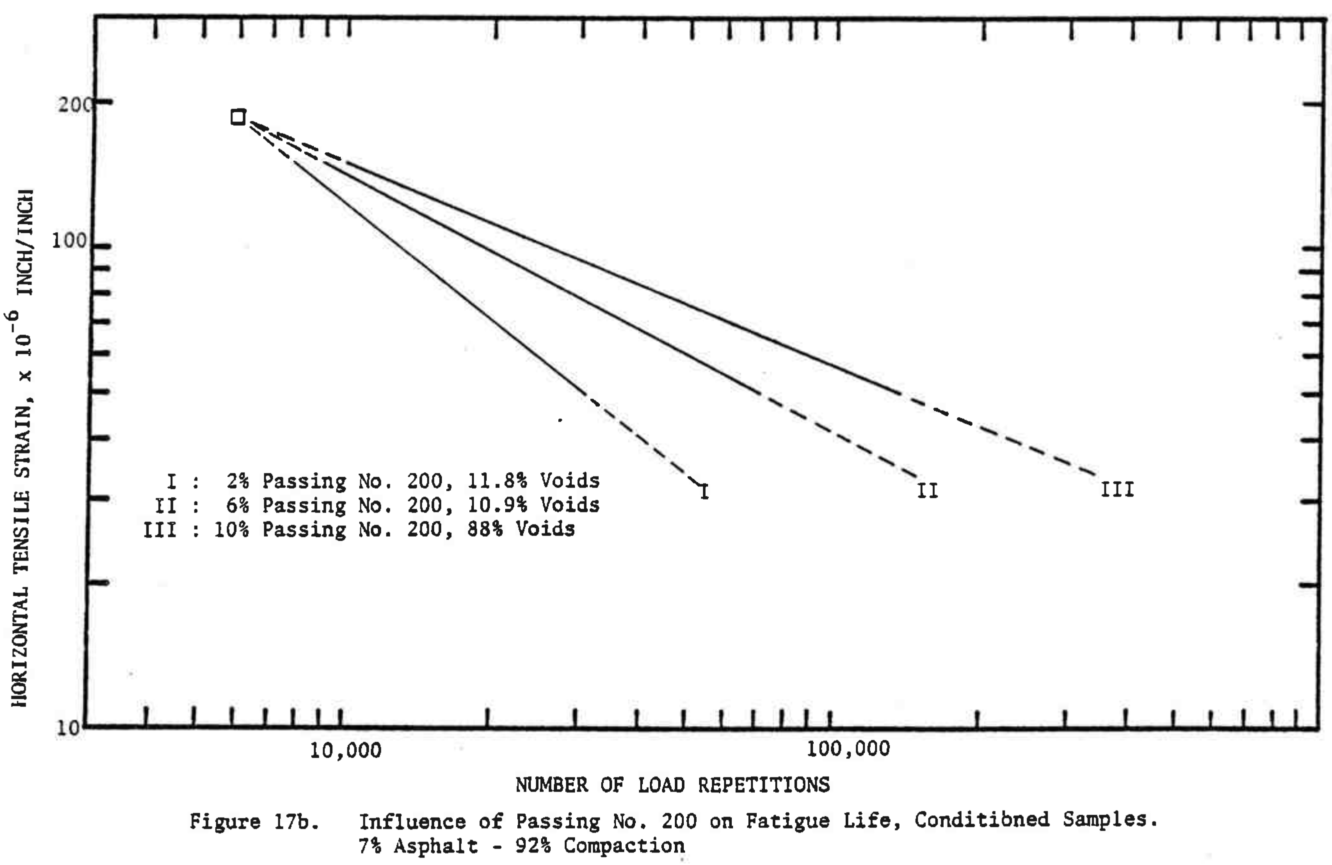


Figure 17b. Influence of Passing No. 200 on Fatigue Life, Conditibned Samples. 7% Asphalt - 92% Compaction

Permanent Deformation Data

Vertical compressive permanent deformation was recorded during fatigue testing using a dial gauge. Readings were taken along a logarithmic scale until failure of the sample. The vertical permanent strain was calculated from the vertical permanent deformation (4), according to:

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

where

ε_c = Compressive permanent strain

μ_t = Total vertical deformation, inches

ν = Poisson's ratio

For a Poisson's ration of .35, equation (14) becomes:

$$\varepsilon_c = \mu_t \times .1485$$

A typical plot of ε_c versus number of load repetitions at different loads is shown in Figure 18 for the mix with 6% passing No. 200, 25% passing No. 10, 6% asphalt and 92% 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 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 = 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 14. Non-consistent values (early readings and reading close to sample failure) have not been included in the linear regression.

A recent study showed that the slope s in equation (15) is constant for a specific mix, whereas the intercept I is a function of the mix tensile

Table 14. Permanent Deformation Constants, As Compacted Samples
6% Passing No. 200 - 25% Passing No. 10
6% Asphalt Content - 92% Compaction

SAMPLE	INITIAL TENSILE STRAIN (INCH/INCH)	PERMANENT DEFORMATION CONSTANTS		
		I	S	r^2
I	65.27×10^{-6}	4.32×10^{-5}	.691	.918
II	51.73×10^{-6}	1.55×10^{-4}	.449	.997
III	124.54×10^{-6}	1.49×10^{-4}	.612	.988
IV	155.52×10^{-6}	1.31×10^{-4}	.653	.999
V	106.36×10^{-6}	1.42×10^{-4}	.551	.996
VI	78.10×10^{-6}	1.35×10^{-4}	.584	1.00

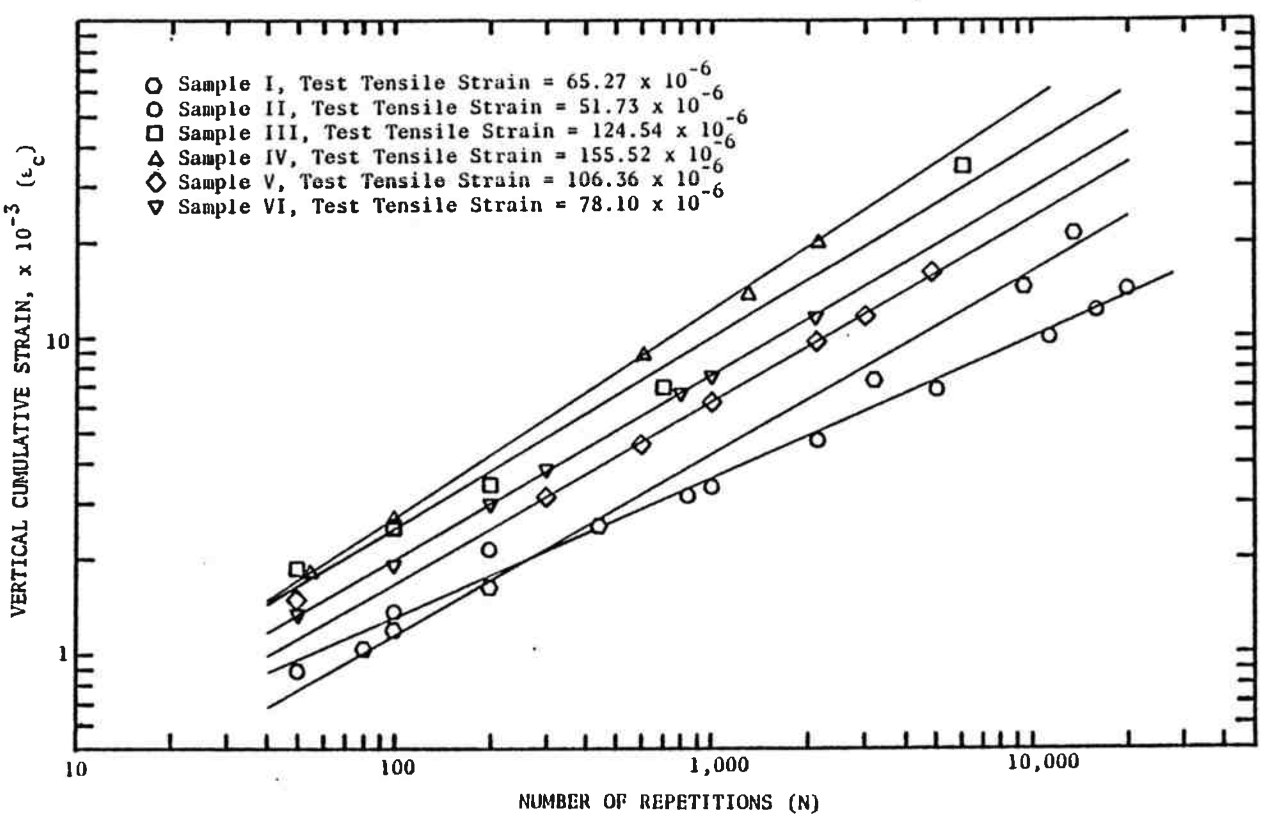


Figure 18. Permanent Deformation Results for Six Samples.

strain ε_t (10). Table 14 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 92% compaction.

A direct plot of the I values shown in Table 14 versus the corresponding tensile strain indicates no correlation (Figure 19), which can be explained by the nonuniformity of the computed slope s (Table 14). The linear regressions were therefore rerun using a fixed value for the slope (equal to the average of the slope values shown in Table 14: 0.445). Plotted in Figure 20, the new relationship I versus ε_t appears more consistent and linear. Using the data from the first four samples (ε_t and I with a constant slope S), a linear regression was run, giving the following equation:

$$I = 1.26\ (\varepsilon_t) - 7.31 \times 10^{-7} \qquad (17)$$

The samples prepared with 6% passing No. 200, 25% passing No. 10, 6% asphalt and compacted at 92% are then characterized by a permanent deformation expressed as follows:

$$\varepsilon_c = [1.26\ (\varepsilon_t) - 7.31 \times 10^{-7}]\ (N)^{.590} \qquad (18)$$

where

ε_c = Compressive permanent strain
ε_t = Horizontal tensile strain
N = Number of load repetitions

The same approach was used for all samples. The results are shown in Table 15a for the as-compacted samples and Table 15b 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. The results are shown in TAble 15c in the as-compacted samples and Table 15d for the conditioned samples. Figure 21a (as-compacted samples) and 21b (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

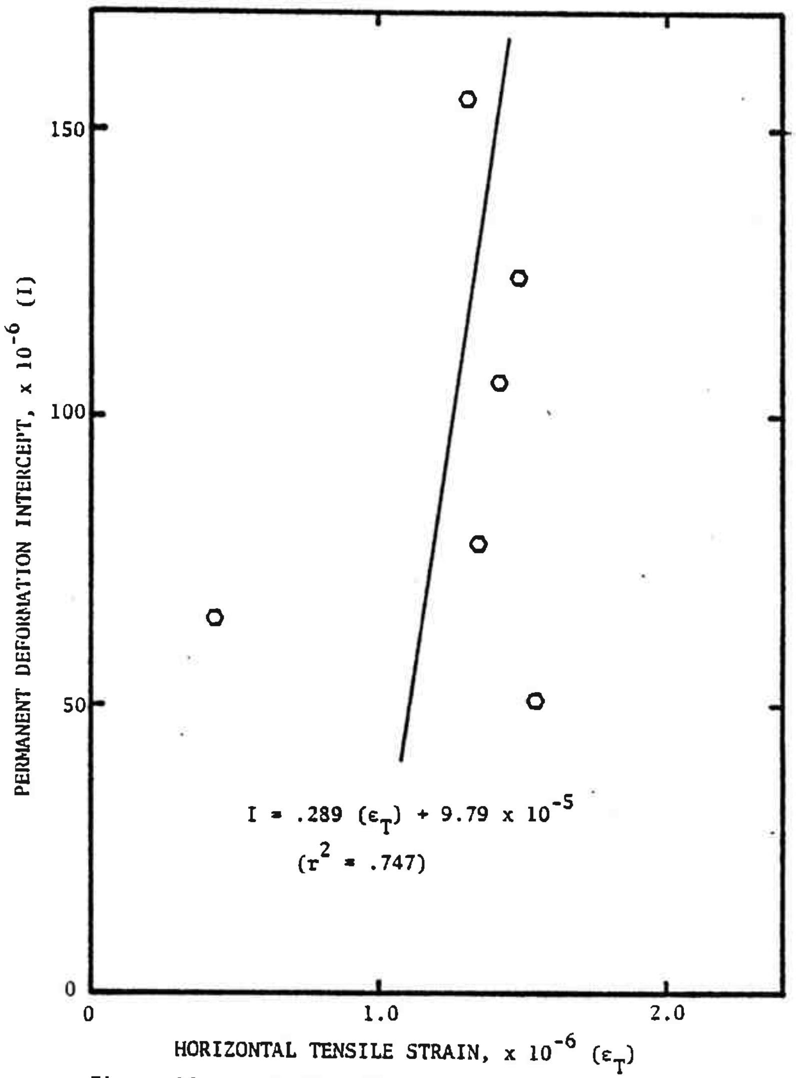


Figure 19. Relationship Between Permanent Deformation Intercept and Horizontal Tensile Strain.
6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt Content - 92% Compaction

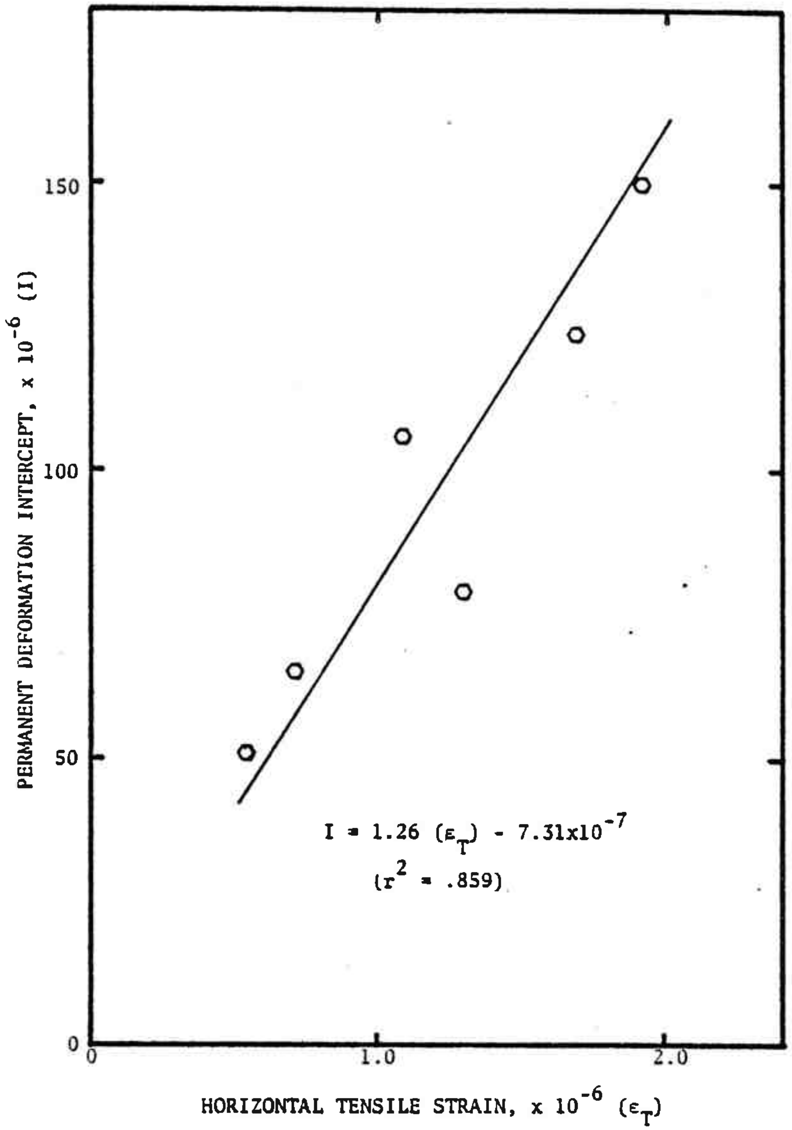


Figure 20. Relationship Between Permanent Deformation Intercept and Horizontal Tensile Strain for a Constant Slope = .445
6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt Content - 92% Compaction

Table 15a. Permanent Deformation Data, As Compacted Samples

I: Intercept S: Slope ε_T: Mix Tensile Strain

		LEVEL OF COMPACTION			
		100%	97%	92%	90%
10% PASSING No.200	ASPHALT CONTENT, % 7			$I=2.21(\varepsilon_T)-6.72\times10^{-6}$ $S=.481$	
	6				
	5			$I=1.53(\varepsilon_T)+1.02\times10^{-5}$ $S=.544$	
6% PASSING No.200	ASPHALT CONTENT, % 7			$I=1.40(\varepsilon_T)+2.80\times10^{-5}$ $S=.514$	
	6	$I=3.12(\varepsilon_T)+3.45\times10^{-5}$ $S=.277$	$I=2.82(\varepsilon_T)-6.16\times10^{-6}$ $S=.362$	$I=1.26(\varepsilon_T)-7.31\times10^{-7}$ $S=.590$	$I=2.06(\varepsilon_T)+1.09\times10^{-5}$ $S=.569$
	5			$I=2.25(\varepsilon_T)-4.78\times10^{-5}$ $S=.541$	
2% PASSING No.200	ASPHALT CONTENT, % 7			$I=2.43(\varepsilon_T)-1.87\times10^{-5}$ $S=.516$	
	6				
	5			$I=1.89(\varepsilon_T)-4.73\times10^{-5}$ $S=.626$	

Table 15b. Permanent Deformation Data - Conditioned Samples

I: Intercept S: Slope ε_T: Mix Tensile Strain

		LEVEL OF COMPACTION			
		100%	97%	92%	90%
10% PASSING NO.200	ASPHALT CONTENT, % 7			$I=2.48(\varepsilon_T)-1.30\times10^{-5}$ $S=.519$	
	6				
	5			$I=1.46(\varepsilon_T)+1.25\times10^{-4}$ $S=.454$	
6% PASSING NO.200	ASPHALT CONTENT, % 7			$I=2.01(\varepsilon_T)+7.42\times10^{-6}$ $S=.538$	
	6	$I=3.41(\varepsilon_T)+1.39\times10^{-4}$ $S=.323$	$I=5.89(\varepsilon_T)-1.53\times10^{-4}$ $S=.385$	$I=1.73(\varepsilon_T)+7.35\times10^{-5}$ $S=.531$	$I=2.55(\varepsilon_T)-3.37\times10^{-5}$ $S=.547$
	5			$I=2.16(\varepsilon_T)+9.92\times10^{-6}$ $S=.512$	
2% PASSING NO.200	ASPHALT CONTENT, % 7			$I=1.38(\varepsilon_T)+3.59\times10^{-4}$ $S=.538$	
	6				
	5			$I=1.90(\varepsilon_T)-2.70\times10^{-5}$ $S=.565$	

Table 15c. Permanent Deformation Strain (εc) for a Mix Tensile Strain (εx) Equal to 100 Microstrains. As compacted Samples.

		LEVEL OF COMPACTION			
		100%	97%	92%	90%
10% Passing No. 200, Asphalt Content	7			$\varepsilon c = 2.14 \times 10^{-4}(N)^{.481}$	
	6				
	5			$\varepsilon c = 1.63 \times 10^{-4}(N)^{.544}$	
6% Passing No. 200, Asphalt Content	7			$\varepsilon c = 1.68 \times 10^{-4}(N)^{.514}$	
	6	$\varepsilon c = 3.47 \times 10^{-4}(N)^{.277}$	$\varepsilon c = 2.76 \times 10^{-4}(N)^{.362}$	$\varepsilon c = 1.25 \times 10^{-4}(N)^{.590}$	$\varepsilon c = 2.17 \times 10^{-4}(N)^{.569}$
	5			$\varepsilon c = 1.77 \times 10^{-4}(N)^{.541}$	
2% Passing No. 200, Asphalt Content	7			$\varepsilon c = 2.24 \times 10^{-4}(N)^{.516}$	
	6				
	5			$\varepsilon c = 1.42 \times 10^{-4}(N)^{.626}$	

Table 15d. Permanent Deformation Strain (εc) for a Mix Tensile Strain (εt) Equal to 100 Microstrains. Conditioned Samples

		LEVEL OF COMPACTION			
		100%	97%	92%	90%
10% PASSING NO.200, ASPHALT CONTENT, %	7			$\varepsilon c = 2.35 \times 10^{-4}(N)^{.519}$	
	6				
	5			$\varepsilon c = 2.71 \times 10^{-4}(N)^{.454}$	
6% PASSING NO.200, ASPHALT CONTENT, %	7			$\varepsilon c = 2.08 \times 10^{-4}(N)^{.538}$	
	6	$\varepsilon c = 4.80 \times 10^{-4}(N)^{.323}$	$\varepsilon c = 4.36 \times 10^{-4}(N)^{.385}$	$\varepsilon c = 2.47 \times 10^{-4}(N)^{.531}$	$\varepsilon c = 2.21 \times 10^{-4}(N)^{.547}$
	5			$\varepsilon c = 2.26 \times 10^{-4}(N)^{.512}$	
2% PASSING NO.200, ASPHALT CONTENT, %	7			$\varepsilon c = 4.97 \times 10^{-4}(N)^{.538}$	
	6				
	5			$\varepsilon c = 1.63 \times 10^{-4}(N)^{.565}$	

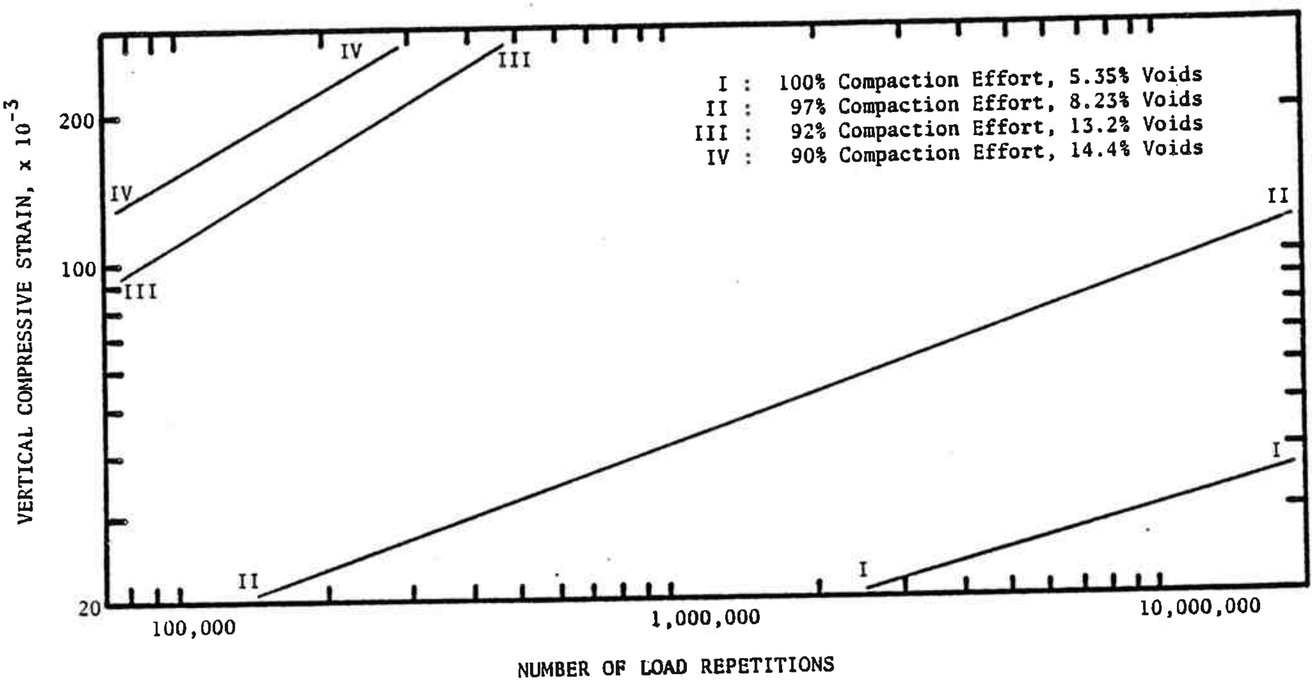


Figure 21a. Influence of Mix Density on Permanent Deformation
As Compacted Samples
6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt

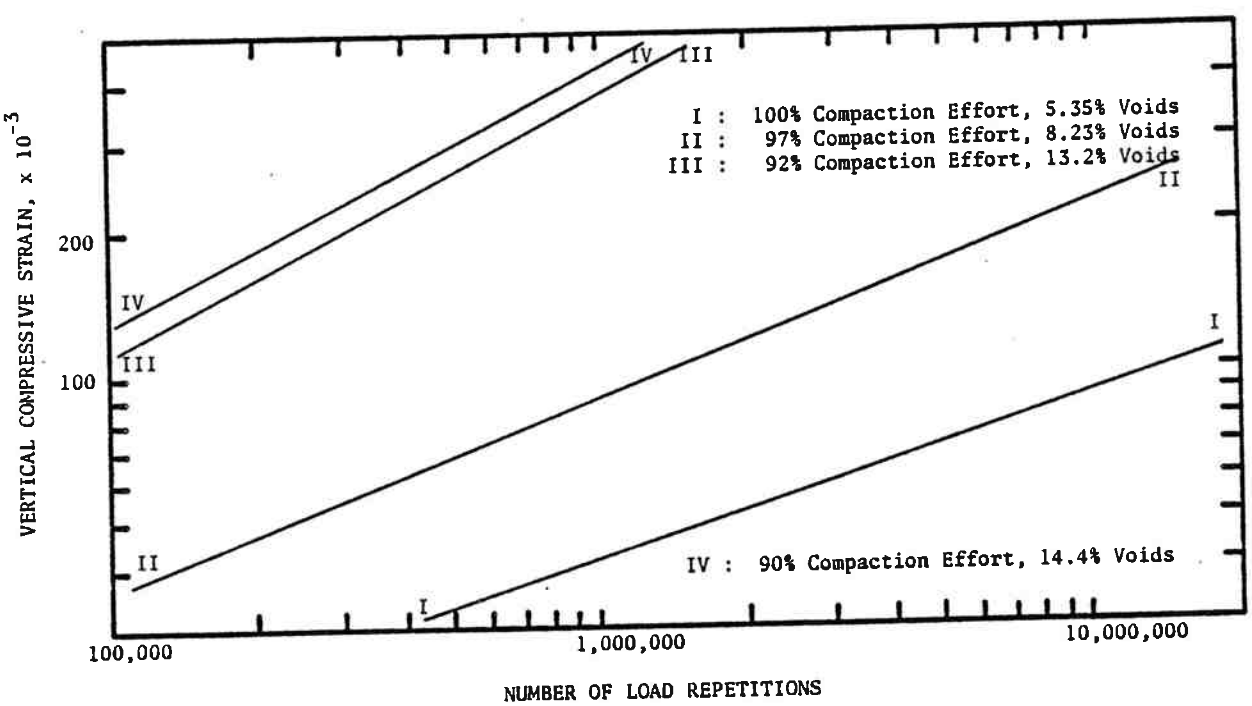


Figure 21b. Influence of Mix Density on Permanent Deformation,
Conditioned Samples.
6% Passing No. 200 - 25% Passing No. 10 - 6% Asphalt

compacted samples (90-92% compaction). The influence of asphalt content on the mix permanent deformation is illustrated in Figures 22a and 22b for samples prepared with 6% passing No. 200. According to the as-compacted 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%. However, after conditioning, the difference between the 6 and 7 percent asphalt content is substantially reduced. Figures 23a and 23b show the influence of asphalt content when samples are prepared with 2 and 10 percent passing the No. 200 sieve. The conditioned data indicate that:

(1) Increasing the asphalt content increases the mix sensitivity to permanent deformation.

(2) Increasing the percent passing No. 200 sieve decreases the mix sensitivity to permanent deformation.

The influence of the percent passing the No. 200 sieve is more clearly shown in Figures 24 and 25. As-compacted and conditioned samples indicate that an increase in the percent passing the No. 200 sieve decreases the mix susceptibility to permanent deformation.

In summary, the mix permanent deformation is minimized when:

(1) The mix level of compaction is increased,

(2) The amount passing the No. 200 sieve is increased, and

(3) The effect of asphalt content depends on whether the samples were tested as compacted or after conditioning.

Indirect Tensile Strength 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 results have been summarized in Table 16. Resilient modulus was measured on all samples before running the indirect tensile test. Table 16a gives the average as-compacted resilient modulus and tensile strength. For the conditioned samples, resilient modulus was measured both before and after conditioning. These data are recorded in Table 16b along with the conditioned tensile strength.

Relationship between resilient modulus and indirect tensile strength is shown in Figure 26a for the as-compacted data and in Figure 26b for the conditioned data. The coefficients of correlation indicate a relatively good correlation Resilient modulus - Indirect tensile strength.

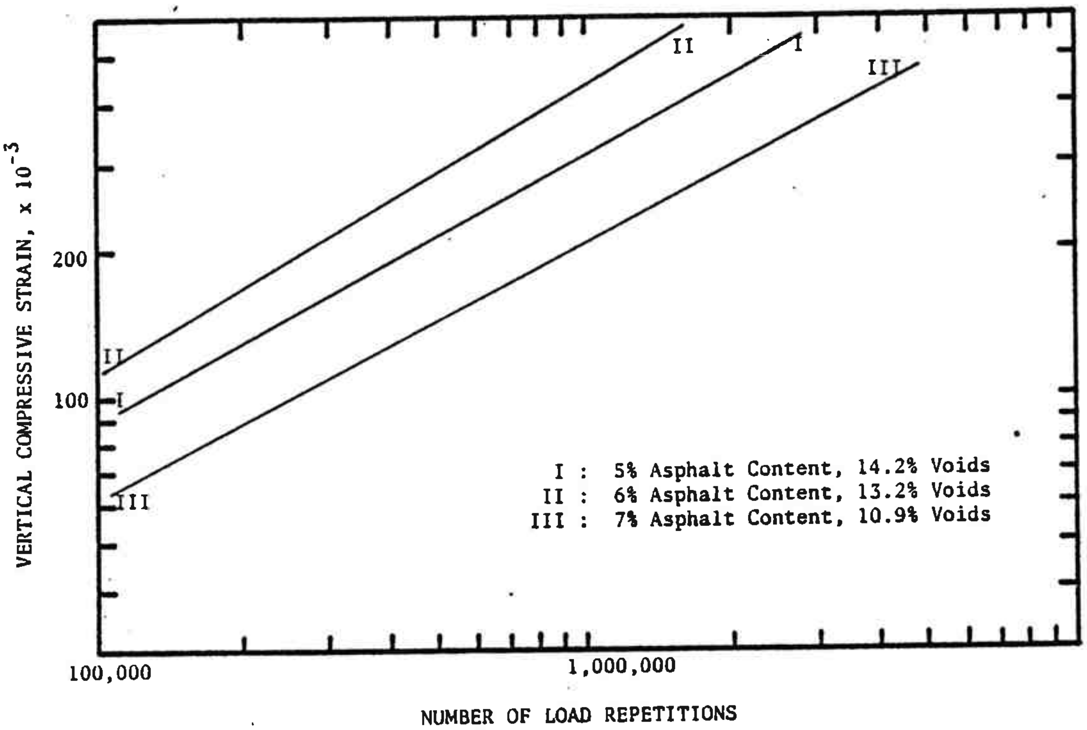


Figure 22a. Influence of Asphalt Content on Permanent Deformation, As Compacted Samples.
6% Passing No. 200 - 25% Passing No. 10 - 92% Compaction.

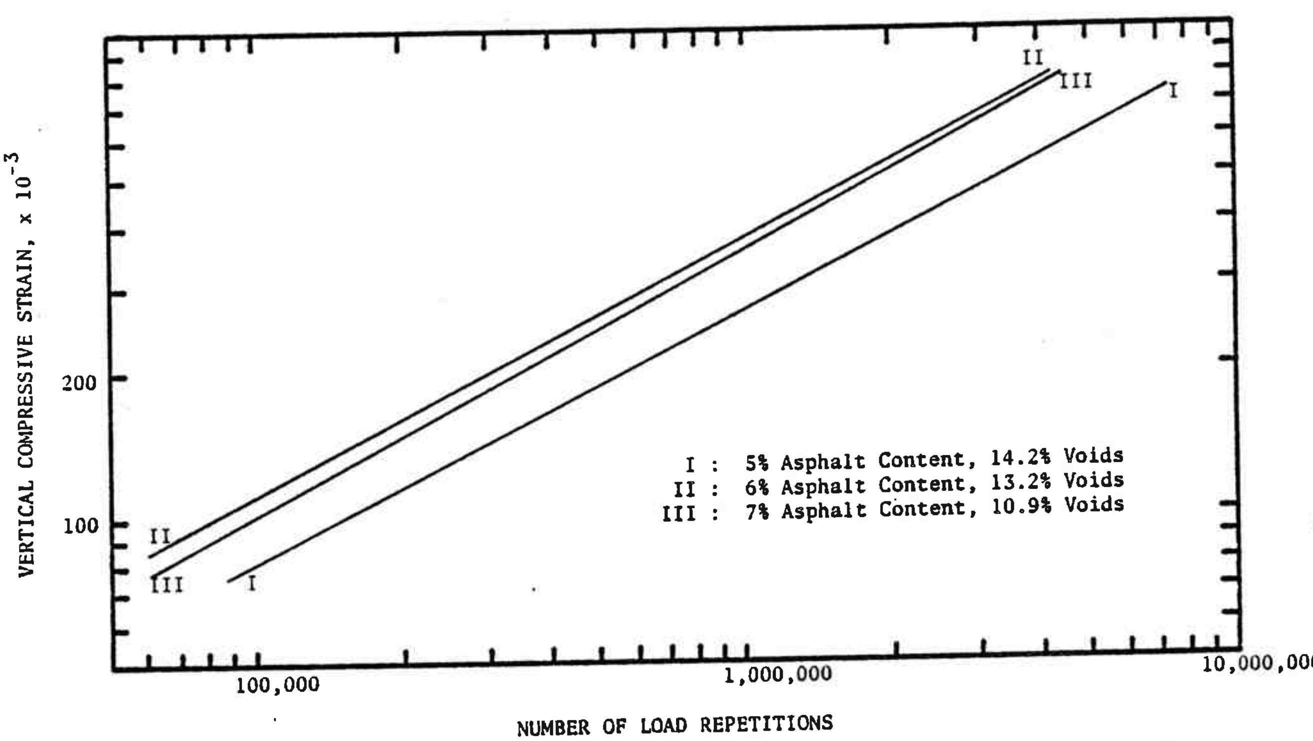


Figure 22b. Influence of Asphalt Content on Permanent Deformation, Conditioned Samples.
6% Passing No. 200 - 25% Passing No. 10 - 92% Compaction

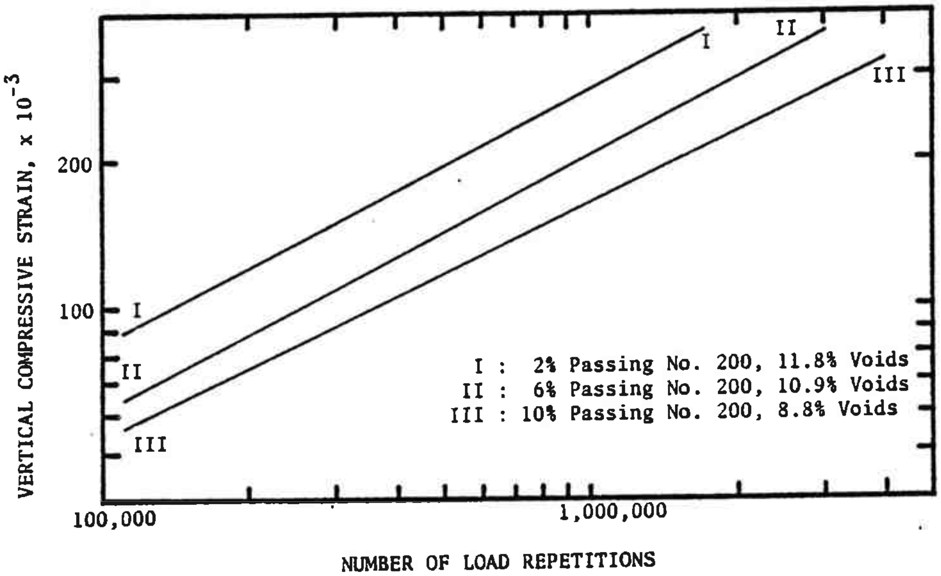


Figure 25a. Influence of Passing No. 200 on Permanent Deformation, As Compacted Samples 25% Passing No. 10 - 7% Asphalt - 92% Compaction

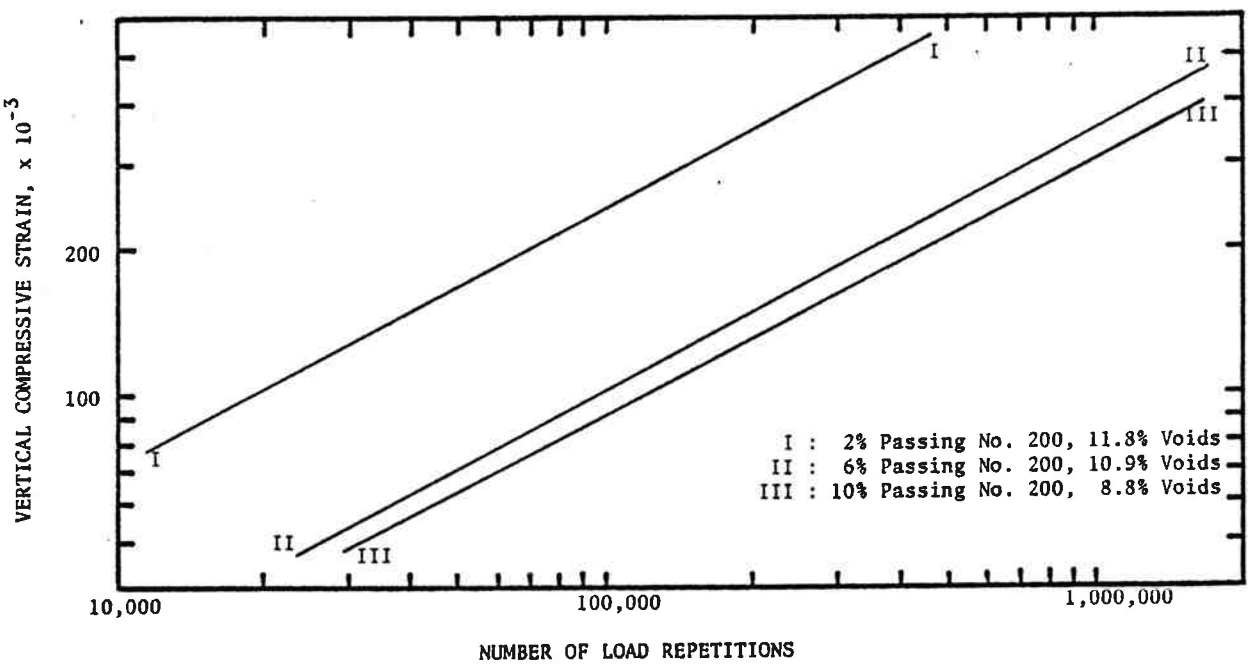


Figure 25b. Influence of Passing No. 200 on Permanent Deformation, Conditioned Samples: 25% Passing No. 10 - 7% Asphalt - 92% Compaction.

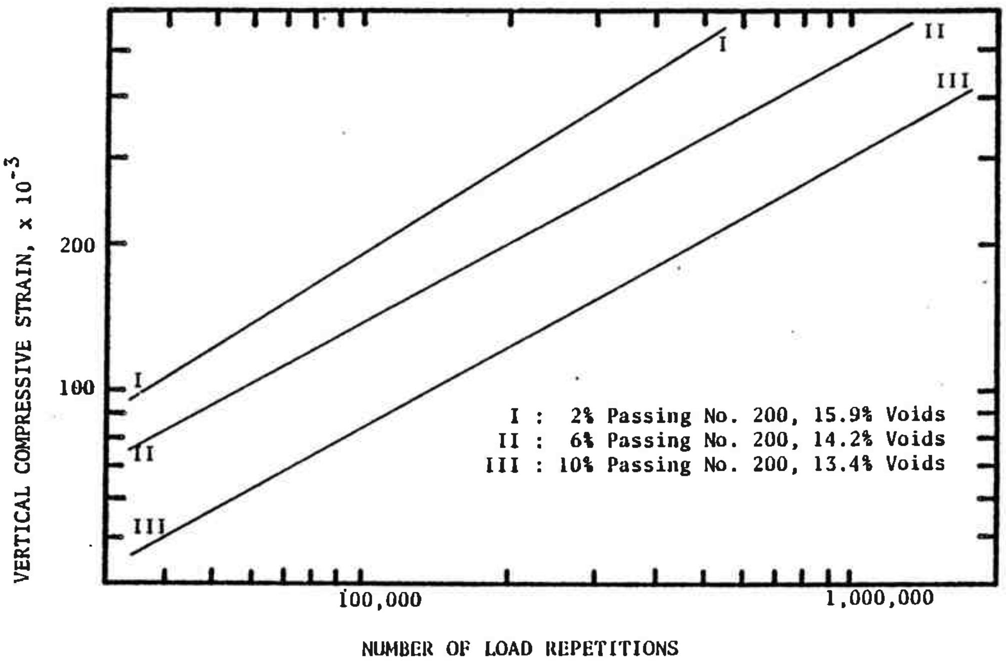


Figure 24a. Influence of Passing No. 200 on Permanent Deformation, As Compacted Samples.
25% Passing No. 10 - 5% Asphalt - 92% Compaction

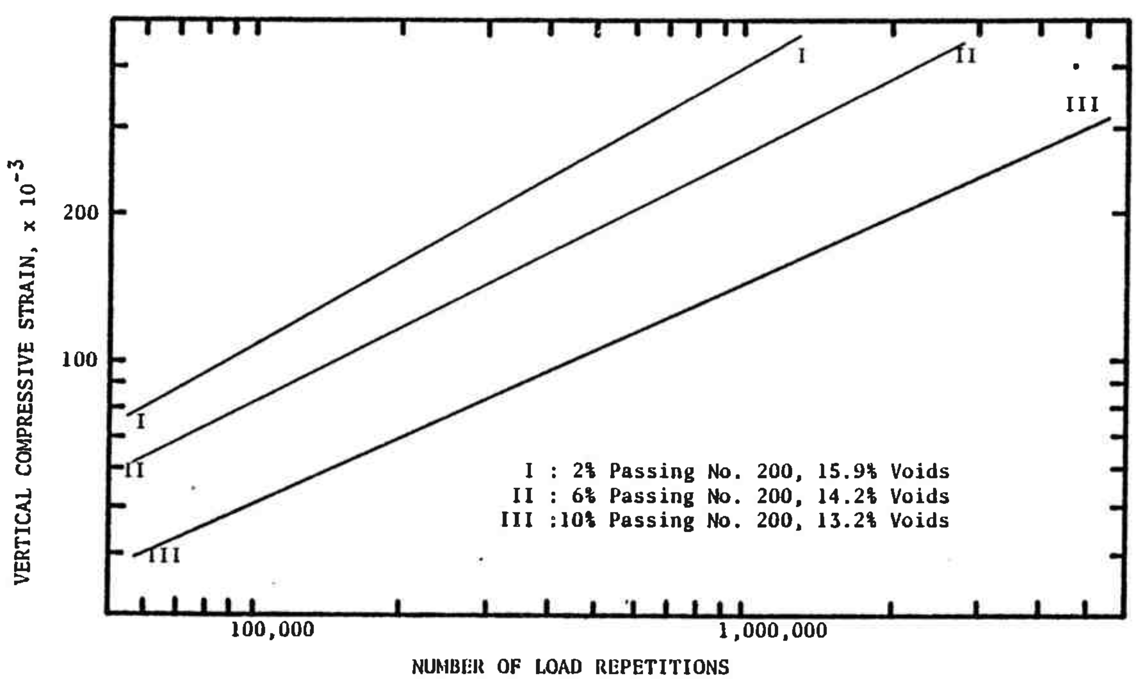


Figure 24b. Influence of Passing No. 200 on Permanent Deformation, Conditioned Samples.
25% Passing No. 10 - 5% Asphalt - 92% Compaction

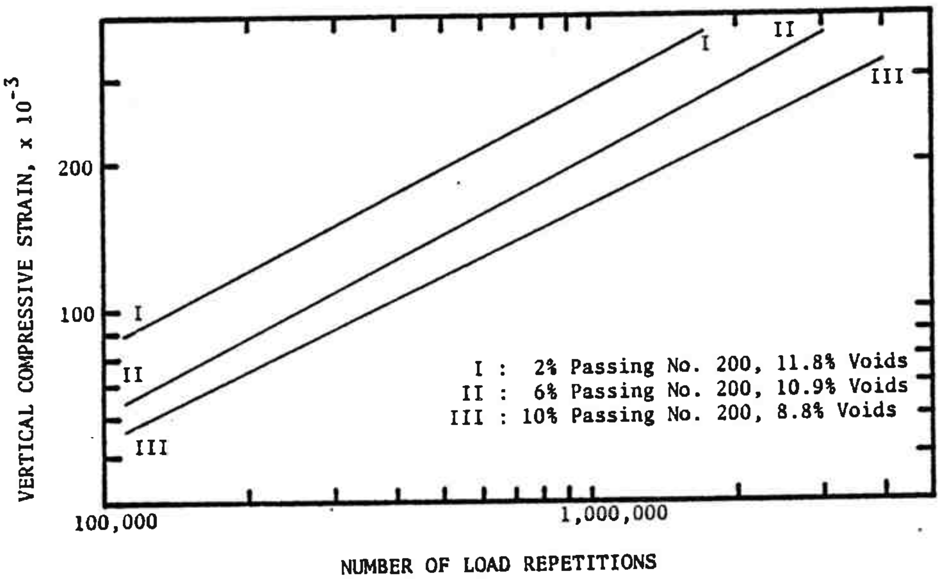


Figure 25a. Influence of Passing No. 200 on Permanent Deformation, As Compacted Samples
25% Passing No. 10 - 7% Asphalt - 92% Compaction

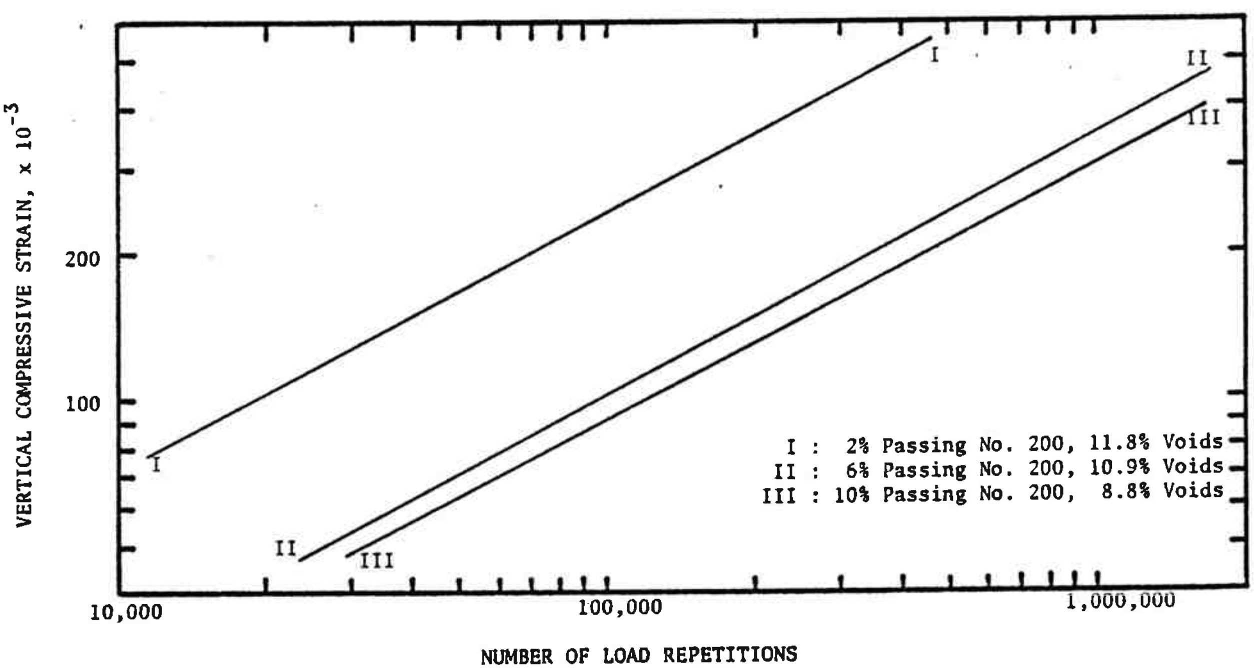


Figure 25b. Influence of Passing No. 200 on Permanent Deformation, Conditioned Samples: 25% Passing No. 10 - 7% Asphalt - 92% Compaction.

Table 16a. Indirect Tensile Strength Test Results - As Compacted Samples.

- Tensile Strength (PSI)
- Resilient Modulus (PSI)

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
100%					230 735,000				
97%					142 524,000				
92%	54 189,000		54 151,000	86 384,000	67 251,000	107 274,000	72 332,000		90 310,000
90%					71 171,000				

Table 16b. Indirect Tensile Strength Test, Conditioned Data.

- Tensile Strength (PSI)
- Resilient Modulus after Conditioning (PSI)
- Resilient Modulus before Conditioning (PSI)

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
100%					227 529,000 749,000				
97%					170 357,000 523,000				
92%	100 241,000 239,000		102 168,000 180,000	132 409,000 395,000	105 228,000 332,000	93 175,000 260,000	111 320,000 371,000		130 210,000 367,000
90%					84 201,000 321,000				

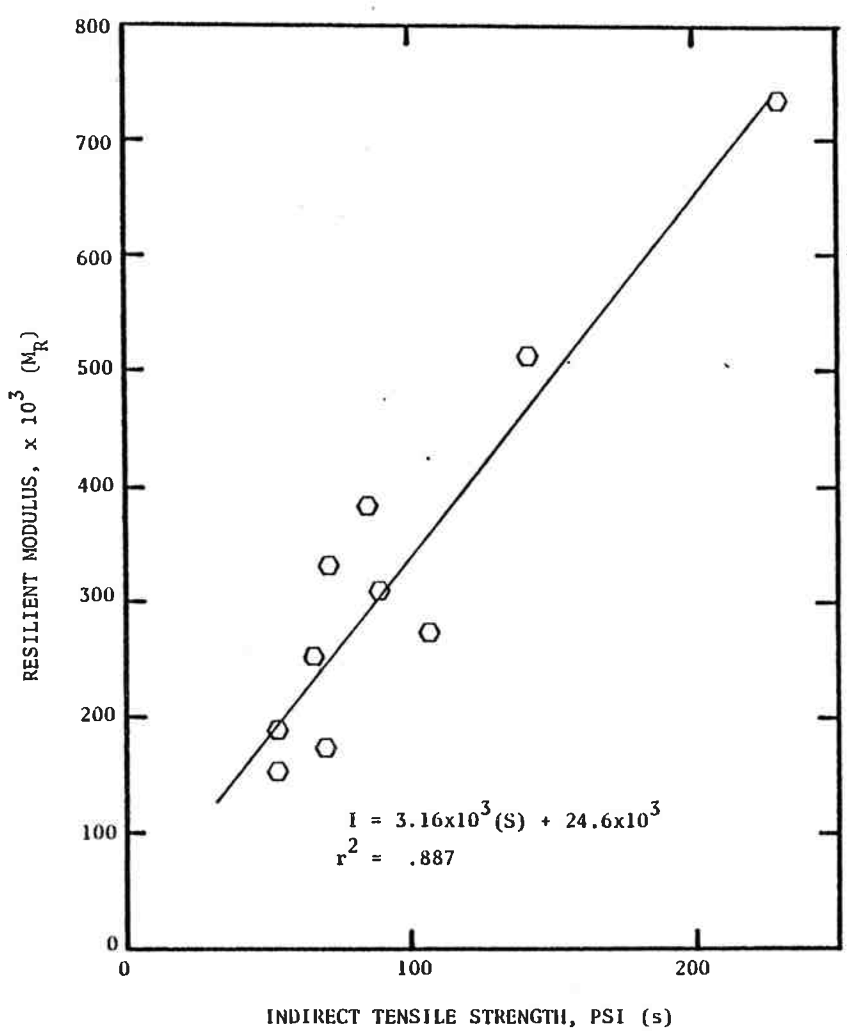


Figure 26a. Relationship Between Indirect Tensile Strength and Resilient Modulus - As-Compacted Samples.

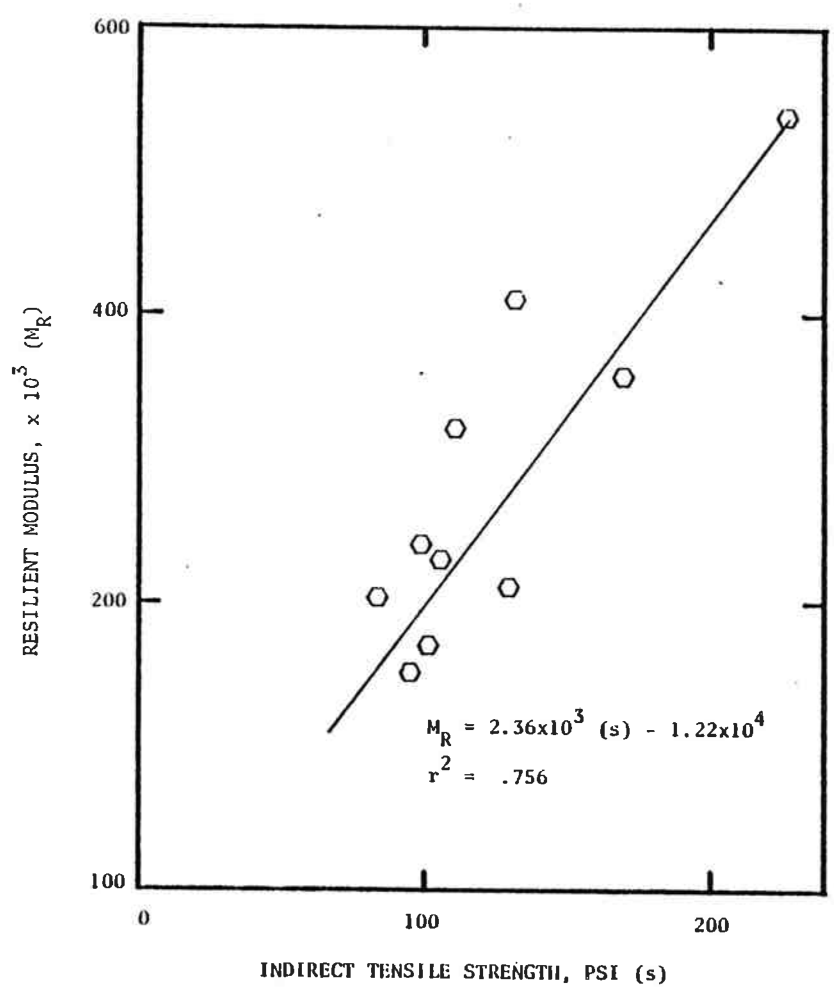


Figure 26b. Relationship Between Indirect Tensile Strength and Resilient Modulus - Conditioned Samples.

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 by calculating the percent reduction in life.

This calculation will be accomplished for three strain levels: 150, 100 and 50 microstrain. The values corresponding to 100 microstrain should be considered as average values, since all tests have been performed between 50 and 150 microstrain. Table 17 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 97% of the maximum laboratory density. This standard mix is compared in Table 17 with mixes compacted at different levels: 100, 92 and 90% 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

$$\frac{N_{os}}{N_{sm}} \times 100 \qquad (19)$$

The pay factors shown in Table 17 indicate that the effect of variations in mix density are more important for low strain values than high strain values. At 150 microstrain, the pay factors have a tendency to become closer to 1.

Table 18 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. At 100 microstrain, the conditioned data show that 6% asphalt content is the optimum.

Table 17. Estimated Reduction in Pavement Life - Fatigue Criteria Effect of Mix Density

	LEVEL OF COMPACTION	MIX B.S.G.	TEST CONDITION*	STRAIN LEVEL 50 $\mu\varepsilon$	100 $\mu\varepsilon$	150 $\mu\varepsilon$
PAVEMENT LIFE	Standard (97%)	2.23	BC	4.09×10^5	4.51×10^4	1.24×10^4
			AC	6.00×10^5	5.19×10^4	1.24×10^4
	100%	2.30	BC	6.62×10^5	6.10×10^4	1.51×10^4
			AC	1.05×10^6	6.72×10^4	1.35×10^4
	92%	2.11	BC	3.96×10^4	1.04×10^4	4.75×10^3
			AC	9.51×10^4	2.14×10^4	8.96×10^3
	90%	2.08	BC	2.31×10^4	7.41×10^3	3.81×10^3
			AC	5.29×10^4	1.66×10^4	8.44×10^3
PAY FACTOR	Standard (97%)	2.23	BC	1.0	1.0	1.0
			AC	1.0	1.0	1.0
	100%	2.30	BC	1.62	1.35	1.22
			AC	1.74	1.29	1.09
	92%	2.11	BC	.09	.23	.38
			AC	.16	.41	.72
	91%	2.08	BC	.06	.16	.31
			AC	.09	.32	.68

* BC - Before Conditioning
AC - After Conditioning

The impact of amount of fines on fatigue life is shown in Table 19 for mix composed of 5% asphalt, and Table 20 for mixes composed of 7% asphalt. Test results for 5 percent asphalt content show the optimum percent passing the No. 200 sieve more clearly than the 7 percent asphalt content. The asphalt content tends to mask the properties of the percent passing No. 200 and create a complex asphalt-fines in which the optimum percent passing the No. 200 sieve is a function of the asphalt content. The results reported on Tables 19 and 20 also indicate that excess percent passing the No. 200 sieve is less detrimental to the mix than if a too low amount of fines is used.

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:

$$\varepsilon_F = \frac{3/4}{T} \times 100 \qquad (20)$$

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

$$\varepsilon_c = (Ax(\varepsilon_T) + B)\ N^S \qquad (21)$$

where

ε_c = Cumulative permanent strain

ε_T = Initial tensile strain

A,B,S = Regression constraints

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

Table 18. Estimated Reduction in Pavement Life - Fatigue Criteria
Effect of Asphalt Content
6% Passing No. 200 - 92% Compaction

	ASPHALT CONTENT	MIX B.S.G.	TEST CONDITION*	STRAIN LEVEL		
				50 με	100 με	150 με
PAVEMENT LIFE	Standard (6%)	2.11	BC	3.96×10^4	1.04×10^4	4.75×10^3
			AC	9.51×10^4	2.14×10^4	8.96×10^3
	5%	2.12	BC	3.60×10^4	1.00×10^4	4.72×10^3
			AC	5.87×10^4	1.73×10^4	8.49×10^3
	7%	2.13	BC	1.23×10^5	2.15×10^4	7.75×10^3
			AC	7.03×10^4	1.92×10^4	9.01×10^3
PAY FACTOR	Standard (6%)	2.11	BC	1.0	1.0	1.0
			AC	1.0	1.0	1.0
	5%	2.12	BC	.91	.96	.99
			AC	.62	.81	.95
	7%	2.13	BC	3.12	2.07	1.63
			AC	.74	.90	1.01

* BC - Before Conditioning
AC - After Conditioning

Table 19. Estimated Reduction in Pavement Life - Fatigue Criteria
Effect of P_{200} at 5% Asphalt

	PERCENT P_{200}	MIX B.S.G.	TEST CONDITION*	STRAIN LEVEL		
				50 με	100 με	150 με
PAVEMENT LIFE	Standard (6%)	2.12	BC	3.60×10^4	1.00×10^4	4.72×10^3
			AC	5.87×10^4	1.73×10^4	8.49×10^3
	2%	2.07	BC	6.81×10^3	3.49×10^3	2.36×10^3
			AC	1.61×10^4	9.48×10^3	6.97×10^3
	10%	2.13	BC	3.19×10^4	9.11×10^3	4.37×10^3
			AC	5.37×10^4	1.70×10^4	8.66×10^3
PAY FACTOR	Standard (6%)	2.12	BC	1.0	1.0	1.0
			AC	1.0	1.0	1.0
	2%	2.07	BC	.19	.35	.50
			AC	.27	.55	.82
	10%	2.13	BC	.89	.91	.93
			AC	.92	.98	1.02

* BC - Before Conditioning
AC - After Conditioning

$$N = \left[\frac{\varepsilon_c}{(A(\varepsilon_1) + B)}\right]^{1/s} \qquad (22)$$

The total pavement thickness on the Castle Rock-Cedar Creek project is about 5 inches (12.7 cm). From equation (20), the cumulative permanent strain at failure is:

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

Equation (19) becomes:

$$N = \left[\frac{.15}{(A(\varepsilon_T) + B)}\right]^{1/s} \qquad (23)$$

Estimated pavement lives have been calculated using equation (23) for three values of the initial tensile strain: 50, 100 and 150 microstrain.

Table 21 illustrates the importance of the mix density in the development of rutting within the asphalt concrete layer. Increasing the mix Bulk Specific Gravity from 2.08 to 2.30 improves the resistance to rutting by a factor of approximately 350. The influence of the asphalt content presented in Table 22 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 23 and 24. The results indicate a strong sensitivity of the conditioned samples to permanent deformation.

Table 20. Estimated Reduction in Pavement Life - Fatigue Criteria
Effect of P_{200} at 7% Asphalt

	PERCENT P_{200}	MIX B.S.G.	TEST CONDITION*	STRAIN LEVEL 50 με	100 με	150 με
PAVEMENT LIFE	Standard (6%)	2.13	BC	1.23×10^5	2.15×10^4	7.75×10^3
			AC	7.03×10^4	1.92×10^4	9.01×10^3
	2%	2.09	BC	6.21×10^4	1.37×10^4	5.66×10^3
			AC	3.06×10^4	1.31×10^4	8.00×10^3
	10%	2.17	BC	1.50×10^5	2.39×10^4	8.16×10^3
			AC	1.36×10^5	2.57×10^4	9.72×10^3
PAY FACTOR	Standard (6%)	2.13	BC	1.0	1.0	1.0
			AC	1.0	1.0	1.0
	2%	2.09	BC	.50	.64	.73
			AC	.43	.68	.89
	10%	2.17	BC	1.21	1.11	1.05
			AC	1.93	1.34	1.08

* BC - Before Conditioning
AC - After Conditioning

Table 21. Estimated Reduction in Pavement Life - Permanent Deformation Criteria
Effect of Mix Density

	LEVEL OF COMPACTION	MIX B.S.G.	TEST CONDITION*	STRAIN LEVEL 50 με	100 με	150 με
PAVEMENT LIFE	Standard (97%)	2.23	BC	2.60×10^8	3.60×10^7	1.15×10^7
			AC	7.21×10^7	3.88×10^6	1.01×10^6
	100%	2.30	BC	2.85×10^{10}	3.29×10^9	8.61×10^8
			AC	2.06×10^8	5.30×10^7	2.07×10^7
	92%	2.11	BC	5.39×10^5	1.65×10^5	8.27×10^4
			AC	3.95×10^5	1.75×10^5	9.94×10^4
	90%	2.08	BC	3.04×10^5	9.79×10^4	4.95×10^4
			AC	7.20×10^5	1.50×10^5	6.52×10^4
PAY FACTOR	Standard (97%)	2.23	BC	1.0	1.0	1.0
			AC	1.0	1.0	1.0
	100%	2.30	BC	110.0	91.5	74.8
			AC	2.86	13.7	20.4
	92%	2.11	BC	.002	.005	.007
			AC	.005	.045	.098
	90%	2.08	BC	.001	.003	.004
			AC	.01	.04	.06

* BC - Before Conditioning
AC - After Conditioning

Table 22. Estimated Reduction on Pavement Life - Permanent Deformation Criteria
Effect of Asphalt Content
6% Passing No. 200 - 92% Compaction

	ASPHALT CONTENT	MIX B.S.G.	TEST CONDITION*	STRAIN LEVEL 50 με	100 με	150 με
PAVEMENT LIFE	Standard (6%)	2.11	BC	5.39×10^5	1.65×10^5	8.27×10^4
			AC	3.95×10^5	1.75×10^5	9.94×10^4
	7%	2.12	BC	1.56×10^6	5.51×10^5	2.80×10^5
			AC	6.95×10^5	2.04×10^5	9.84×10^4
	5%	2.13	BC	1.66×10^6	2.58×10^5	1.04×10^5
			AC	1.16×10^6	3.25×10^5	1.52×10^5
PAY FACTOR	Standard (6%)	2.11	BC	1.0	1.0	1.0
			AC	1.0	1.0	1.0
	7%	2.12	BC	2.91	3.34	3.38
			AC	1.76	1.17	.99
	5%	2.13	BC	3.06	1.56	1.26
			AC	2.93	1.86	1.52

* BC - Before Conditioning
AC - After Conditioning

Table 23. Estimated Reduction in Pavement Life - Permanent Deformation Criteria
Effect of P_{200} at 5% Asphalt

	PERCENT P_{200}	MIX B.S.G.	TEST CONDITION*	STRAIN LEVEL 50 με	100 με	150 με
PAVEMENT LIFE	Standard (6%)	2.12	BC	1.66×10^6	2.58×10^5	1.04×10^5
			AC	1.16×10^6	3.25×10^5	1.52×10^5
	10%	2.13	BC	8.96×10^5	2.80×10^5	1.38×10^5
			AC	2.20×10^6	1.10×10^6	6.52×10^5
	2%	2.07	BC	3.93×10^5	6.79×10^4	3.00×10^4
			AC	8.28×10^5	1.76×10^5	7.81×10^4
PAY FACTOR	Standard (6%)	2.12	BC	1.0	1.0	1.0
			AC	1.0	1.0	1.0
	10%	2.13	BC	.54	1.09	1.33
			AC	1.90	3.39	4.30
	2%	2.07	BC	.24	.26	.29
			AC	.71	.54	.52

* BC - Before Conditioning
AC - After Conditioning

Table 24. Estimated Reduction in Pavement Life - Permanent Deformation Criteria Effect of P_{200} at 7% Asphalt

	PERCENT P_{200}	MIX B.S.G.	TEST CONDITION*	STRAIN LEVEL 50 με	100 με	150 με
PAVEMENT LIFE	Standard (6%)	2.13	BC	$1.57x10^6$	$5.51x10^5$	$2.80x10^5$
			AC	$6.95x10^5$	$2.04x10^5$	$9.84x10^4$
	10%	2.17	BC	$3.71x10^6$	$8.22x10^5$	$3.46x10^5$
			AC	$1.08x10^6$	$2.54x10^5$	$1.12x10^5$
	2%	2.09	BC	$1.36x10^6$	$2.99x10^5$	$1.29x10^5$
			AC	$5.37x10^4$	$4.07x10^4$	$3.19x10^4$
PAY FACTOR	Standard (6%)	2.13	BC	1.0	1.0	1.0
			AC	1.0	1.0	1.0
	10%	2.17	BC	2.36	1.49	1.24
			AC	1.55	1.24	1.14
	2%	2.09	BC	.86	.54	.46
			AC	.08	.19	.32

* BC - Before Conditioning
AC - After Conditioning

6.0 CONCLUSIONS AND RECOMMENDATIONS

Performance of the mix used in the construction of the Castle Rock-Cedar Creek 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%, 97%, 92% and 90%

(2) Asphalt content: 5%, 6% and 7%

(3) Percent of fines: 2%, 6% and 10%

Study of the three variables presented above was done using the same type of aggregate as used for the construction of the Castle Rock-Cedar Creek 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. The percent passing the No. 200 sieve is the second important factor for both the mix fatigue life and the mix permanent deformation. Study of the effect of the amount of fines in the mix was done at 5 and 7% asphalt content. In both cases, decreasing the amount of fines decreased the fatigue life and resistance to permanent deformation. When comparing the results for 5 and 7% asphalt content, the mix resilient modulus and fatigue life data indicate that the optimum percent passing the No. 200 sieve could be a function of the asphalt content in the mix. This is particularly noticeable with the conditioned samples. An optimum percent passing the No. 200 sieve was found for the 5% asphalt content, but was not found when the asphalt content was increased to 7 percent.

One percent change in asphalt content from the design optimum did not change significantly the fatigue life of the mix. However, a maximum fatigue life was observed at 6 percent asphalt content. Reducing the asphalt content to 5 percent increased the mix resistance to permanent deformation.

Based on fatigue data, 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 25. Pavement life data indicate that fatigue life is generally shorter than permanent deformation life. Therefore, the permanent

Table 25. Summary of Most Critical Pay Adjustment Factors*
(at 100 microstrain)

	PERCENT LEVEL OF COMPACTION			
	97	100	92	90
PAY FACTOR	1.0	1.29	.41	.32

	PERCENT ASPHALT CONTENT		
	6.0	5.0	7.0
PAY FACTOR	1.0	.81	.90

	PERCENT FINES		
	6	2	10
PAY FACTOR	1.0	.62	1.16

* 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.

deformation data have not been included in the calculation of the pay factors. All pay factors were calculated for a mix tensile strain of 100 microstrain. Only the conditioned data have been considered in Table 25, since conditioned samples are assumed to be more closely duplicating a cured pavement condition. Pay factors developed at 2% and 10% passing the No. 200 sieve are the average pay factors calculated at 5 and 7 percent asphalt. The results corroborate earlier remarks:

1) Lowering the mix density decreases the mix resistance to fatigue.
2) The design optimum asphalt content is corroborated by the fatigue test results.
3) Low percent passing the No. 200 sieve affect primarily the mix permanent deformation, whereas excess amount of fines affect the mix fatigue life favorably.

Recommendations for further research on this project are related to the complex asphalt-fines.

The optimum percent passing the No. 200 sieve seems to be related to the asphalt content in the mix (See effect of P200 at 5% asphalt on fatigue life and on mix resilient modulus). More data are therefore required at 6 and 7 percent asphalt content to follow any change in the optimum amount of fines.

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APPENDIX A

SUMMARY OF CORE DATA

This appendix summarizes core data taken across the panel at five locations on the Castle Rock-Cedar Creek Project. Locations are given in Table A-1. Table A-2 summarizes the results of individual tests on the cores.

Table A-1. Location of Asphalt Cores

STATION LOCATION	DIRECTION	DISTANCE FROM ℄, FT
Mile Post 7.8	West Bound	2 Lt, 4 Lt, 6 Lt, 8 Lt, 10 Lt
Mile Post 5.3	West Bound	2 Lt, 4 Lt, 6 Lt, 8 Lt, 10 Lt
Mile Post 6.3	West Bound	2 Lt, 4 Lt, 6 Lt, 8 Lt, 10 Lt
Mile Post 9.1	West Bound	2 Rt, 4 Rt, 6 Rt, 8 Rt, 10 Rt
Mile Post 11.3	East Bound	1 1/2 Rt, 3 1/2 Rt, 5 1/2 Rt, 7 1/2 Rt, 9 1/2 Rt

Table A-2. Summary of Test Results

(a) Mile Post 7.8, West Bound

	LOCATION									
	2' Lt		4' Lt		6' Lt		8' Lt		10' Lt	
	TOP	BASE	TOP	BASE	TOP	BASE	TOP	BASE	TOP	BASE
MIX PROPERTY										
Thickness, in.	1.7	2.1	1.6	1.8	1.6	1.8	1.6	1.8	1.8	2.8
Bulk Specific Gravity, in place	2.15	2.19	2.18	2.16	2.18	2.13	2.09	2.12	2.12	2.16
Bulk Specific Gravity, recompacted	2.31	2.31	2.31	2.32	2.32	2.33	2.22	2.31	2.30	2.30
Rice Gravity	2.45	2.40	2.44	2.39	2.41	2.40	2.48	2.42	2.44	2.42
% Voids, in place	12.2	8.8	10.7	9.6	9.5	11.2	15.7	12.4	13.1	10.7
Gradation, % passing										
1/4-inch	68	73	63	74	66	72	48	64	64	66
No. 10	33	36	28	38	32	34	21	32	30	31
No. 200	7.3	7.5	6.8	7.6	7.5	5.7	5.3	6.6	6.7	6.5
Asphalt Content, %	6.7	7.0	6.0	6.8	6.8	8.9	5.1	6.5	6.2	6.6
Water Content, %	1.04	1.05	1.64	1.12	1.74	2.10	1.93	1.90	1.66	2.32
Modulus, ksi	198	245	243	223	268	-	124	202	247	257
ASPHALT PROPERTIES										
Pen @ 77°F, dmm	50	56	-	-	-	-	-	-	-	-
Viscosity @ 275°F, cs	408	405	-	-	-	-	-	-	-	-
Viscosity @ 140°F, p	2457	2263	-	-	-	-	-	-	-	-

Table A-2. Summary of Test Results

(b) Mile Post 5.3, West Bound

	LOCATION									
	2' Lt		4' Lt		6' Lt		8' Lt		10' Lt	
	TOP	BASE	TOP	BASE	TOP	BASE	TOP	BASE	TOP	BASE
MIX PROPERTY										
Thickness, in.	1.8	2.5	1.8	2.4	1.7	2.5	1.8	2.8	1.8	2.9
Bulk Specific Gravity, in place	2.07	2.11	2.11	2.13	2.10	2.11	2.09	2.16	2.15	2.15
Bulk Specific Gravity, recompacted	2.28	2.29	2.28	2.27	2.27	2.25	2.20	2.28	2.31	2.29
Rice Gravity	2.47	2.42	2.49	2.42	2.46	2.43	2.49	2.42	2.42	2.43
% Voids, in place	16.2	12.8	15.3	12.0	14.6	13.2	16.0	10.7	11.2	11.5
Gradation, % passing										
1/4-inch	58	58	52	61	51	57	31	54	50	52
No. 10	25	26	24	28	23	25	14	24	21	23
No. 200	5.6	5.9	5.5	6.2	5.1	6.0	3.5	6.1	4.7	5.8
Asphalt Content, %	6.0	6.6	5.7	7.0	6.5	6.7	5.3	6.3	6.7	6.4
Water Content, %	0.9	0.84	0.84	1.0	2.32	1.64	2.55	2.26	1.93	2.47
Modulus, ksi	176	196	159	209	158	223	209	221	200	185
ASPHALT PROPERTIES										
Pen @ 77°F, dmm	58	54	-	-	-	-	-	-	-	-
Viscosity @ 275°F, cs	386	419	-	-	-	-	-	-	-	-
Viscosity @ 140°F, p	2079	2326	-	-	-	-	-	-	-	-

Table A-2. Summary of Test Results

(c) Mile Post 6.3, West Bound

	LOCATION									
	2' Lt		4' Lt		6' Lt		8' Lt		10' Lt	
	TOP	BASE	TOP	BASE	TOP	BASE	TOP	BASE	TOP	BASE
MIX PROPERTY										
Thickness, in.	1.9	1.9	2.0	2.0	2.0	1.7	2.0	2.0	1.7	2.1
Bulk Specific Gravity, in place	2.08	2.04	2.08	2.01	2.14	2.22	2.05	2.14	2.08	2.14
Bulk Specific Gravity, recompacted	2.22	2.29	2.23	2.28	2.28	2.35	2.23	2.33	2.21	2.30
Rice Gravity	2.52	2.40	2.52	2.44	2.47	2.39	2.48	2.44	2.50	2.44
% Voids, in place	17.5	15.0	17.5	17.6	13.4	7.1	17.3	12.3	16.8	12.3
Gradation, % passing										
1/4-inch	49	65	48	51	61	64	48	64	47	60
No. 10	21	29	21	22	29	30	20	30	21	27
No. 200	5.4	6.6	5.6	5.2	6.8	6.7	4.8	6.8	5.8	6.3
Asphalt Content, %	4.7	7.4	5.1	6.6	5.8	7.5	4.9	6.9	4.8	6.8
Water Content, %	1.61	1.70	1.74	1.19	1.69	1.43	1.22	1.50	1.40	1.48
Modulus, ksi	158	185	152	113	266	228	136	211	211	-
ASPHALT PROPERTIES										
Pen @ 77°F, dmm	43	65	-	-	-	-	-	-	-	-
Viscosity @ 275°F, cs	478	390	-	-	-	-	-	-	-	-
Viscosity @ 140°F, p	3507	1875	-	-	-	-	-	-	-	-

Table A-2. Summary of Test Results

(d) Mile Post 9.1, West Bound

	LOCATION									
	2' Rt		4' Rt		6' Rt		8' Rt		10' Rt	
	TOP	BASE	TOP	BASE	TOP	BASE	TOP	BASE	TOP	BASE
MIX PROPERTY										
Thickness, in.	2.0	2.1	1.8	2.1	1.8	2.0	1.8	1.9	1.7	1.6
Bulk Specific Gravity, in place	2.12	2.14	2.13	2.17	2.23	2.17	2.15	2.12	2.19	2.18
Bulk Specific Gravity, recompacted	2.29	2.29	2.27	2.33	2.31	2.31	2.28	2.29	2.33	2.32
Rice Gravity	2.43	2.43	2.45	2.42	2.44	2.42	2.45	2.42	2.43	2.44
% Voids, in place	12.8	11.9	13.1	10.3	8.6	10.3	12.2	12.4	9.9	10.7
Gradation, % passing										
1/4-inch	68	62	59	65	65	66	67	64	66	64
No. 10	30	28	27	30	30	30	31	29	32	30
No. 200	6.8	6.4	6.0	7.0	6.7	7.5	6.9	6.7	6.3	6.9
Asphalt Content, %	6.6	6.7	5.8	7.1	6.3	7.2	6.6	7.2	7.4	6.9
Water Content, %	1.56	1.52	1.43	1.17	1.54	1.58	1.85	1.74	2.15	1.94
Modulus, ksi	184	186	203	220	201	301	263	214	-	-
ASPHALT PROPERTIES										
Pen @ 77°F, dmm	62	54	-	-	-	-	-	-	-	-
Viscosity @ 275°F, cs	385	404	-	-	-	-	-	-	-	-
Viscosity @ 140°F, p	1985	2210	-	-	-	-	-	-	-	-

Table A-2. Summary of Test Results

(e) Mile Post 11.3, East Bound

	LOCATION									
	1 1/2' Rt		3 1/2' Rt		5 1/2' Rt		7 1/2' Rt		9 1/2' Rt	
	TOP	BASE	TOP	BASE	TOP	BASE	TOP	BASE	TOP	BASE
MIX PROPERTY										
Thickness, in.	1.2	1.9	1.3	1.5	1.2	1.5	1.1	2.0	1.4	2.0
Bulk Specific Gravity, in place	2.12	2.10	2.12	2.12	2.14	2.17	2.14	2.17	2.11	2.29
Bulk Specific Gravity, recompacted	2.29	2.33	2.28	2.34	2.29	2.36	2.29	2.34	2.28	2.38
Rice Gravity	2.47	2.42	2.45	2.41	2.43	2.40	2.44	2.44	2.44	2.52
% Voids, in place	14.2	13.2	13.5	12.0	11.9	9.6	12.3	11.1	13.5	9.1
Gradation, % passing										
1/4-inch	66	64	65	74	64	66	67	66	66	67
No. 10	31	29	30	29	30	31	33	31	32	32
No. 200	7	6.6	7.1	6.6	6.9	7.1	7.6	7.1	7.4	7.1
Asphalt Content, %	6.4	7.5	6.3	7.1	6.3	7.7	6.5	7.5	6.3	7.2
Water Content, %	1.55	1.09	1.36	1.32	1.22	1.17	1.03	1.08	1.09	0.97
Modulus, ksi	-	203	-	209	-	-	214	-	-	250
ASPHALT PROPERTIES										
Pen @ 77°F, dmm	60	61	-	-	-	-	-	-	-	-
Viscosity @ 275°F, cs	401	364	-	-	-	-	-	-	-	-
Viscosity @ 140°F, p	1865	1778	-	-	-	-	-	-	-	-