IMPACT OF VARIATION IN MATERIAL PROPERTIES ON ASPHALT PAVEMENT LIFE

FINAL REPORT

HP & R Study: 0815157

May 1982

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Oregon Department of Transportation

in cooperation with

U. S. Department of Transportation
Federal Highway Administration

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-OR-82-3		

4. Title and Subtitle	5. Report Date / 6. Performing Organization Code / 8. Performing Organization Report No.
Impact of Variations in Material Properties on Asphalt Pavement Life	5. Report Date: May 1982
	6. Performing Organization Code:
7. Author(s) Prachuab Puangchit, R.G. Hicks, James E. Wilson, and C.A. Bell	8. Performing Organization Report No.: TE-82-6
9. Performing Organization Name and Address Department of Civil Engineering Oregon State University Corvallis, OR 97331	10. Work Unit No. (TRAIS) 11. Contract or Grant No. HPR 081-5157
12. Sponsoring Agency Name and Address Oregon Dept. of Transportation Materials & Research Sec. Salem, OR 97310 & U.S. Dept. of Transport. Federal Highway Administra. Office of Research & Dev. Washington, D.C. 20590	13. Type of Report and Period Covered Final (Jan 1980-May 1982) 14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract The effect of variations in asphalt concrete mix properties on pavement performance life is a crucial factor in highway pavement construction. In this study, data on samples prepared from field mix designs and using materials from North Oakland-Sutherlin, Castle Rock-Cedar Creek and Warren-Scappoose projects were analyzed by using the Statistical Interactive Programming System (SIPS) of Oregon State University. Regression analysis techniques were used to develop predictive models of pavement performance life based on each of the mix characteristics. The asphalt concrete mixture properties that were evaluated in this study were percent air void, asphalt content, gradation and aggregate type used. The percent air void or mix density is found to be the most dominant factor and most highly significant in controlling both fatigue cracking and rutting failure. The mixtures compacted to low void content showed remarkably long fatigue lives and high resistance to deformation. The highest performance life was obtained when the asphalt content and the amount of fines were at an optimum level, respectively. Deviation from optimum content in either of these properties causes a reduction in pavement life. The optimum asphalt content also appeared to be a function of aggregate type used. Thus, the degree of influence of air voids and asphalt content on mix behavior are reflected by the type of aggregate. Pay adjustment factors were developed based upon fatigue performance life compared to proposed standard pavement life. Summary tables of pay adjustment factors are included in the chapter on recommendations.

17. Key Words	18. Distribution Statement
Asphalt, concrete, pay adjustment, fatigue, permanent deformation	No restrictions. This document is available to the public through National Technical Information Service, Springfield, VA 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	152	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

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ACKNOWLEDGEMENTS

This is the last report in a series of reports concerned with the impact of variations in material properties on asphalt pavement life. The authors are grateful for the cooperation of all the agencies which participated in this study. Assistance in the preparation and testing of samples, and in assembling the final report was provided by Jose R. Montalvo and Michael Wynkoop, both students of Oregon State University.

The project was conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration.

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1.0 INTRODUCTION

1.1 PROBLEM DEFINITION

One of the major difficulties in asphalt concrete pavement construction is the control of materials and the subsequent level of payment to contractors for materials not conforming to specifications. Quite often, it is found that the material quality does not meet specification requirements. The effect of this noncompliance on pavement serviceability is not fully established; however, it frequently results in reduced payments to contractors which in turn causes much controversy between the two parties (i.e., client and contractor).

Specific pavement properties outside specification tolerance that are accepted by a majority of the agencies through reduced payment include percent compaction, asphalt content, asphalt properties, and mix gradation [1]. Some agencies reject construction and materials that do not meet specifications and pay nothing to contractors. Most agencies accept construction and materials outside specification tolerance and apply a pay adjustment that reduces the compensation to penalize the contractors. The pay factor relies heavily on the experience and judgement of project engineers. Therefore, there is a wide disparity in the pay adjustment factors currently used.

Several approaches have been used to determine pay factors for each material's property, but they are not always based on sound engineering approaches. So, the pay factors do not always measure reduced serviceability and the agencies do not have a uniform procedure for accepting noncompliance work. With all these reasons, the contractors object to the use of pay adjustment systems. This leads to administrative and legal problems. The agencies lose considerable time and money every year handling these problems.

Past practices indicate that the extent to which the quality of work should be accepted or rejected is inconclusive. Also the properties that can be allowed to be lower than the standard are poorly defined, and the compensation that should be paid for substandard quality is not well established. This study aims at finding a relationship between asphalt concrete quality and pavement serviceability, which would indicate whether the work should be accepted or rejected and how much compensation should be paid.

1.2 PURPOSE

The purpose of this study is to develop rational pay adjustment factors for asphalt concrete mixtures. Specifically, the objectives are to:

1. assess the most important factors that affect properties of asphalt concrete mixtures;
2. evaluate the effect of these variations on pavement life;
3. develop rational pay factors consistent with engineering principles; and
4. compare traditionally used prescription quality control techniques to that of the newly accepted statistical quality control techniques.

1.3 SCOPE OF REPORT

The approach used in the conduct of this study is shown in Figure 1-1. The conventional pay factors and the approaches used in the past are discussed as the state-of-the-art in Chapter 2.0. Questionnaire results and local pay adjustment factors are also included. In Chapter 3.0, the fatigue and permanent deformation characteristics of asphalt concrete mixtures are reviewed and the results of the Oregon DOT study are summarized. Regression analyses of the data from North Oakland-Sutherlin, Castle Rock-Cedar Creek, and Warren-Scappoose projects were performed using the Statistical Interactive Programming System (SIPS) of Oregon State University. In Chapter 4.0, the approaches considered for pay adjustment factors are discussed. The rational pay adjustment factors were developed based on mix fatigue performance for each mix property and the results compared with those used by other agencies. Conclusions and recommendations based on the results of the analysis for developing pay adjustment factors for asphalt concrete mixtures in pavement construction are in Chapter 5.0.

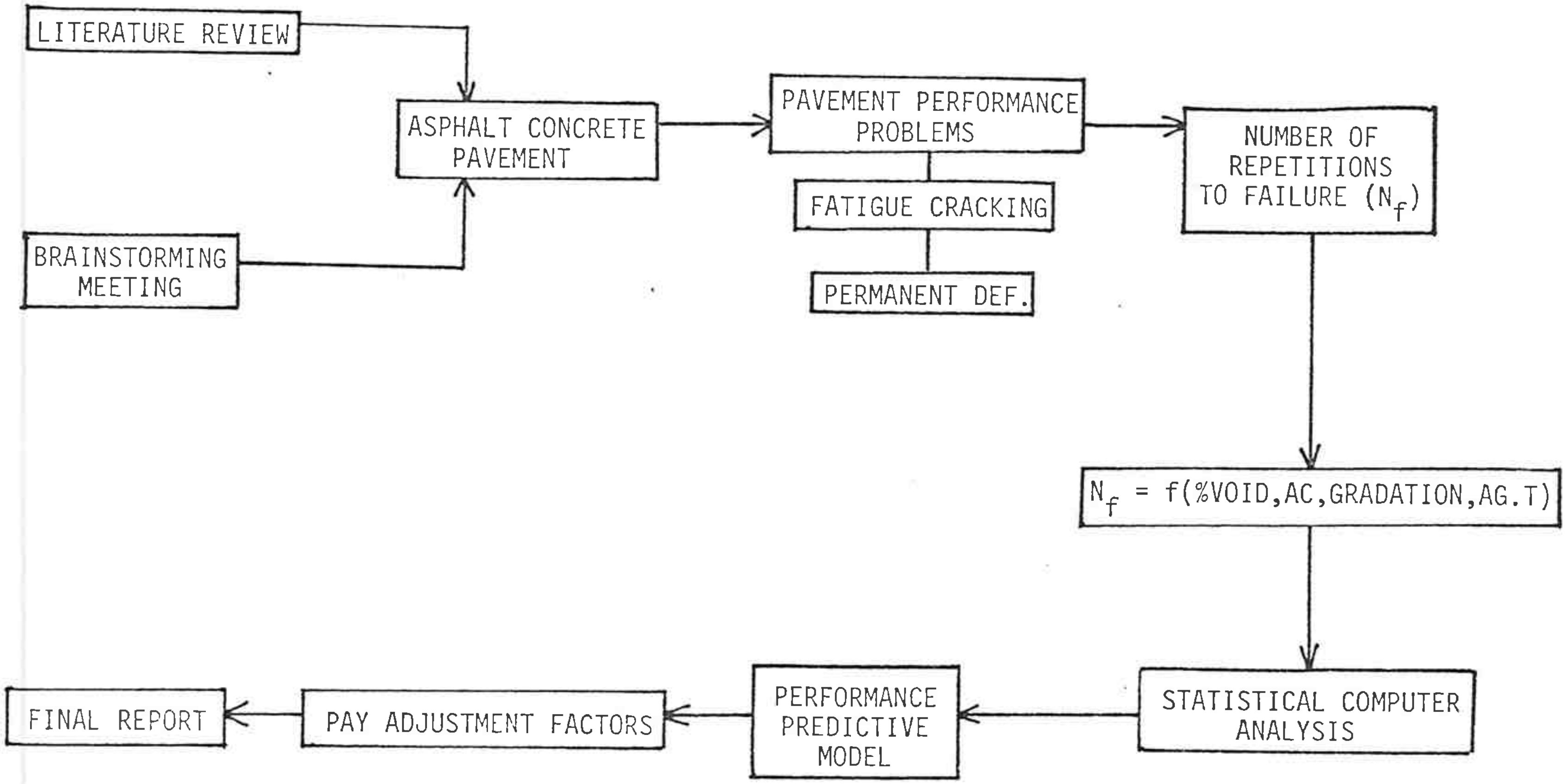


Figure 1-1. Research Approach

2.0 PAY ADJUSTMENT FACTORS STATE-OF-THE-ART

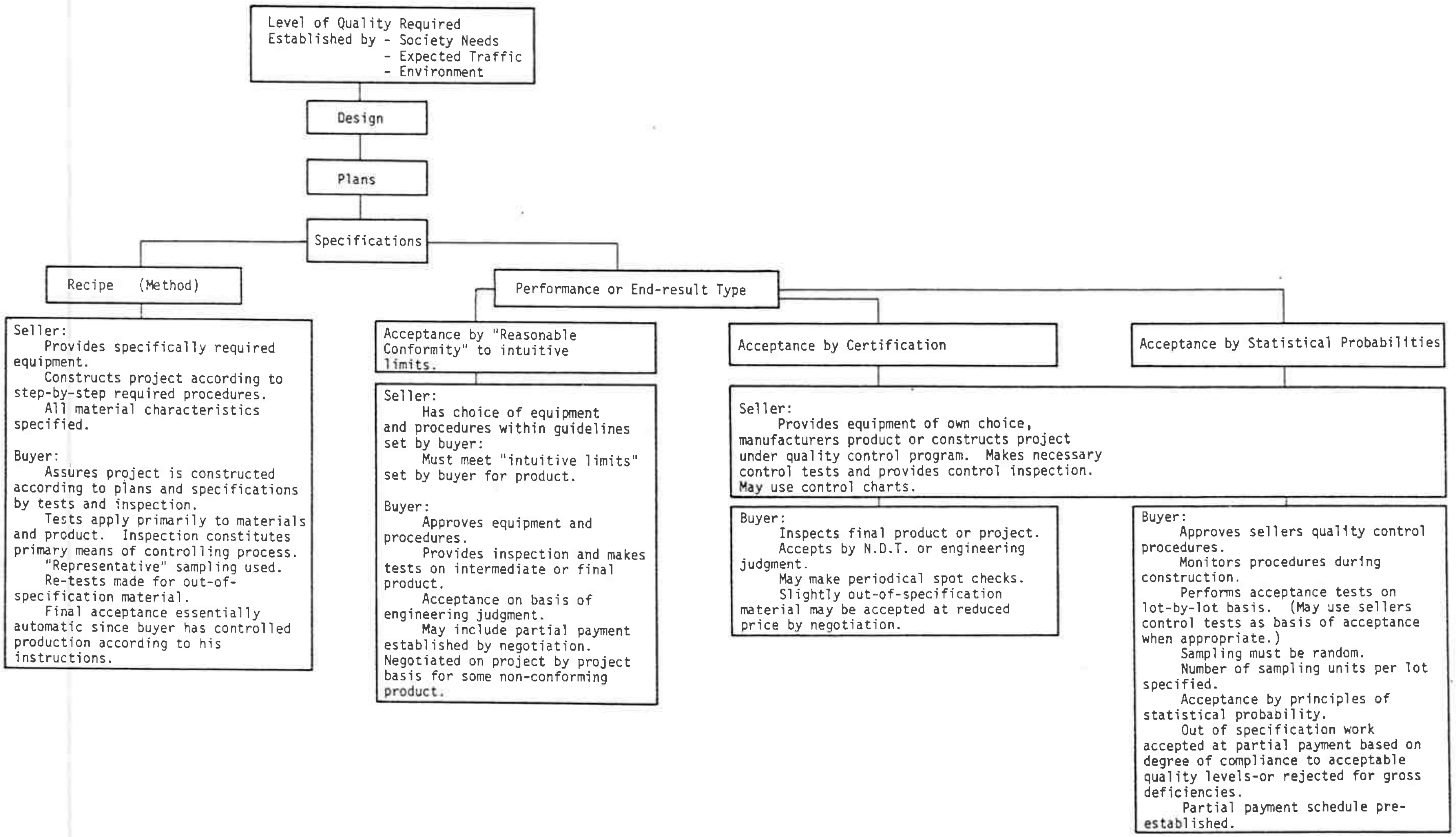
2.1 INTRODUCTION

The proper performance of highway structures is necessary to insure economical and safe transportation facilities for the public. The performance of any highway structure is linked closely to the system utilized during construction to assure quality. Poorly controlled construction and poor quality materials can nullify any of the benefits derived from detailed and superior roadway design. Conventional methods of controlling material and construction quality used are: (1) Recipe (Method) and (2) Performance or End-Result Type. The typical procedures for quality assurance systems for highway construction are presented in Figure 2-1. Under these systems (except the Acceptance by Statistical Probabilities), the control testing of quality is primarily the responsibility of the engineer. Even though the work is checked with appropriate tests whenever the inspector or engineer requires it, the quality of the construction based on these tests is determined by the judgement of the engineer. Completed projects are either accepted or rejected on engineering judgement only.

There are several inherent disadvantages associated with these methods of quality control, as follows:

1. With the method specifications, an agency buying the construction product is quite often put into a difficult position when a product they have "controlled" does not meet the specifications they set forth.
2. With the method specifications, the contractor does not get a chance to use his technical expertise in control-

Figure 2-1. Quality Assurance System in Highway Construction [2]



ling the project because the buying agency controls the specifications.

3. For both methods, the contractor has to deal with both different production tolerances and with the different engineering personnel of various agencies. It is highly unlikely that all engineers will make the same decision. This puts the contractor in the position of knowing that his work, which may be acceptable to one specification or engineer, is not acceptable in a similar situation to another.

4. The inherent variability of all material and construction processes may not be properly evaluated by the method specification. Although the quality control method itself does not affect the variability of the construction process or material, a statistical type of quality control procedure can better define the limits of this variability.

5. In utilizing the two conventional quality control processes, the contractor may not know what is achievable in highway construction. The results produced by the AASHO Road Test in 1959-60 have shown that even with well-equipped laboratories, well-trained inspectors, and competent contractors, in some cases specifications cannot be met.

The AASHO Road Test also suggests that reasonably acceptable work which does not meet specifications should be accepted at a reduced rate.

2.2 APPROACHES USED

In the recent past, a system based on mathematical justifications has filtered its way from other engineering disciplines into highway construction. Statistical methods utilize end-result specifications which are based on engineering judgement, experience and intensive research, along with the incorporation of statistical concepts [4]. Design target values are essentially the same as for the old methods for any particular highway structure, but the limits of the control specifications utilize statistical concepts that recognize the existence of natural variability. Incorporated into this new quality control procedure are the concepts of random sampling, probability theory, and known forms of data distributions.

2.2.1 Statistical Concepts

The control of construction of highway structures involves checking the values of certain properties of the finished products. Since large quantities of materials are involved, it is impractical to measure the value of a particular property at every location or to measure a particular property of each object used in construction. Therefore, a "sample" is usually measured to determine the characteristics of the entire mass of material called the "population." To ensure unbiased sampling, random samples should be taken with the aid of a random number table or other random methods. Stratified random sampling (random sampling of sublots) may be used to avoid clustered samples [3,4,5,6,7]. Any set of data obtained in a random fashion possesses several properties important to the understanding of applied statistics; two of these are the central tendency and the dispersion.

The central tendency of a set of data is the property that reflects the bunching of a set of data around a particular value. The most commonly used property in highway construction is the arithmetic mean which is defined as:

$$\text{Arithmetic Mean} = \frac{\text{sum of the measurements}}{\text{the number of measurements}} \qquad (2\text{-}1)$$

The dispersion of a data set describes the degree of scatter of individual data points. Three means of defining the dispersion of a data set are commonly used. They are the range, the variance, and the standard deviation. The range is the difference between the largest and the smallest value in the data set. The range is quite easy to determine, yet it tells nothing of the distribution of the data points between the largest and the smallest value. The variance is defined as the average of the squares of the numerical difference of each observation from the arithmetic mean. The standard deviation is defined as the square root of the variance [5,8].

In order to understand the relation that distributions of data has to quality control methods, it is essential to understand fully at least one type of data distribution. When an understanding of the distribution is gained, it is then possible to see how statistical concepts can be used to define construction specifications. Although there are many types of specifications which can be developed utilizing statistical concepts, the one discussed herein is of the "percentage within limits" variety [6].

The distribution function of the population is considered to be a normal distribution (see Fig. 2-2). Actual field data on pavement quality control generally fit a normal distribution curve (bell-shaped distribution function).

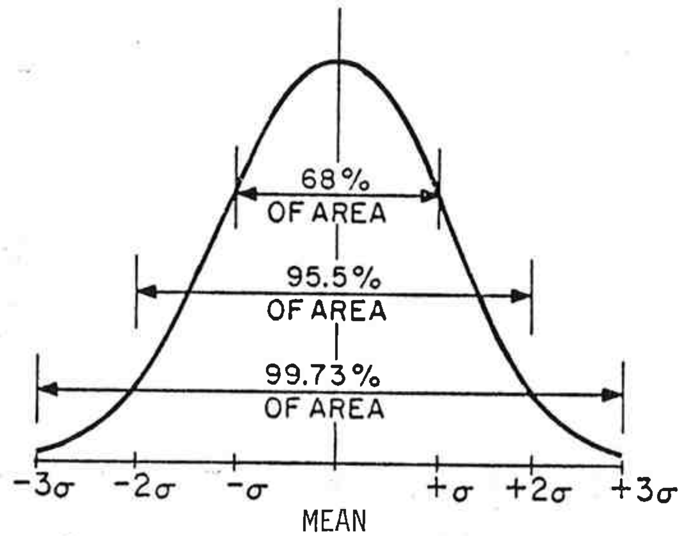


Figure 2-2. Standard normal distribution curve and the percentage of the area under the normal distribution which are within certain sigma limits [6].

The area under the normal distribution represents the frequency within which a value can occur. The normal distribution in theory, extends to infinity along the horizontal axis in both directions. It includes all the possible values of the property in question that can occur. Therefore, the area under the curve is equal to 100 percent or one. A normal distribution curve for a population is described by two variables - the mean and the standard deviation. The population mean (μ) indicates the location of the peak of the curve; the population standard deviation (σ) indicates the width of the bell-shaped curve. A low value for σ indicates a narrow curve and a well-controlled product (see Fig. 2-2). Given μ and σ, the percentage of a population falling within any given range can be computed. For instance, $\mu \pm \sigma$, $\mu + 2\sigma$, and $\mu \pm 3\sigma$

contain approximately 68, 95.5 and 99.73 percent of the total population, respectively.

Where a finite number of random samples are obtained, the mean is an unbiased estimator of the population mean and is represented by $\bar{x}$. Similarly, the random sample standard deviation represented by s is an unbiased estimator of the population standard deviation [5,7,8]. The sample mean $(\bar{x})$, which is simply the arithmetic mean, is defined by:

$$\bar{x} = \sum_{i=1}^{n} x_i/n \tag{2-2}$$

where x_i is an individual test value (the test result of the i-th sample) and n is the total number of t^st^s. The sample variance is defined by:

$$s^2 = \frac{1}{(n-1)} \sum_{i=1}^{n} (x_i - \bar{x})^2 \text{ for small sampling } (n < 30) \tag{2-3}$$

Coghlan [9] suggests that the total sample variance also includes the variation from testing, sampling, and production. Standard deviation, an indication of the bell width, is defined as the square root of the total sample variance.

$$s = \sqrt{\frac{1}{(n-1)} \sum_{i=1}^{n} (x_i - \bar{x})^2}$$

or

$$s = \sqrt{\frac{1}{(n-1)} \left[\sum_{i=1}^{n} x_i^2 - n\bar{x}^2 \right]} \tag{2-4}$$

This parameter is the tool used by engineers to define the acceptable limits of the samples about the sample mean.

2.2.2 Acceptance Plans

The engineer must develop an acceptance plan that can be thoroughly understood by the contractor for this approach to be effective. Basically, it must specify the following requirements:

1. The Size of a Lot to be Sampled. The lot is the basic unit of the construction that is controlled under the plan. The size of the lot varies from job to job depending on the type of construction. The common sizes of lots are often chosen by the approximated amount of daily production by a contractor or the number of tests that would usually be conducted per day.
2. The Number of Samples Collected from Each Lot. Generally, each lot is divided into a number of equal sublots, and one sample is collected from each sublot. White et al. [10] recommend that the four samples selected at random from four equal sublots could minimize the expense of testing and give a satisfactory level of confidence.
3. The Point Where the Sampling is to be Conducted. For the purpose of good representative samples, the sampling is usually done at random from in-place material. The hot mix samples should be taken from behind the spreader [10]. The purpose of this is that if additional samples need to be obtained, it will still be possible to obtain random samples.
4. The Sampling Method. Simple random sampling by using a random table is typically used to ensure equal chance of

selection. Another method of random sampling is stratified random sampling which is done by dividing the lot into equal sublots and taking random samples from each sublot.

5. <u>The Size of the Samples</u>. Size is dependent upon the test method used. Generally, each test method requires an appropriate sample size. For example, a diametral test to investigate mix behavior requires a four-inch diameter by 2.5-inch high core sample of asphalt concrete mixture.

6. <u>The Target or Desired Value of the Property Being Controlled</u>. The properties that are commonly emphasized for asphalt concrete mixture in most agencies include asphalt content, density, and gradation.

7. <u>The Test Method Used to Evaluate the Specified Property (AASHTO, ASTM, etc.) Should be Clearly Stated</u>. For example, the test for bitumen content is carried out according to the AASHTO T-164, Method B (reflux method).

8. <u>The Realistic Tolerance for the Target Value</u>. Tolerances above and below the target value should be defined. These tolerances can be set by experience from previous projects, acceptable design value or other factors affecting the required performance of the structure. Tolerances should take into account the normal material and testing variations since they vary from sample to sample in a lot.

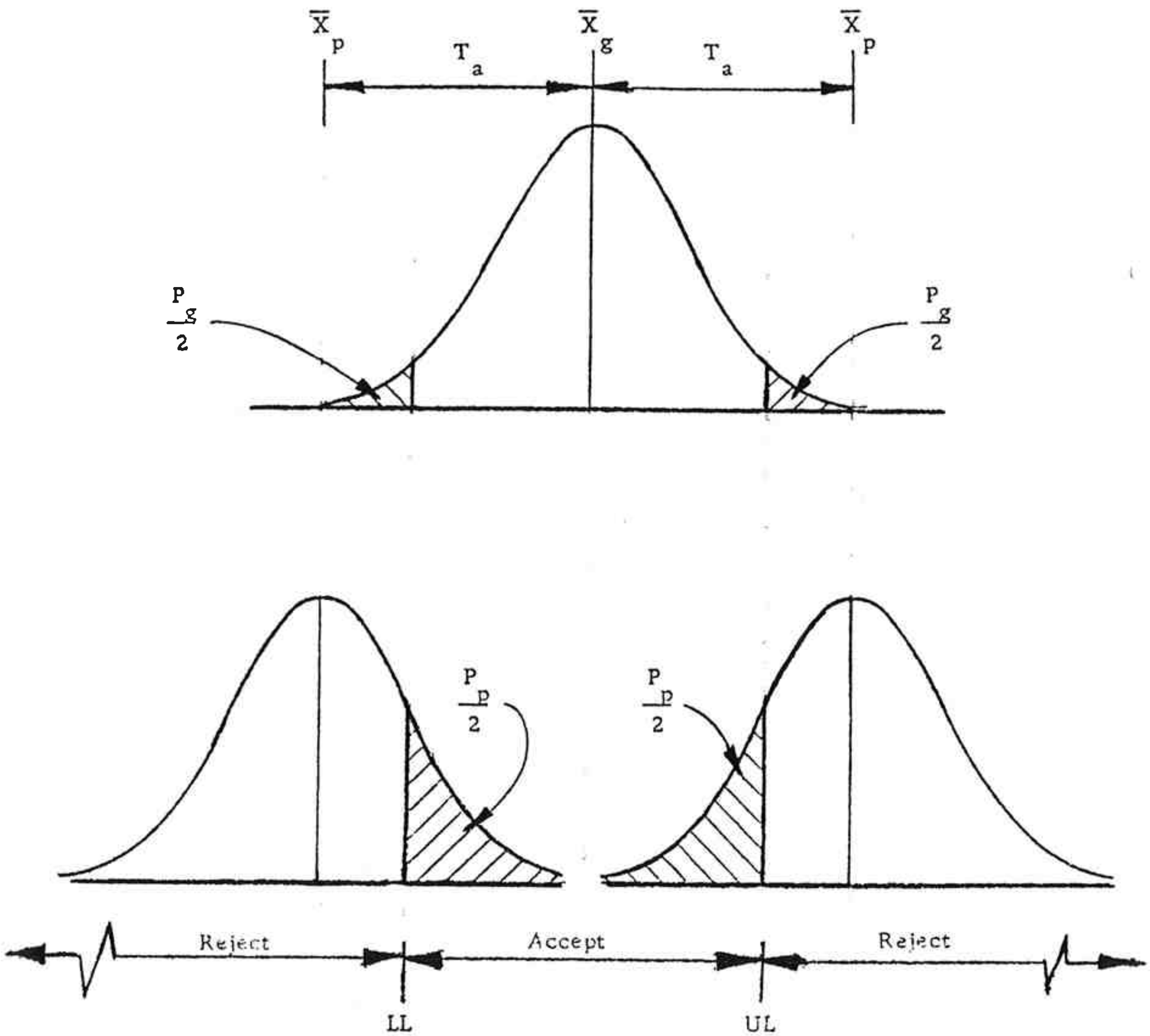
9. The Action to be Taken in Case of Non-Compliance with Specifications. The usual procedure is to use reduced payment plans. In the "percent within limits" type of specification, this takes the form of reduced payments when certain percentages do not meet the requirements. In situations where the construction is far from satisfactory, the material must be removed and replaced.

There are two generally accepted approaches to establishing the acceptance limits. The first approach is assigned numerical values based solely on engineering judgement. The permissible range for specified properties is determined by means of a theoretical or experimental procedure. It may be possible in some cases to derive this value, but this is not generally done. Thus, this assignment often leads to misplacement of acceptance limits and economic hardships for contractors. A more practical approach uses limits taken from field samples on satisfactorily performing construction.

The probability of accepting an inferior product or of rejecting an adequate product is always a finite amount, as shown in Figure 2-3. However, as the mean of an unacceptable material varies from the mean of the acceptable material the probability of accepting this poor material through limited sampling becomes very small.

2.2.3 Quality Control

Control charts are graphical devices used in the quality control process to indicate trends of deviation of the sample mean from the specified mean under field conditions. The target value of a control chart represents the theoretical population average (μ) of normal curve. Upper and lower control limits are obtained from the standard



$\overline{X}_p$	Mean of undesired, poor material.
$\overline{X}_g$	Mean of desired, good material.
T_a	Allowable tolerance between good and poor material.
P_g	Probability of rejecting good material (producer risk).
P_p	Probability of accepting poor material (purchaser risk).
LL	Lower specification limit.
UL	Upper specification limit.

Figure 2-3 Relationship Between Limits, Risks and the Normal Distribution of Population of Materials [9]

deviation of the population (σ) and the degree of confidence desired. A process is considered out of control when one test value on any chart is not in between the upper and lower limit. An example control chart typical of those used where a target design value is specified, along with tolerance limits, is shown in Figure 2-4.

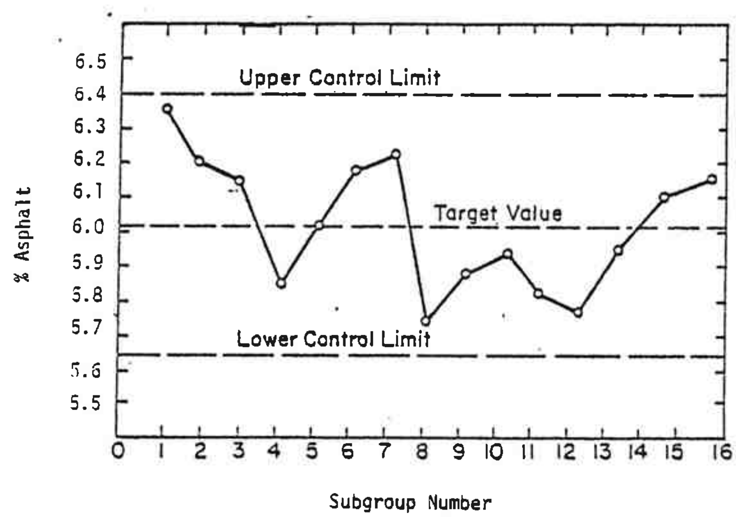


Figure 2-4. Control chart for asphalt content.

2.2.4 Payment Reduction

Sample data are not always acceptable or rejectable. Quite often they are somewhere between acceptable and unacceptable limits. For this situation a reduced payment system is used in accepting an unsatisfactory job, although most agencies utilize different percentage and rate schedules developed according to what they feel is equitable or representative of reduced service life of inferior quality construction. Kopac [11] states that price adjustment factors can be broken down into three categories: (1) those that are based on sample mean, (2) those that are based on sample mean and an estimate of variability, and (3) those that are based on other bases. Typical examples of these three categories of price adjustment factors are shown in Table 2-1.

Three approaches have been used to develop pay adjustment factors: (1) the serviceability approach, (2) the cost of production approach, and (3) the operating characteristic curve approach. The serviceability approach relates pavement reductions to expected losses in pavement performance to reduced quality of construction. Several agencies have used a serviceability approach for one or more of their price adjustment factors [1]. Two examples are the thickness pay factors for asphalt concrete pavements of the Illinois and New Jersey Department of Transportation and the smoothness pay factors of Florida DOT. Willenbrock et al. [12] stated that pay adjustment factors based on serviceability is highly desirable, but it is not always possible to develop these factors due to the lack of desired correlation between the quality characteristic and pavement performance, or between performance data relating the quality characteristic and maintenance-free pavement life. The cost of production approach is only used for acceptance characteristics that lower acceptance limits [12]. In this approach, the payment reduction is usually greater than the reduction in cost that results when a lower quality job is being produced. It concludes that the penalty should be greater than what the contractor saves in the material costs in producing low-quality work. The operating characteristic curves approach uses a set of curves which show probabilities of applying price adjustments to various lots having different levels of quality. The sample of operating characteristic curve for asphalt concrete pavement density is shown in Figure 2-5. An agency can use an

Table 2-1. Typical Price Adjustment Factors for Asphalt Concrete Pavement Construction [12].

(a) Price Adjustment Factors Based on Sample Mean Mississippi Bituminous Concrete Density

% Target Density	% Pay
94.0-100	100
94.2- 94.8	90
93.5- 94.1	70
92.8- 93.4	50
92.8	

(b) Price Adjustment Factors Based on Sample and Estimate of Variability

(1) Continuous Price Adjustments in Use in Several States (e.g., Colorado, Alaska, Washington, New Mexico and Wyoming)

$P = (\bar{x}_n + aR - T_u)F$ for upper limit specification, and

$P = (T_L + aR - \bar{x}_n)F$ for lower limit specification;

where:

p = percent reduction

$\bar{x}_n$ = average of n values

a = variable dependent on n

R = highest minus lowest value

T_u = upper specification limit

T_L = lower specification limit

F = price reduction factor (dependent on quality characteristic)

(2) Pennsylvania Percentage Within Limits for Asphalt Content

PWL	% Pay
90-99	95
85-89	90
80-84	80
75-79	70
70-74	60
65-69	50
65	Reject

(c) Other

(1) Florida Attributes Schedule for Smoothness

No. Defects Per Lot	% Pay
1	100
2-3	95
4-5	90
6-7	85
7	75

(2) Vermont Continuous Measurement Schedule for Smoothness

Roughometer in./mi	% Pay
0-100	100
101-110	98
110	95

operating characteristic curve and curve of the contractor's expected payment (see Fig. 2-6) to ensure that the proposed schedule is realistic and meets the desires of the specifying agency. If the agency is satisfied with the contractor's prepared curve, then the schedule can be incorporated into the specification. If not, then either the price adjustment schedule or the acceptance plan must be modified by means of changing the sample size, loosening or tightening the acceptance or specification limits, increasing or decreasing payment for a given quality level, and increasing or decreasing the number of payments. Appropriate modifications are made until both contractual parties are satisfied. Therefore, this approach is based upon the agreement of both parties and the experience and judgement of the agency's engineer. This approach seems to be very flexible, but it is doubtful that pay factor indicates serviceability and pavement performance.

2.3 QUESTIONNAIRE RESULTS

In the Fall of 1979, the Oregon State Highway Division and Oregon State University initiated a research project to study the impact of variations in material properties on asphalt life. The questionnaire was developed and distributed to all state agencies, the District of Columbia, and the Federal Highway Administration. Each agency was asked to respond to seven questions with reference to their current method for acceptance or rejection of asphalt concrete paving materials. The items of emphasis on the questionnaire included:

1. Acceptance of noncompliance construction and materials with or without pay adjustment factors.

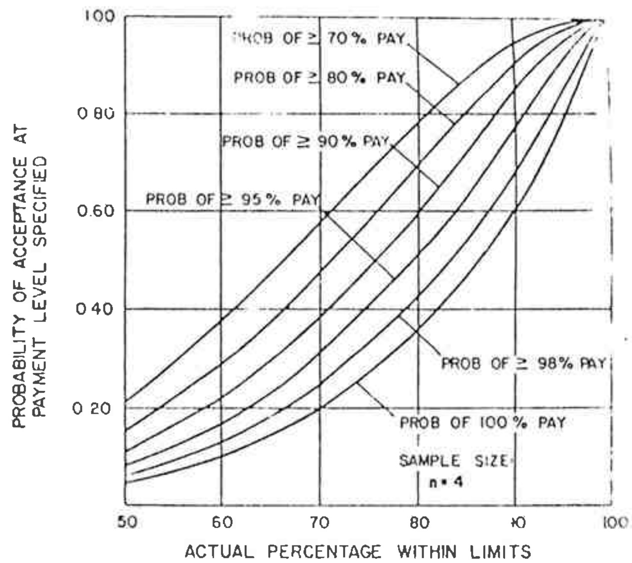


Figure 2-5. Operating Characteristics Curves
for Density Acceptance Plan of FAA [after Ref. 12]

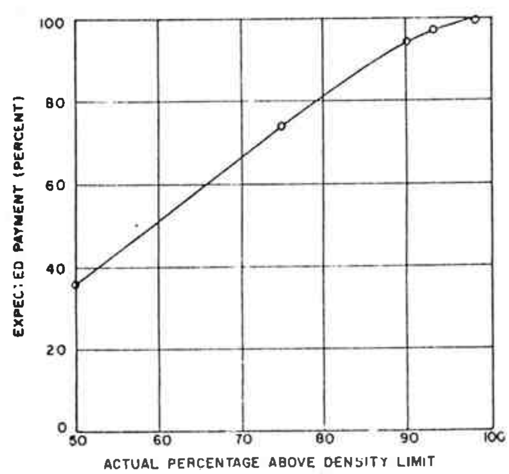


Figure 2-6. Expected Payment Curve for Penn DOT
Acceptance Plan for Bituminous Concrete Pavement Density
[after Ref. 13]

2. Identification of properties tested for acceptance and the method of test used.
3. Pay adjustment factors used in relation to each tested property.
4. Rationale used in establishing pay adjustment factors.
5. Relationship of pay adjustment factors to pavement serviceability or other criteria.
6. Effectiveness of pay adjustment factors in encouraging compliance with specifications.
7. Summary opinions regarding the use of pay adjustments.

The data from the questionnaire were summarized and the analysis of the results indicate that [1]:

1. Most state agencies accept one or more properties in the construction and materials of asphalt concrete pavement that are outside specification tolerances.
2. The specific properties accepted outside of specification tolerances with pay adjustment factors are mix gradation, asphalt properties, and compaction and asphalt content for most state agencies. The aggregate quality, smoothness and pavement thickness are additional properties accepted outside of specification tolerances.
3. Most state agencies accept non-compliance work by reduced payment to contractor. It is significant that the current philosophy is to penalize the contractor for unsatisfactory properties. A few agencies are considering the provision of a bonus for high quality and uniform work.

4. The predominate criterion relied on for establishing pay factors is experience.
5. Only 26 percent of the agencies consider their pay factors are proportional to pavement serviceability. Other widely used rationale for pay factors are: (1) to discourage noncompliance by application of the penalty, and (2) to comply with recommendations of the FHWA.
6. Half of the state agencies consider the use of pay factor plans as effective in encouraging compliance with specifications. The remaining agencies either do not use specified pay factors or they do not believe the plans currently available are sufficient.
7. There is still a wide disparity in the pay adjustment factors currently used by the different state agencies. Several approaches are used to determine pay factors for each material property evaluated. Even though the agencies used the same approach, they arrived at widely varying values for the pay factors applied to a common level of material quality.

2.4 LOCAL PAY ADJUSTMENT FACTORS

Pay adjustment factors have been included in specifications for a long time, but the criteria used in accepting finished work and in making payment vary from agency to agency. Each agency has its own standard based on different approaches about the need to apply pay adjustment factors.

Table 2-2. Pay Adjustment Factors for Bituminous Plant-Mix Surfacing [14].

MIXTURE CHARACTERISTIC	PAY FACTOR	PERCENT DEVIATION OF THE MEAN FROM THE TARGET VALUE					SINGLE VALUE DEVIATION*
		1 TEST	2 TESTS	3 TESTS	4 TESTS	5 TESTS	
BITUMEN CONTENT	1.00	0-0.7	0-.54	0-.46	0-.41	0-.38	0.50
	0.95	0.8	.55-.61	.47-.52	.42-.46	.39-.43	0.50
	0.85	0.9	.62-.68	.53-.58	.47-.51	.44-.47	0.50
3/4" SIEVE	1.00	0-10	0-7.1	0-6.1	0-5.4	0-5.0	6
	0.95	11-12	7.2-8.1	6.2-6.9	5.5-6.1	5.1-5.6	6
	0.85	13	8.2-9.1	7.0-7.7	6.2-6.8	5.7-6.2	6
NUMBER 4 SIEVE	1.00	0-9	0-6.7	0-5.7	0-5.2	0-4.8	6
	0.95	10	6.8-7.6	5.8-6.3	5.3-5.8	4.9-5.4	6
	0.85	11	7.7-8.5	6.4-6.9	5.9-6.4	5.5-5.9	6
NUMBER 8 SIEVE	1.00	0-7	0-5.6	0-4.8	0-4.3	0-4.0	5
	0.95	8	5.7-6.3	4.9-5.4	4.4-4.8	4.1-4.5	5
	0.85	9	6.4-7.0	5.5-6.0	4.9-5.3	4.6-4.9	5
NUMBER 40 SIEVE	1.00	0-6	0-4.5	0-4.1	0-3.7	0-3.5	3
	0.95	7	4.6-5.1	4.2-4.5	3.8-4.1	3.6-3.8	3
	0.85	7	5.2-5.6	4.6-4.9	4.2-4.4	3.9-4.1	3
NUMBER 200 SIEVE	1.00	0-3.0	0-2.4	2-2.0	0-1.8	0-1.7	2.5
	0.95	3.1-4.0	2.5-2.7	2.1-2.2	1.9-2.0	1.8-1.9	2.5
	0.85	3.1-4.0	2.8-3.0	2.3-2.4	2.1-2.2	2.0-2.1	2.5
SAND EQUIVALENT	1.00	When no single test value is less than 35					NOT APPLICABLE
	0.95	When the mean of the test values is 35 or above and one or more individual test value is less than 35					

* Maximum plus or minus percent deviation of single test value from the mean of the lot results.

2.4.1 Federal Highway Administration (FHWA), Region 17 [14]

The mix characteristics involved in the acceptance of finished asphaltic concrete pavement are bitumen content, density (relative compaction), aggregate gradation and sand equivalent. The random samples are taken from each lot and tested to investigate the required characteristics. If any test indicates that the lot contains deficient properties, the pay adjustment factor will be applied.

The acceptance schedule for required characteristics is based on sample mean and deviation of the lot results. The pay adjustment factors for bitumen content, aggregate gradation and sand equivalent are presented in Table 2-2. If the pay factor for the lot is less than 0.85, it will be rejected or paid at a lower pay factor (0.70), subject to the engineer's judgement.

The mean and range in density of the materials are calculated from the results of random samples to determine acceptable mix density. The pay adjustment factor for density for each lot is evaluated as shown in Table 2-3.

FHWA still follows the method specification in controlling pavement construction. Therefore, not only is the finished work controlled, but the construction process is also checked closely. Although FHWA has been using statistical techniques in the acceptance schedule of finished work, the contractor is still limited in selecting his own techniques in the construction process. The benefit of using statistical concepts is that it gives the contractor a chance to get his work approved.

Table 2-3. Pay Factors for Density or Relative Compaction (Mean of All Acceptance Test Results) [14].

Pay Factors	1.00	0.95
Number of Acceptance Samples		
5 to 10	96% + 0.22R or higher	Less than 96% + 0.22R

R (Range): The difference between the highest and lowest values of the test results from a given lot.

Mean : Arithmetic mean or average of individual test results for a lot.

2.4.2 U.S. Forest Service [15]

The U.S. Forest Service uses the end result specification as the criteria for accepting asphalt concrete pavement. Thus, the responsibility of controlling quality of mixture lies with the contractor. The engineer may spot-check from time to time, but the results of these tests are not used for accepting the work. Generally, in the acceptance process, five random samples are taken from a lot which is defined as the number of materials approximately 2,000 tons, or one day's production. The acceptance of the final product is based upon the mean of the test results and on a lot basis. The mix properties required for acceptance are bitumen content, gradation, and compaction. Materials which do not meet the requirements, but which are in substantial compliance, are accepted at an adjusted unit price. The acceptance schedule for bitumen content and gradation are presented in Table 2-4 and for compaction in Table 2-5.

The payment for materials in the lot is the product of the minimum pay factor from Table 2-4, Table 2-5, and the contract unit price. The final product is rejected if the product of the pay factor is less than 0.6 for compaction and 0.7 for other properties, and the engineer may require that lot to be removed from the job at the contractor's expense.

Table 2-4. Acceptance Schedule - Bituminous Plant - Mix Bitumen Content and Gradation [15].

MIXTURE CHARACTERISTIC	PAY FACTOR	MAXIMUM PERCENTAGE POINTS OF THE MEAN FROM THE TARGET VALUE	MAXIMUM PERCENTAGE POINT DEVIATION FROM THE TARGET VALUE
Bitumen Content	1.00	0 - 0.28	0.30
	0.95	0.29 - 0.34	0.50
	0.90	0.35 - 0.41	0.70
	0.70	0.42 - 0.50	1.00
3/8" & Larger Sieves	1.00	0 - 5.0	5.0
	0.95	5.1 - 5.5	5.5
	0.90	5.6 - 6.2	6.5
	0.70	6.3 - 7.5	8.0
1/4" or No. 4 Sieve	1.00	0 - 4.8	5.0
	0.95	4.9 - 5.4	5.5
	0.90	5.5 - 5.9	6.5
	0.70	6.0 - 7.0	8.0
No. 8 or No. 10 Sieve	1.00	0 - 4.0	5.0
	0.95	4.1 - 4.5	5.5
	0.90	4.6 - 4.9	6.5
	0.70	5.0 - 5.8	8.0
No. 16 - 50 Sieves	1.00	0 - 3.5	5.0
	0.95	3.6 - 3.8	5.5
	0.90	3.9 - 4.1	6.0
	0.70	4.2 - 4.7	7.0
No. 200 Sieve	1.00	0 - 1.7	2.0
	0.95	1.8 - 1.9	2.7
	0.90	2.0 - 2.1	3.3
	0.70	2.2 - 2.5	4.0

Table 2-5.
Acceptance Schedule - Bituminous Plant - Mix Compaction [15].

Pay Factor	Percent of Relative Maximum Compaction of the Mean From Job Mix - Formula	Minimum Percent Relative Compaction
1.00	95.0	92
0.87	92.5 - 94.9	90
0.61	90.0 - 92.4	88

2.4.3 Washington State Department of Transportation (WDOT) [16]

WDOT does not fully use pay adjustment factors in highway pavement construction. The only pay adjustment factor used is for percent of maximum theoretical density (AASHTO T-209) as a penalty schedule [16]. Thus, the only mix property of concern with the WDOT approach is density. The other penalties for other mix properties are still based on either rules-of-thumb or the Colorado penalty [16]. The completed work is accepted on a lot-to-lot basis. A lot consists of five approximately equal sublots that are 1,000 linear feet of paving lane long or not to exceed one day's production. One random sample is taken from each sublot. The average of the sample densities from a lot is used in an acceptance schedule and payment is in accordance with Table 2-6.

Table 2-6. WDOT's Pay Adjustment Schedule for Percent of Maximum Theoretical Density (AASHTO - T-209) [16].

% Rice	% Pay
92.0 and above	100
91.0 - 91.9	95
90.0 - 90.0	90
89.0 - 89.9	80
Below 89.0	50 (maybe)

2.4.4 City of Portland [17]

The properties of asphaltic concrete mix considered in establishing pay adjustment factors for the City of Portland are mix gradation, asphalt content, thickness, and class of asphalt concrete. The deficiency in thickness of asphalt concrete appears to be considered most in the acceptance plan. The payment concerning thickness is on a square yard basis. The payment is made at an adjusted price as specified in the following table.

Table 2-7. City of Portland Price Adjustment for Deficiency in Thickness of Asphalt Concrete Pavement [17].

Deficiency in Thickness Inches	Proportional Part of Contract Unit Price Allowed
0.00 to 0.20	100 percent
0.21 to 0.30	80 percent
0.31 to 0.40	72 percent
0.41 to 0.50	68 percent
0.51 to 0.75	57 percent
0.76 to 1.00	50 percent

The pay adjustment factor for other properties is made by deducting one percent of the in-place price for each one percent cumulative deviation from the allowable tolerance of each component of the job mix formula required by the specification. The deviation in asphalt content is weighted for eight times, and two times for the deviation in percent passing No. 200 sieve, of the other specified properties. The payment schedule is based on a ton of mixture and no payment for the cumulative deviation equal to exceeding 12 percent.

2.4.5 Oregon State Department of Transportation (ODOT) [18]

The main criteria used in this specification in accepting asphaltic concrete pavement construction are compaction, asphalt content, and mix gradation. According to the procedure specification used in this state not only the final product is controlled, but also the process and materials have to be proved. The acceptance plan for asphalt content and mix gradation is based on target values from the mix formula. The specification requirements in accepting asphalt content and mix gradation are shown in Table 2-8.

Table 2-8. ODOT Specification Limits for Asphalt Content and Mixture Gradation [18].

Constituent of Mixture	Tolerance (Plus or minus to mix formula)	Bonus Pay Tolerance (Plus or minus (to mix formula)
Aggregate percent passing 1/4" sieve	6.0%	4.0%
Aggregate percent passing No. 10 sieve	4.0%	3.0%
Aggregate percent passing No. 200 sieve	2.0%	1.4%
Asphalt Cement	0.5%	0.3%

The target value for density is determined from a 500-foot long control strip as controlled and specified by the engineer. The results of five density tests of random samples from the control strip are used as the target density. The specification requirements in accepting mix density is presented in Table 2-9.

Table 2-9. ODOT Specification Requirement for Mix Moisture and Density Content [18].

Constituent of Mixture	Requirement	Bonus Pay Requirement
Moisture Content	1.0% (max)	0.8% (max)
Compaction (density)	98.0% (min)	99.0% (min)

Price adjustment is applied to material or work when any of the test results of the lot do not meet the specification limits according to Tables 2-7 and 2-8. The price adjustment of asphalt content and mix gradation for the lot is determined from the following formula.

(a) Maximum Limit Specified

$$P = (\bar{x}_n + aR - T_u)F \tag{2-5}$$

(b) Minimum Limit Specified

$$P = (T_1 + aR - \bar{x}_n)F \tag{2-6}$$

where:

P = percent of adjustment in the contract price

$\bar{x}_n$ = the arithmetic means of n random samples from a lot

a = variable factor according to sample size
(n = 3, a = 0.45; n = 4, a = 0.38;
n = 5, a = 0.33; n = 6, a = 0.30;
n = 7, a = 0.28)

R = the difference between the highest and lowest values of test results in the lot

T_u = the upper or maximum specification tolerance limit

T_1 = the lower or minimum specification tolerance limit

F = the price reduction factor as given in Table 2-10.

Table 2-10. Price Reduction Factors (f) for Mix Properties [18].

Constituent	Factor "F"
All aggregate passing 1", 3/4", 1/2" and 1/4" sieves	2
All aggregate passing No. 10 and 40 sieves	3
Aggregate passing No. 200 sieve	6
Asphalt cement	22
Moisture content	11
Compaction (density)	4

The material is accepted as being in reasonably close conformity if P is negative or less than 2. If the total P value is between 2 and 15, the material is accepted at the reduced price. If the P value is greater than 15, the engineer may call to remove unsatisfactory material or replace it with acceptable material at the contractor's expense, or an adjustment in a payment at 15 percent reduction to no payment at all. A bonus payment is made for quality work as described in Table 2-8.

2.5 SUMMARY

The interaction among material, construction, and environmental factors contributes to the problem in the performance of pavements. Traditional methods of quality control for highway construction have many short-comings that have led to the search for better methods. Statistically oriented end-result specifications are becoming widely accepted. By application of statistics, a relatively small number of randomly collected samples can be used to evaluate the quality of materials and construction. Statistically oriented end-result specifications shift the quality control responsibility to the contractor, leaving the engineer free to evaluate the contractor's performance. When test results indicate that material or construction meets the desired quality, it is accepted at 100 percent payment; when it fails to meet the minimum requirements, it is rejected. When material or construction is found to be below the desired quality but above the minimum requirements, the job is paid for at a reduced cost. Many approaches are proposed for the development of a pay adjustment factor. Most have been developed primarily through judgement and further adjustments that have been made have been dictated by experience under actual conditions. The results from pay factor questionnaires have indicated that most state agencies apply a pay adjustment to accept noncompliance jobs, but still rely on experience and judgement. Pay adjustment factors, in effect, are a device to penalize a contractor for properties which do not meet specification. Actually, rather than treating the pay adjustment factors as a punitive measure, they should be looked upon as devices that reflect reduced pavement serviceability.

3.0 PAY ADJUSTMENT FACTORS--LAB TESTS

3.1 INTRODUCTION

This chapter presents the results of a search of literature related to an asphalt concrete mix performance characteristic. The results of the Oregon DOT study are discussed. The regression analysis techniques are used to analyze data from North Oakland-Sutherlin, Castle Rock-Cedar Creek and Warren-Scappoose projects. The effect of each mix property on pavement performance is studied to provide better understanding for developing pay adjustment factors.

3.2 APPROACHES

The expected serviceability or performance of the finished product is of great concern for all those involved in pavement construction. The serviceability of asphalt pavements is controlled by many factors, such as expected loads, and mixture and environmental variables. Failure of asphalt concrete pavement is the result of (1) rutting or washboarding due to stability problems, (2) progressive cracking due to fatigue problems, and (3) fracture due to strength failure. Fatigue cracking is probably the most important mode of distress for highway engineers and researchers [19]. Fatigue is the phenomenon of cracking due to a repeated stress or strain level having a maximum value less than the tensile strength of the material when subjected to a single load application [20]. The fatigue life of asphalt concrete is governed by many factors, namely, loading, base and sub-base support, climatic and environmental factors, and the asphalt concrete mixture variables. Of these factors, the asphalt mix plays an important role in the ultimate life of bituminous concrete. The asphalt content, aggregate gradation and type, air void content (density), and asphalt viscosity are all mix variables that are important when correlated to high fatigue resistance. Many of these variables are interrelated and the effect of changing one is comparable to change in another.

In this chapter, changes in mix variables are correlated to appropriate changes in fatigue resistance. From this, it is possible to determine which of the mix variables are most effective in prolonging the fatigue failures. Then the best set of mix variables are obtained as the predictive model to estimate the expected serviceability of a pavement. And, finally, pay adjustment factors are established based on the expected losses in pavement performance.

3.3 TYPICAL FATIGUE PREDICTION MODEL

The fatigue response of asphalt paving mixtures has been investigated for a number of years throughout the world [21,22,23,24,25,26]. The mix variables that most influence the fatigue of asphalt concrete pavement include binder type and binder content, aggregate gradation and type, air void content, asphalt viscosity, temperature, and the compactive effort.

Numerous studies [21,22,23,24] have indicated that there is a relationship between fatigue performance of asphalt mixes and initial tensile strain which could be considered linear when plotted on a logarithmic basis. This relationship can be expressed as:

$$N_f = K \left(\frac{1}{\varepsilon_t}\right)^C \qquad (3\text{-}1)$$

where:

N_f = number of load repetitions to failure

ε_t = horizontal elastic tensile strain

K,C = regression constants

The factors K and C depend on the composition and properties of the mix and are also affected by the testing method and temperature [25]. The mix variables that affect fatigue life are aggregate type and gradation, filler, binder type, viscosity and content, degree of mix compaction and resulting air void content. Pell [21] stated that the most primary important variables are binder content and void content.

The results of research investigations [21,22,26] indicate that mixes containing high void contents exhibit comparatively short fatigue lives. A typical result is shown in Figure 3-1.

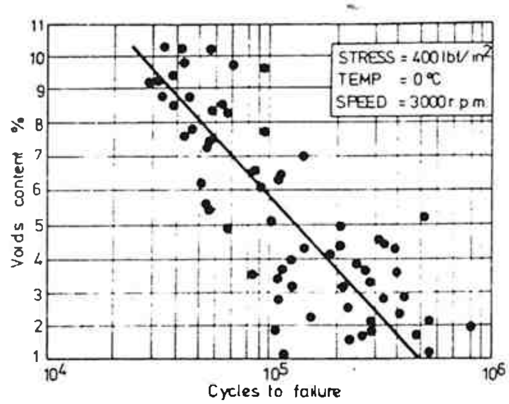


Figure 3-1. Effect of void content on fatigue life [21].

Increasing the asphalt content of a mix results in increases in fatigue life (see Fig. 3-2) up to an optimum. Further increases result in a decrease in life.

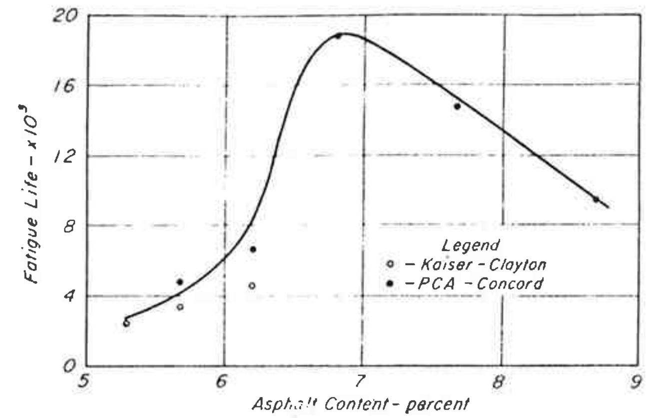


Figure 3-2. Effect of asphalt content on fatigue life California medium grading, basalt aggregate, 60-70 penetration asphalt [26].

Other variables, such as aggregate type and gradation, also affect the fatigue performance of asphaltic mixes in terms of stiffness, binder content and voids content. Pell and Cooper [22] suggested that the use of a more rounded irregular gravel generally leads to longer fatigue lives than crushed rock, as exhibited in Figure 3-3.

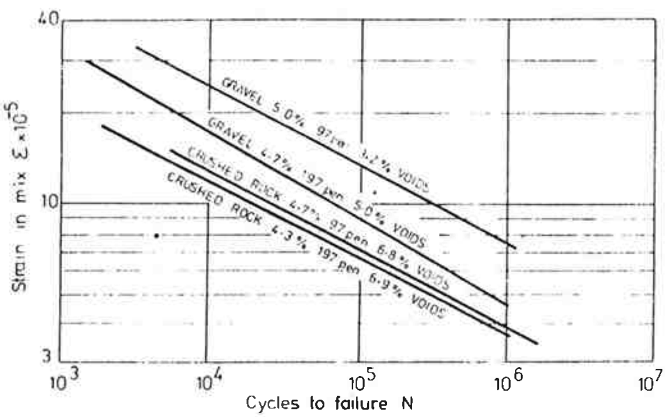


Figure 3-3. Effect of aggregate type on fatigue life of dense bitumen macadams [22].

This was due to the higher density of these mixes and smaller amount of aggregate crushing as the particles slid together more readily during the static compaction process. It is believed that mixture stiffness explains much of the variation in the fatigue relationships more than any other variables. Monismith and McLean [27] studied California type mix at five percent air voids and six percent asphalt content and emphasized the influence of stiffness on stress-strain relationship as shown in Figure 3-4.

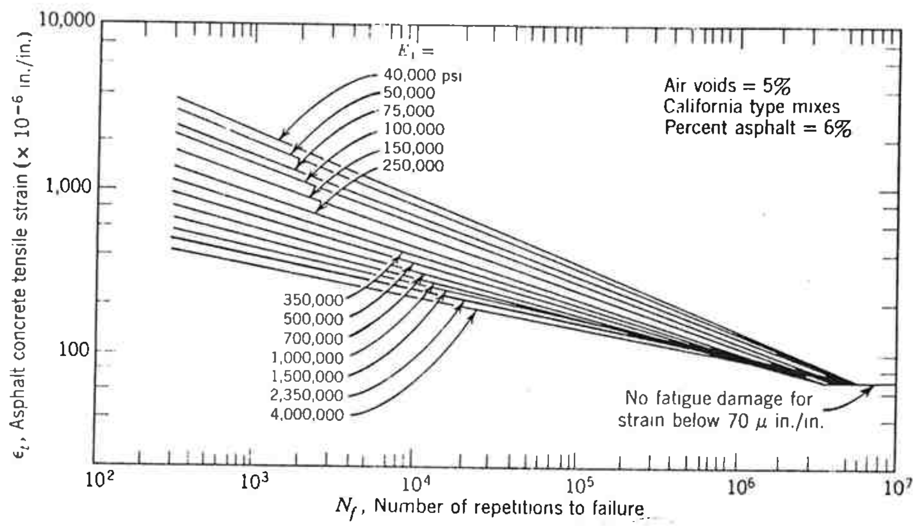


Figure 3-4. The influence of stiffness on stress-strain relationship [27].

In general, increased stiffness results in longer lives at a given stress level and shorter fatigue lives at a given tensile strain level. It appears that stiffness pays a predominant role in determining the fatigue behavior of bituminous mixes. A summary of factors affecting the stiffness and fatigue life of asphalt concrete mix is presented in Table 3-1.

3.4 OREGON DOT STUDY RESULTS

Oregon State Highway Department and Oregon State University initiated a laboratory study to establish the relationship between asphalt concrete pavement performance and mix level of compaction asphalt content, percent passing No. 200 sieve, and aggregate quality. Samples of

Table 3-1. Factors Affecting the Stiffness and Fatigue Behavior of Asphalt Concrete Mixtures [after Ref. 28].

Factor	Change in Factor	Effect of Change in Factor		
		On Stiffness	On Fatigue Life	
			In Controlled Stress Mode	In Controlled Strain Mode
Asphalt Penetration	Decrease	Increase	Increase	Decrease
Asphalt Content	Increase	Increase[1]	Increase[1]	Increase[2]
Aggregate Type	Increase Roughness and Angularity	Increase	Increase	Decrease
Aggregate Gradation	Open to Dense Gradation	Increase	Increase	Decrease[4]
Air Void Content	Decrease	Increase	Increase	Increase[4]
Temperature	Decrease	Increase[3]	Increase	Decrease

1 Reaches optimum at level above that required by stability considerations.

2 No significant amount of data; conflicting conditions of increase in stiffness and reduction of strain in asphalt make this speculative.

3 Approaches upper limit at temperature below freezing.

4 No significant amount of data.

asphalt concrete mix from the North Oakland-Sutherlin (NO-S), Castle Rock-Cedar Creek (CRCC) and Warren-Scappoose (WS) projects were prepared and tested at Oregon State University's laboratory [29,30,31]. The main types of pavement failure considered during the test program included fatigue cracking and rutting. Conventional tests and improved dynamic tests were performed to determine mix stiffness, fatigue life and permanent deformation characteristics. The percent reduction in pavement life is based on fatigue and permanent deformation characteristics from the standard mix and used as a criterion in developing pay adjustment factors.

The North Oakland-Sutherlin results were reported by Walter et al. [29]. It was found that the mix level of compaction was the controlling factor for all mix dynamic properties. Increasing the mix density increased the fatigue life and resistance to permanent deformation. Fatigue life improved when increasing asphalt content and the amount of fines were increased. Aggregate used in this project was a submarine basalt as a low-quality aggregate which may have caused the reduction in pavement life. The pay factors computed for the effects of variations in mix properties are presented in Tables 1 - 10 in Appendix A.

The Castle Rock-Cedar Creek projects, as reported by Walter et al. [30], indicated that the mix level of compaction was the most dominant factor in controlling mix characteristics. Increasing the mix density or decreasing voids content increased the mix stiffness, fatigue life, and the mix resistance to permanent deformation. An increase in the percent of fines decreased the fatigue life and resistance to permanent deformation. A maximum fatigue life was obtained at six percent asphalt content, such that a one percent change from the design optimum content decreased the fatigue life of the mix. A summary of pay adjustment factors for this project is given in Tables 11 - 18 in Appendix A.

In the Warren-Scappoose Project, Walter et al. [31] indicated that performance of a mix is primarily affected by the mix level of compaction. Fatigue data corroborated the design optimum asphalt content (5.5%) and showed a strong interaction between the asphalt content and the amount of fines. Mix susceptibility to permanent deformation was always decreased when increasing the amount of fines for the limits tested. A similar trend was observed in the fatigue data when the asphalt content was increased. A summary of the most critical pay adjustment factors between the fatigue and the permanent deformation criteria is given in Tables 19 - 26 in Appendix A.

3.5 STATISTICAL COMPUTER ANALYSIS OF DATA

The samples of asphalt concrete mix from North Oakland-Sutherlin, Castle Rock-Cedar Creek, and Warren-Scappoose projects were prepared and tested at Oregon State University's laboratory. The main types of pavement failure considered during the test program include fatigue cracking and rutting. All samples were tested in the diametral mode for fatigue life and permanent deformation at 50, 100 and 125 microstrains level. The number of repetitions to failure for both failure criteria was recorded as shown in Table 1 - 26 in Appendix A and used in statistical computer analyses to develop predictive models of pavement performance.

The experimental design of the analyses is depicted in Figure 3-5. The analysis for each project was performed separately and for all projects together at each microstrain level. Fatigue, the primary

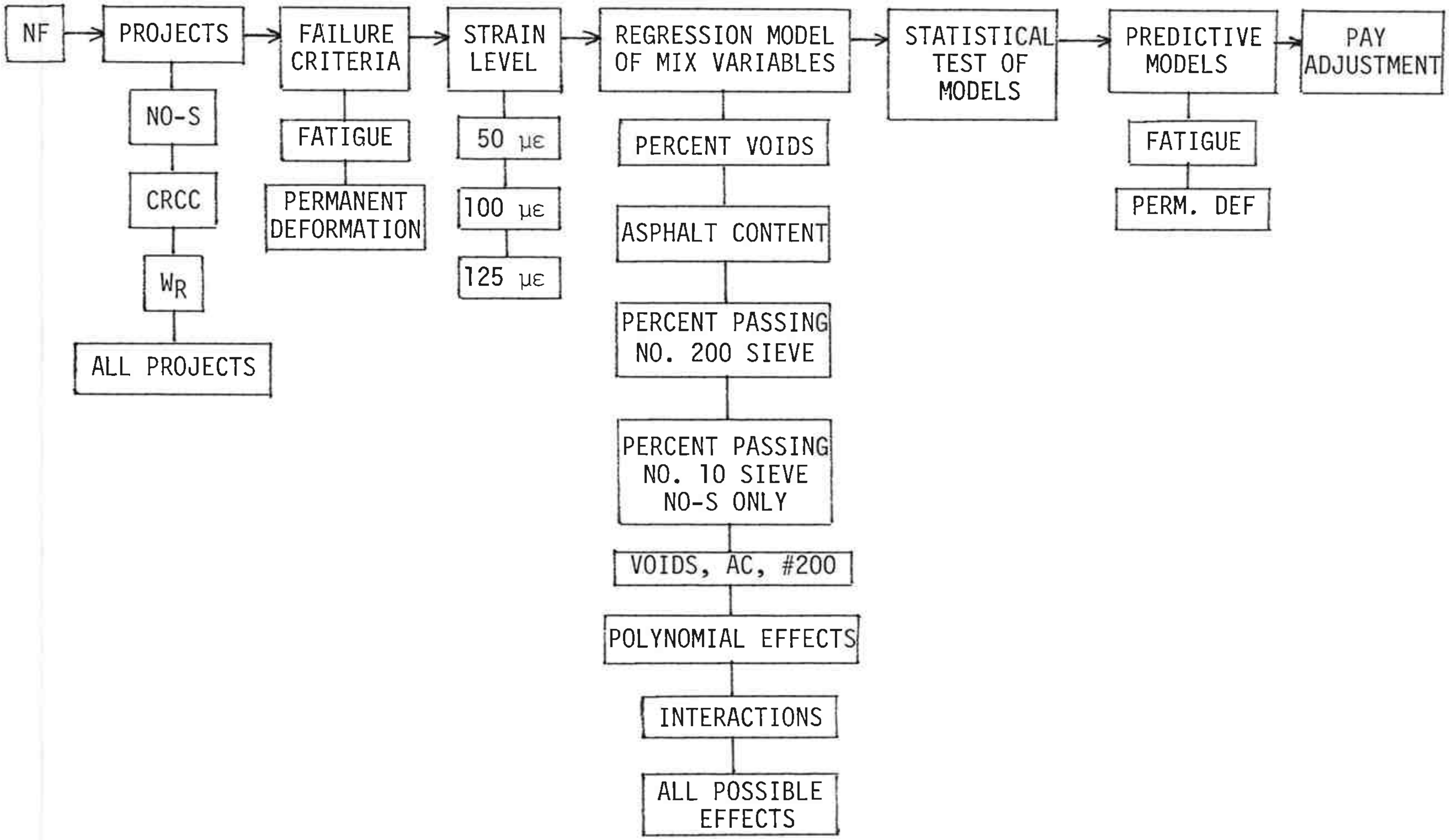


Figure 3-5 Experimental Design of Statistical Computer Analysis of Data to Develop Rational Pay Adjustment Factors.

failure criterion concerned in this analysis, is compared to rutting at only one strain level (100 microstrain). Mix properties such as gradation, percent void, asphalt content and aggregate type are considered in the development of a pavement performance model by means of regression analysis techniques. Search procedures used to find the best set of mix variables are Forward Selection, Backward Elimination and t-Directed Search of SIPS at Oregon State University. The best possible sets of all mix properties at each level of microstrain are listed in Tables 1 - 10 in Appendix C.

3.5.1 Regression Analysis of Data

The statistical analysis and procedures used to develop all best sets of mix variables can be separated into the following steps:

1. Define all mix properties as independent variables and number of repetitions to failure as the dependent variable.

   where:

   Y = number of repetitions to failure (NF)

   X1 = percent passing No. 200 sieve

   X2 = percent passing No. 10 sieve (this variable is analyzed separately only for N. Oakland-Sutherlin Project)

   X3 = asphalt content

   X4 = percent voids

   X5 = 1 for aggregate type used on N. Oakland-Sutherlin Project (crushed stone)

   = 0 otherwise

$X6 = 1$ for aggregate type used on Castle Rock-Cedar Creek Project (crushed stone)

$= 0$ for aggregate type used on Warren-Scappoose Project (crushed gravel)

2. Determine the influence of each mix variable on the number of repetitions to failure (NF) by
   (a) scattering to obtain trend of relationship
   (b) coefficient of determination (r^2)
3. Search for "best" set of mix variables by using:
   (a) Forward Selection
   (b) Backward Elimination
   (c) t-Directed Search
4. Testing hypotheses for existence of all terms in the model at level of significance of (α) 0.05 and 0.10
   (a) $H_o : \beta_k = 0$

   $H_a : \beta_k \neq 0$
   (b) test statistic

   $$t^* = \frac{\beta_k}{S\ (\beta_k)}$$
   (c) decision rule

   If $|t^*| \leq t\ (1 - \alpha/2;\ n-p)$ conclude H_o

   If $|t^*| > t\ (1 - \alpha/2;\ n-p)$ conclude H_a
5. Aptness test of all models to determine the best application of the model. Tests include:
   (a) linearity of model
   (b) constancy of error variance
   (c) independence of error terms

(d) presence of outliers

(e) normality of error terms

(f) omission of important independent variables

6. Determine the best set of mix variables as a predictive model for each microstrain level of each project and for all projects together, and

7. Develop the pay adjustment factor for each project and rational pay adjustment factor. This is described in detail in Chapter 4. Appendix B summarizes the regression techniques used.

3.5.2 Influence of Mix Properties on Fatigue Life

The influence of each mix property on fatigue life is determined separately by scatter of data versus number of repetitions to obtain the trend of relationship. Then the regression equation of fatigue life as a function of each property was set at 50, 100 and 125 microstrain level. The discussion of each mix characteristic on fatigue life is given in the following sections.

3.5.2.1 Effect of Density

The mix density (or percent air voids) is the most dominant factor for all mix properties. Fatigue life is primarily affected by the mix level of compaction; increasing the mix density or decreasing the percent voids increases the fatigue life of the pavement. In mix design, when considering other variables, the void content should be minimized; the binder content, the aggregate gradation, density, and the use of fillers are selected to obtain the smallest void space possible. Regression analysis showing effect of percent voids on fatigue life of North Oakland-Sutherlin, Castle Rock-Cedar Creek and Warren-Scappoose projects are shown in Figures 3-6 through 3-8. The results of the analysis indicate that the amount of air voids control the number of repetitions to failure of asphalt concrete pavements. Small increases in air voids content causes substantial decreases in fatigue life. The decrease in fatigue life is governed by the traffic load level and aggregate type when all other factors are fixed. At the low microstrain level or heavy traffic load level, the fatigue life drops sharply with increasing voids content of mix.

Figure 3-9 shows the effect of void content on fatigue life when negligible variation in aggregate type is used. Even though the relationship of fatigue life and void content seem to be satisfactory in this case, the large decrease in the coefficient of determination (R^2) indicates the high variation in the data sets. Therefore, the aggregate type appears to be one of the controlling factors of pavement performance life.

3.5.2.2 Effect of Asphalt Content

Binder content is one of the critical single factors that regulate all mix properties. Not only is the binder invariably the most expensive constituent of the mix, but it also directly controls the stiffness and/or flexibility of the asphaltic concrete. As the content of binder fluctuates within the mix, the amount of void space filled by the binder in the aggregate gradation is also altered. This modification in void space filled influences aggregate interparticle friction which, in turn, affects the stability, durability, strength and fatigue.

The effects of asphalt content on fatigue life are shown in Figures 3-10 through 3-13. As the asphalt content increases, the fatigue life increases up to the optimum level. There exists, though, an optimum

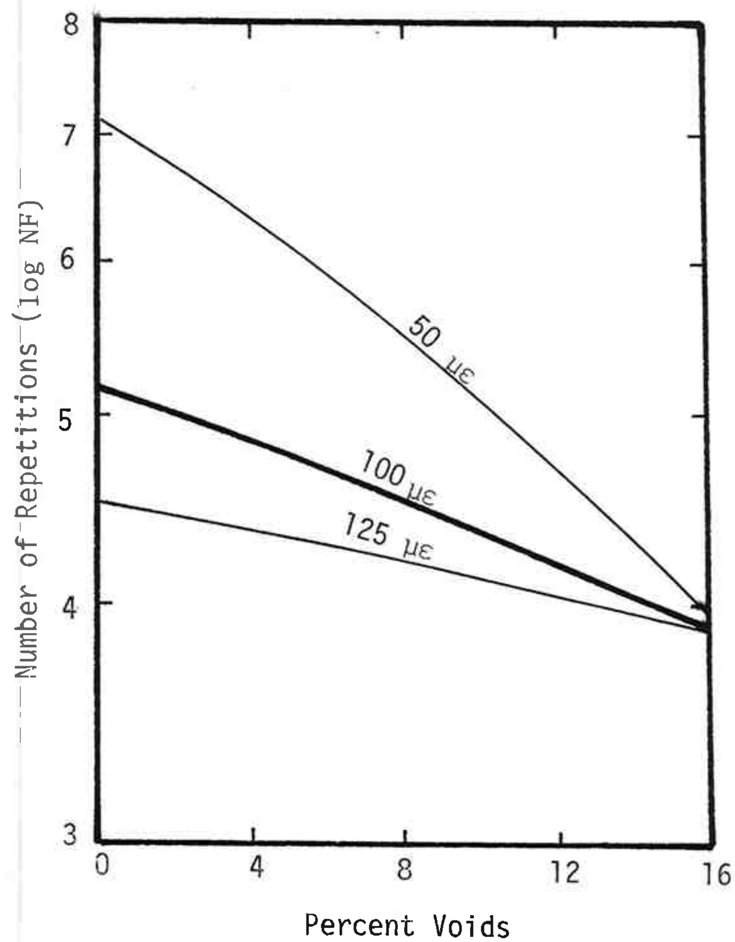


Figure 3-6 The Effect of Percent Voids on Fatigue Life at Different Microstrain Levels for N. Oakland-Sutherlin Project

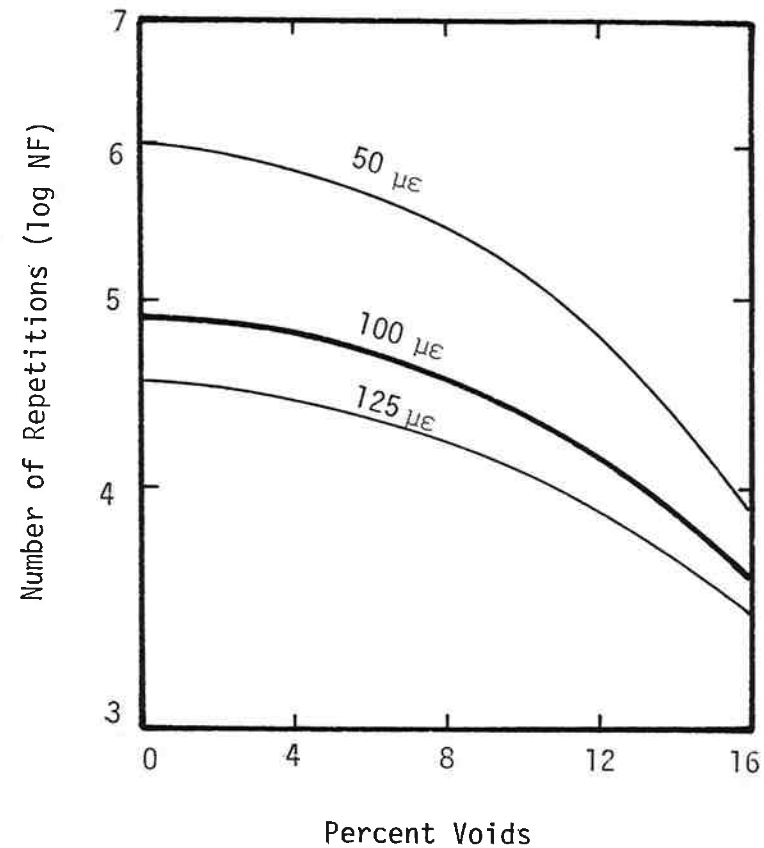


Figure 3-7 The Effect of Percent Voids on Fatigue Life at Different Microstrain Levels for Castle Rock-Cedar Creek Project

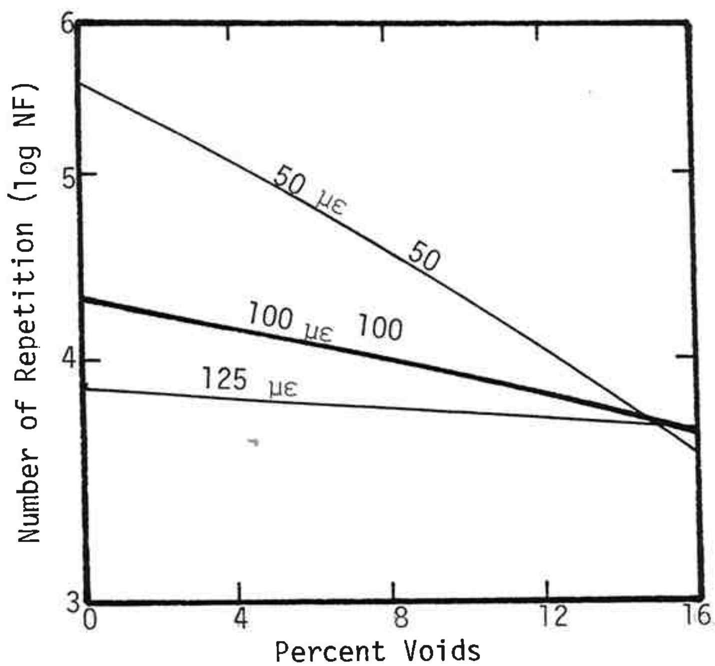


Figure 3-8 The Effect of Percent Voids on Fatigue Life at Three Different Microstrain Levels for Warren Scappoose Project

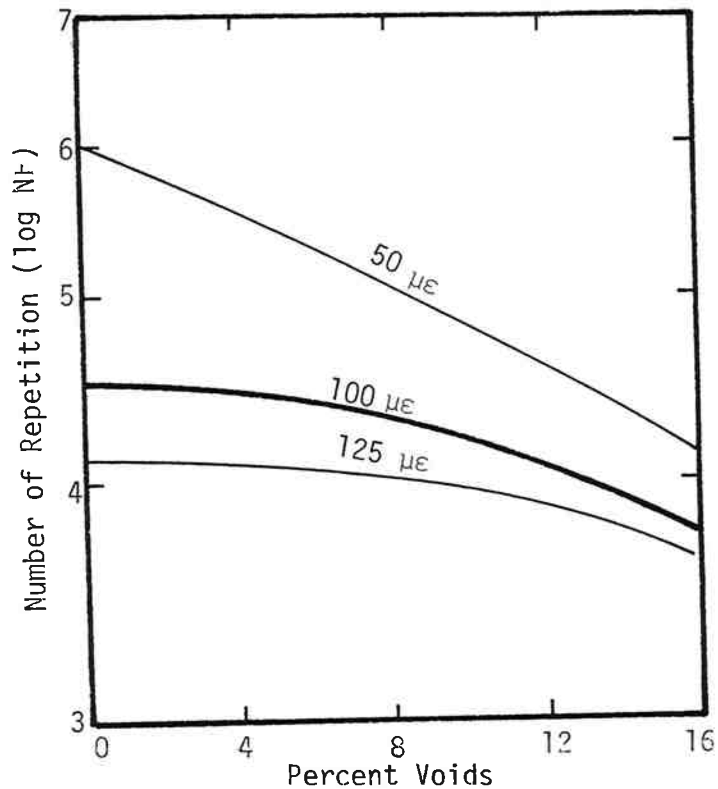


Figure 3-9 The Effect of Percent Voids on Fatigue Life at Three Different Microstrain Levels for All Projects

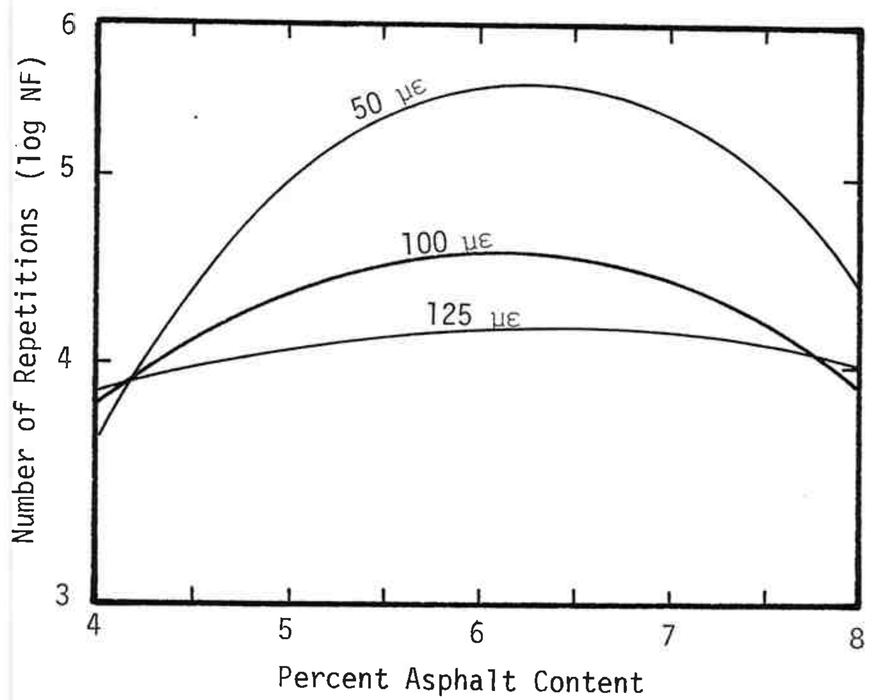


Figure 3-10 The Effect of Asphalt Content on Fatigue Life at Three Different Microstrain Levels for North Oakland-Sutherlin Project

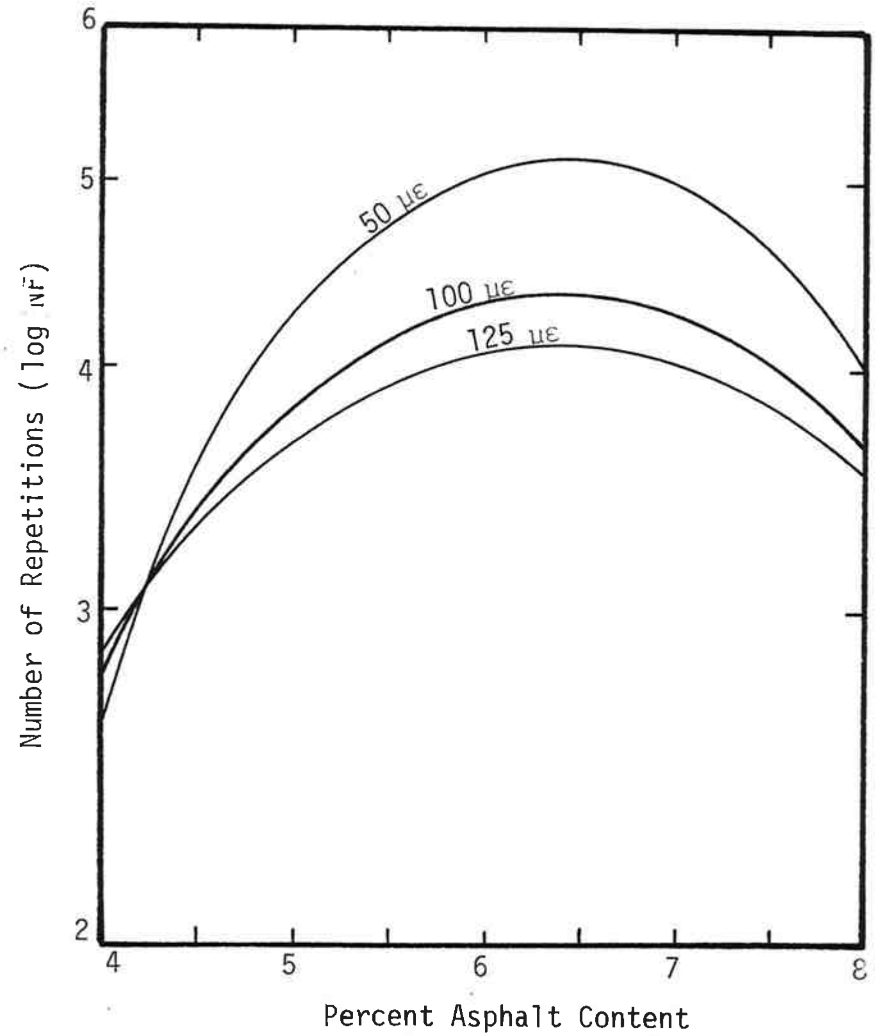


Figure 3-11 The Effect of Asphalt Content Fatigue Life at Three Different Microstrain Levels for Castle Rock-Cedar Creek Project

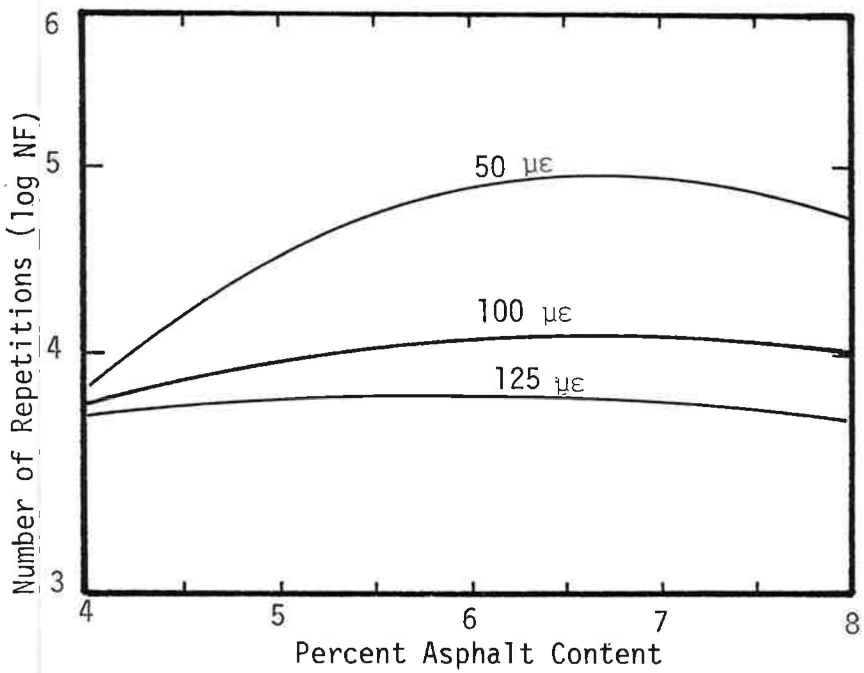


Figure 3-12 The Effect of Asphalt Content on Fatigue Life at Three Different Microstrain Levels for Warren-Scappoose Project

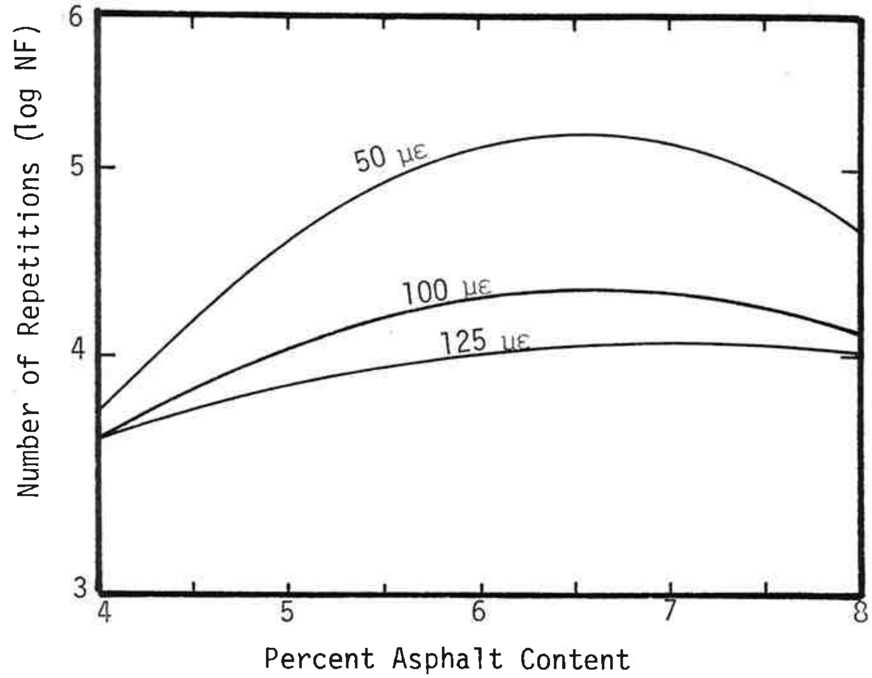


Figure 3-13 The Effect of Asphalt Content on Fatigue Life at Three Different Microstrain Levels for Combined Projects

asphalt content that corresponds to the point where maximum fatigue life is obtainable. This is a product of the binder filling the aggregate void space. As the voids become filled, the binder cements aggregate particles together causing an increase in the strength of bonding. When the voids are completely filled the bonding and interparticle reactions are at a maximum and thus a maximum stiffness is realized. As the voids become overfilled, the aggregate friction decreases and the binder takes more of the load. In this situation the stiffness decreases as more and more binder is added. Table 3-2 shows the optimum asphalt content related to fatigue life at each traffic load level. These asphalt percentages are an optimum for the particular aggregate type, the binder type, the gradation of the aggregate used and level of traffic load.

Table 3-2. The Optimum Asphalt Content Related to Fatigue Life at Each Microstrain Level of NO-S, CRCC, WS and Combined Project.

	Optimum Asphalt Content			Maximum Fatigue Life		
Microstrain Level / Projects	50 με	100 με	125 με	50 με	100 με	125 με
NO-S (6% design)	6.36	6.20	6.39	4.23×10^5	3.95×10^4	1.61×10^4
CRCC (6% design)	6.41	6.41	6.41	9.23×10^4	2.38×10^4	1.32×10^4
WS (5.5% design)	6.71	6.70	6.01	9.53×10^4	1.30×10^4	6.63×10^3
ALL	6.58	6.60	7.21	1.79×10^5	2.19×10^4	1.21×10^4

The fatigue life varies not only with asphalt content but also with traffic load and aggregate type. At the heavy traffic load level, the fatigue performance life changes with increasing asphalt content, and the relative change in fatigue is controlled by aggregate quality. For the relative change in fatigue is controlled by aggregate quality. For example, in the Castle Rock-Cedar Creek project, the fatigue life is sensitive to change as increasing asphalt content (refer to Fig. 3-11); however, the rate of change in fatigue performance life is smaller in the Warren-Scappoose project. In the North Oakland-Sutherlin project, the variation in asphalt content altered the fatigue life less than in the other two projects. Assuming there is no difference in aggregate type used in all three projects, the effect of asphalt content on fatigue life is shown in Figure 3-13.

3.5.2.3 Effect of Aggregate Gradation

Since asphalt and aggregate are the two main ingredients in the asphalt concrete mix, the gradation of an aggregate indicates the amount of void space available to be filled with the asphalt binder. The degree to which the voids are filled with binder greatly influences the stiffness and fatigue life of a mix. In the same respect, the amount of void space provided by aggregate also controls the stiffness and fatigue life. Figures 3-14 through 3-17 verify that the fatigue is related to the percent passing No. 200 sieve at each microstrain level, and the amount present increasing fatigue life until an optimum is reached. For the North Oakland-Sutherlin and Castle Rock-Cedar Creek projects at 100 microstrain level, an increasing amount of fines increases the life of the pavement with a maximum fatigue life at the optimum percent fines of ten and seven percent, respectively. The Warren-Scappoose project reflected a higher optimum amount of fines to reach maximum fatigue life. In general, for all three projects together, the optimum amount of passing No. 200 material required to obtain maximum fatigue life at 100 microstrain level is nine percent, as shown in Figure 3-17. The

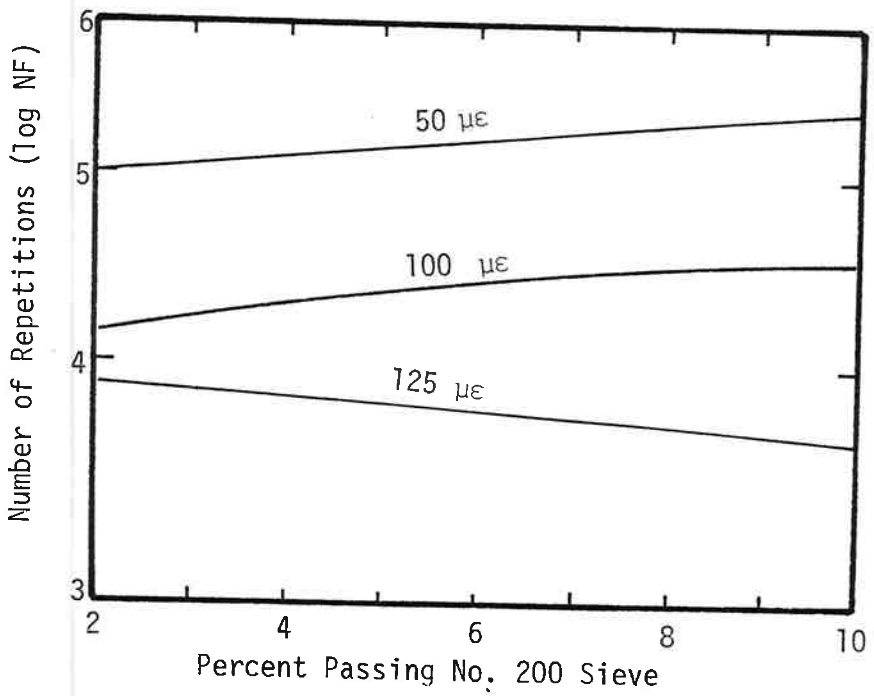


Figure 3-14 The Effect of Percent Passing No. 200 Sieve on Fatigue Life at Three Different Microstrain Levels for N. Oakland-Sutherlin Project

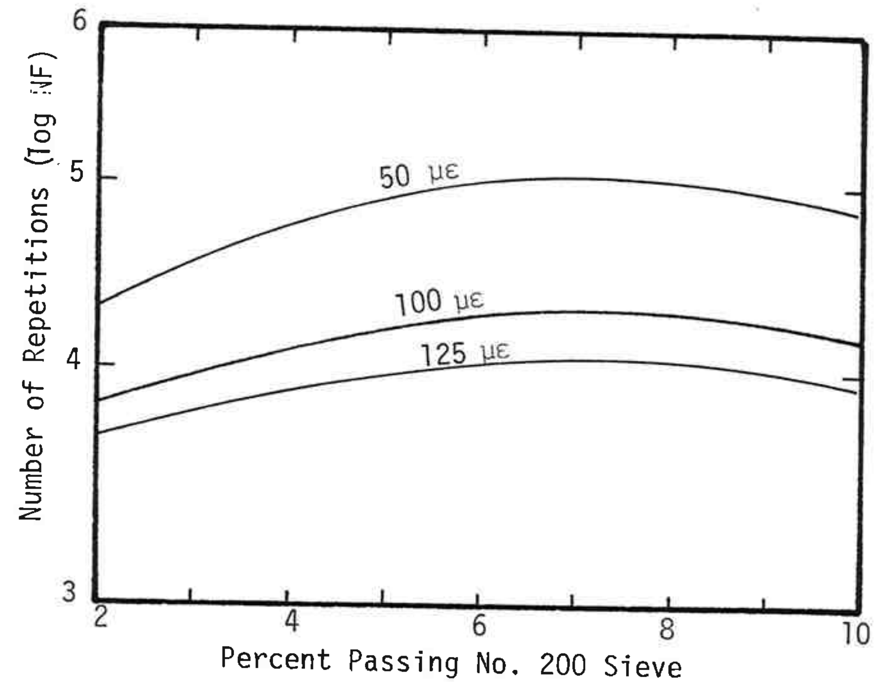


Figure 3-15 The Effect of Percent Passing No. 200 Sieve on Fatigue Life at Three Different Microstrain Levels for Castle Rock-Cedar Creek Project

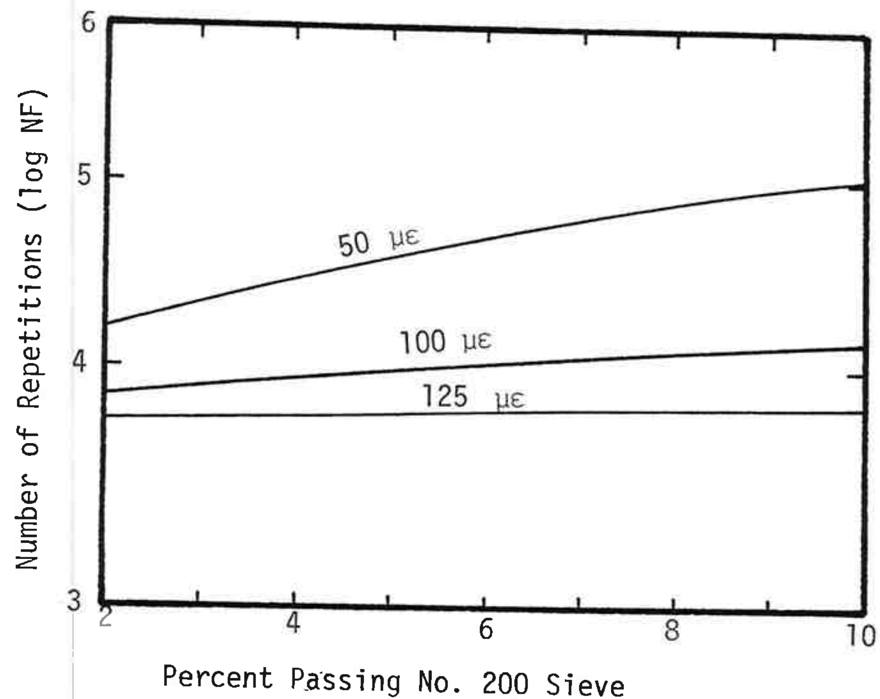


Figure 3-16 The Effect of Percent Passing No. 200 Sieve on Fatigue Life at Three Different Microstrain Levels for Warren-Scappoose Project

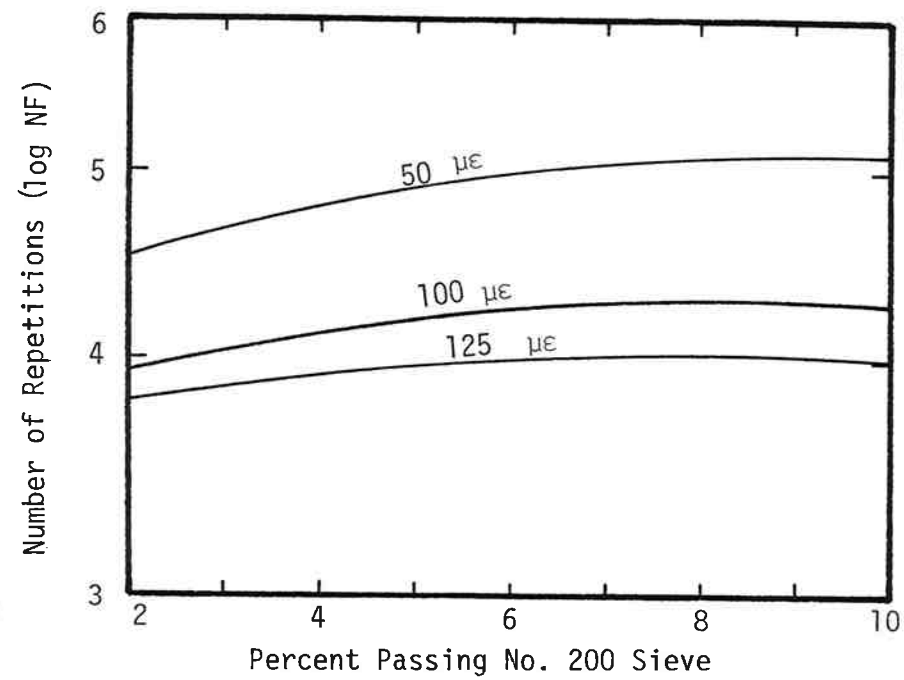


Figure 3-17 The Effect of Percent Passing No. 200 Sieve on Fatigue Life at Three Different Microstrain Levels for Combined Projects

North Oakland-Sutherlin project and the results are shown in Figure 3-18. For the project conditions increase the amount of material retained on the No. 10 sieve mix tends to reduce the fatigue life. In view of the test results, the gradation does have some effect on fatigue life, but is not as significant and critical as other factors.

3.5.2.4 Effect of Aggregate Type

Since the aggregate is the primary load-carrying substance, it is expected to have some control on fatigue life. Shape, surface texture, and durability properties are of most interest when investigating aggregate types. Probably the most important is durability. In Oregon, pavement performance has been greatly reduced due to aggregate breakdown. The regression analyses of aggregate type effect on fatigue used for all three projects together are shown below:

Model I : Not Considering Aggregate Type Effect

$$\log NF = 4.6875 - 0.05103\ (VOIDS)$$

$$R^2 = 0.3300$$

Model II: Considering Aggregate Type Effect

$$\log NF = 4.5732 - 0.0744\ (VOIDS) + 0.5325\ (AG.T1) + 0.4585\ (AG.T2)$$

$$R^2 = 0.8266$$

where:

NF = number of repetitions to failure

AGT1 = 1 for marginal quality aggregate

= 0 for good quality aggregate

AGT2 = 1 for crushed stone

= 0 for crushed gravel.

The following applied for each project:

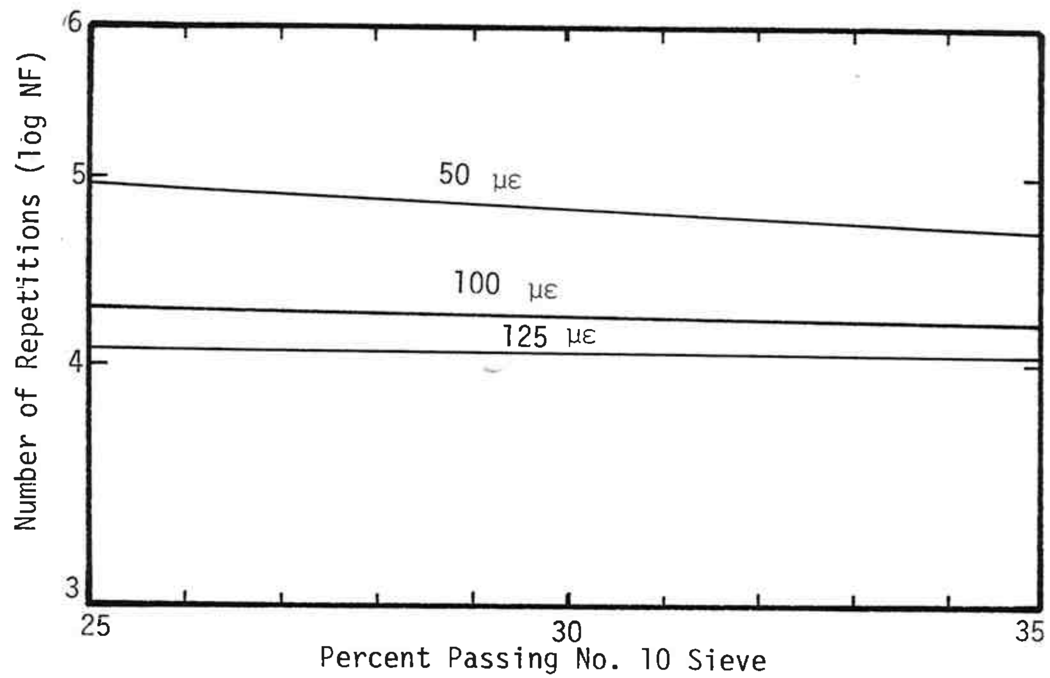


Figure 3-18 The Effect of Percent Passing No. 10 Sieve on Fatigue Life at Three Different Microstrain Levels for N. Oakland Sutherlin Project

1) North Oakland-Sutherlin. Crushed stone with good and marginal quality aggregate.

2) Castle Rock-Cedar Creek. Crushed stone with good quality aggregate.

3) Warren-Scappoose. Crushed gravel with good quality aggregates.

Model II which considers aggregate type effect not only shows high statistical significance but also indicates the influence of aggregate type on fatigue life. At certain values of voids and asphalt contents, the fatigue life varies with the quality of aggregates as shown in Figures 3-19 and 3-20. If the asphalt content is fixed at optimum and voids content at any value, the aggregate type and quality (used in North Oakland-Sutherlin) gives the highest fatigue life to the pavement, as shown in Figure 3-20.

3.6 SUMMARY

The most frequently occurring mode of distress in asphalt highway pavement in Oregon is fatigue cracking associated with traffic loads. This distress mechanism causes serious damage and loss of serviceability in pavement life. Numerous research studies have been conducted in the past to investigate the factors that influence this phenomenon and to develop criteria for prediction of pavement performance. The typical prediction of fatigue life is expressed by the following equation:

$$N_f = K \left(\frac{1}{\varepsilon_t}\right)^C$$

The factors K and C depend strongly on the mix properties. Asphalt content, aggregate gradation and type, and air void content have shown

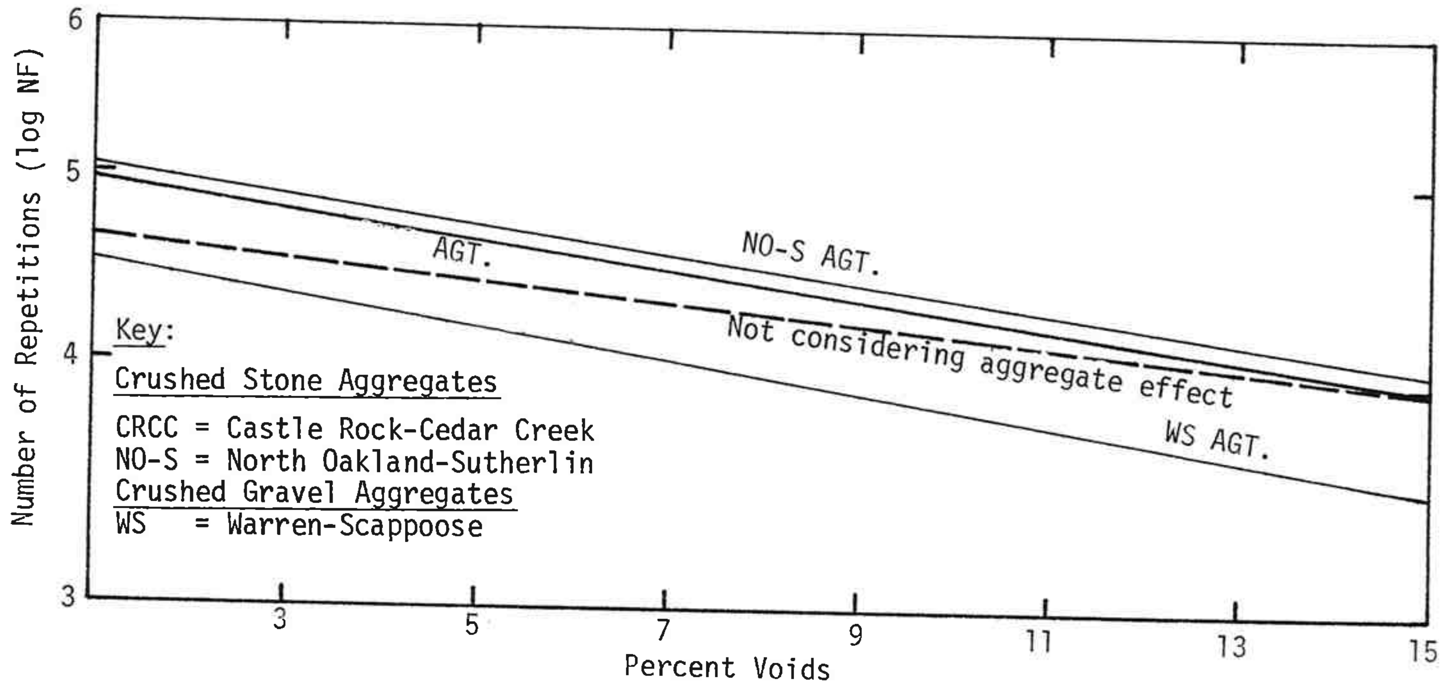


Figure 3-19 Effects of Aggregate Type on Fatigue Performance of Asphalt Concrete Mixtures

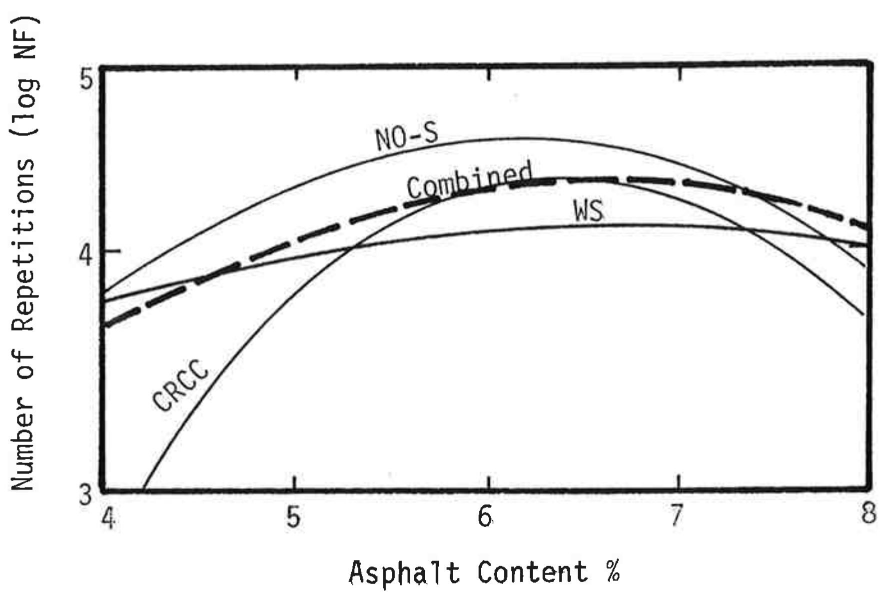


Figure 3-20 Effect of Aggregate Type Incorporated with Asphalt Content on Fatigue Performance of Asphalt Concrete Mixtures

the most influence on fatigue performance of asphalt pavement. The mix containing high void contents exhibits remarkably short fatigue lives. With an increase in asphalt content of mix up to an optimum, the fatigue performance is improved. Aggregate gradation and texture also have some influence on mixture fatigue. The rougher textured aggregate allows more asphalt to be incorporated into the mixture and causes increasing fatigue resistance results.

Oregon DOT has studied the impact of variation in mix properties on asphalt pavement lives of North Oakland-Sutherlin, Castle Rock-Cedar Creek and Warren-Scappoose projects. The results of the study indicate that the mix level of compaction is the dominant factor in controlling pavement performance life. Pay adjustment factors for each mix property were developed based upon the fatigue and permanent deformation lives of pavements. With the question as to what extent the quality of work should be accepted or rejected and what properties can or cannot be allowed to be lower than a standard in mind, all the pavement data from these three projects were analyzed by means of computer statistical analysis at Oregon State University. The regression analysis techniques are used to evaluate the effect of variations in mix properties on pavement life. The predictive models for fatigue and permanent deformation are established to predict the pavement performance. The results of analysis led to the conclusion that mix density or void content is highly statistically significant in controlling both fatigue cracking and rutting failure. Increasing the mix air voids content causes a reduction in pavement life. The other mix properties, such as asphalt content and amount of fines, exhibit minor influence when their values are close to optimum. As the values deviate from an optimum level, the mix behavior reduces in performance life. The amount of voids and optimum asphalt content are a function of the aggregate type used. Therefore, the influencing degree of air voids and asphalt content on mix behavior is controlled by aggregate quality.

4.0 RECOMMENDED PAY ADJUSTMENT FACTORS

4.1 INTRODUCTION

The purpose of this chapter is to isolate the most critical pavement performance conditions for use in developing pay adjustment factors. The estimated fatigue and permanent deformation performance of mix are compared for each mix variable. Pay adjustment factors are developed based upon shorter performance life for each project and for combined projects. The summary of pay adjustment factors is presented and compared with existing local pay factors.

4.2 APPROACHES

The predictive models at three different tensile strain levels were evaluated. The models corresponding to 100 microstrain should be considered as the most reliable representation of general traffic load conditions. The model of 50 microstrain represents a thick pavement carrying heavy traffic loads and the models of 125 microstrain represent light traffic loads or private roads. Therefore, the predictive models of 100 microstrain are used to obtain the pavement life in this study. The effect of variations of percent voids, asphalt content and percent passing No. 200 sieve on pavement performance are evaluated for both fatigue and permanent deformation criteria. The predictive models of fatigue and permanent deformation life are obtained and evaluated. The results are described as follows.

The pavement performance life as a function of air voids for both fatigue and rutting are shown in Figures 4-1 through 4-4. An increasing percent of voids results in reduced pavement life for both permanent

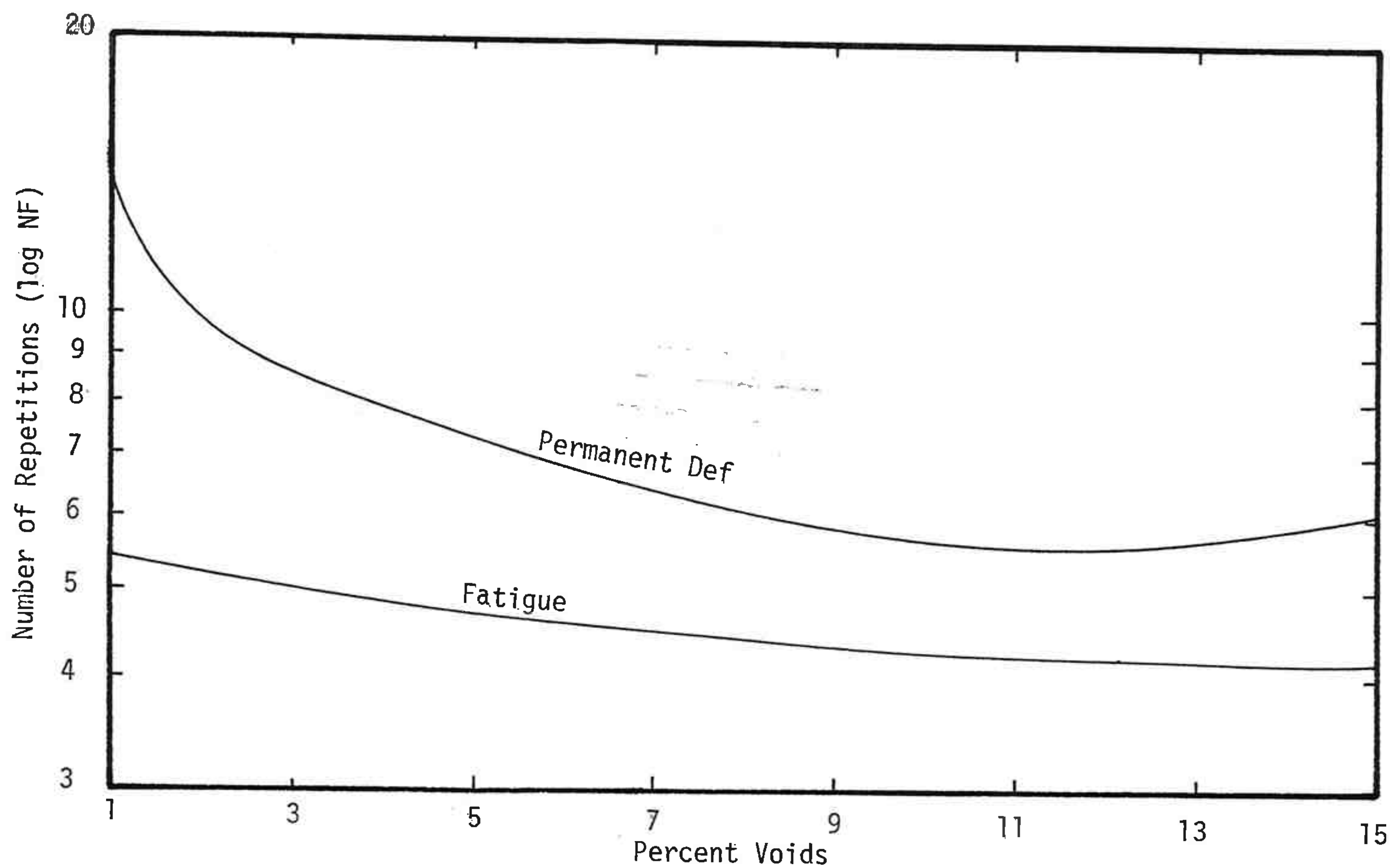


Figure 4-1 The Permanent Deformation and Fatigue Curves as a Function of Percent Voids for N. Oakland-Sutherlin Project at 100 microstrain

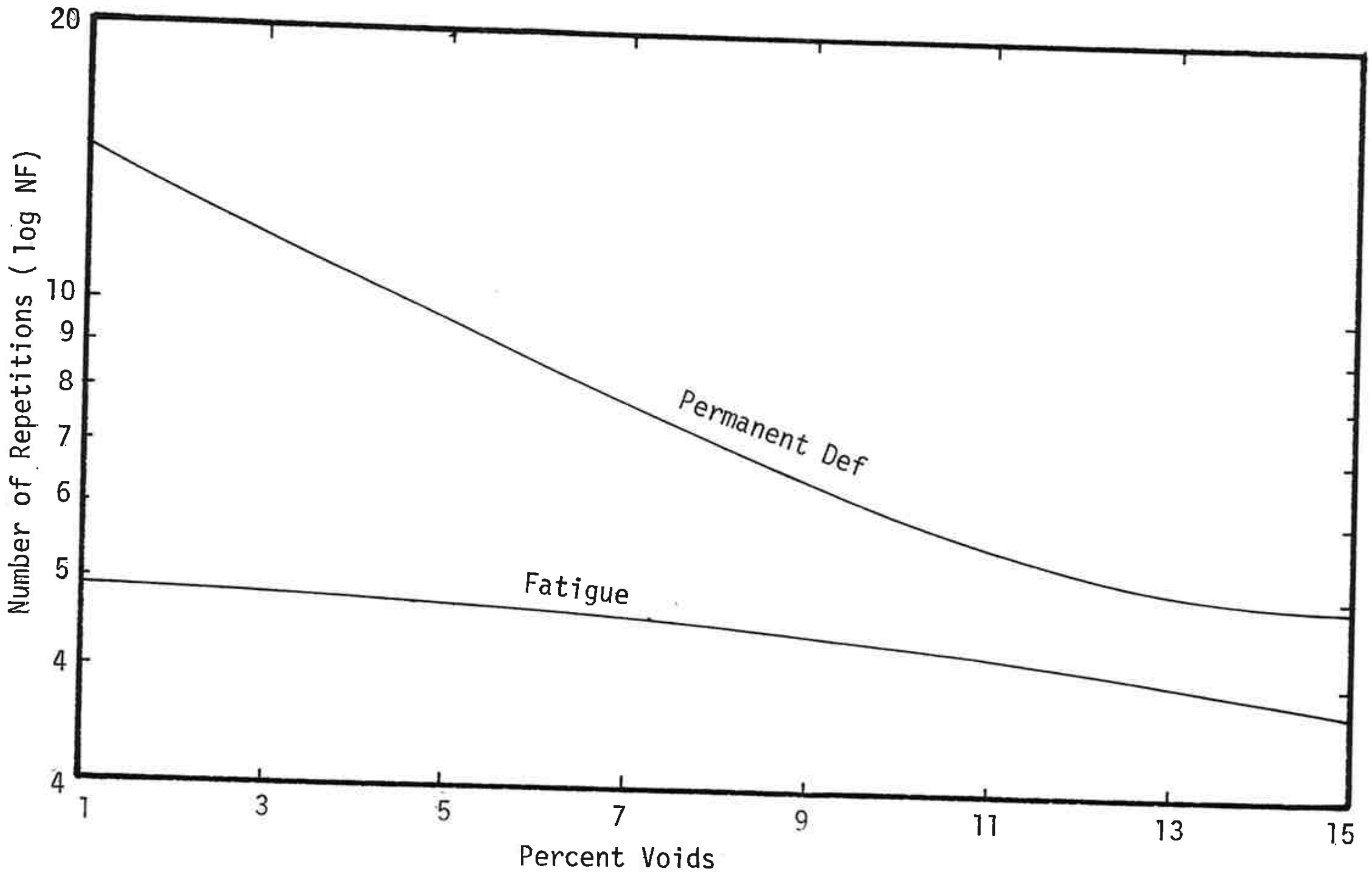


Figure 4-2 The Permanent Deformation and Fatigue Curve as a Function of Percent Voids for Castle Rock-Cedar Creek Project at 100 microstrain

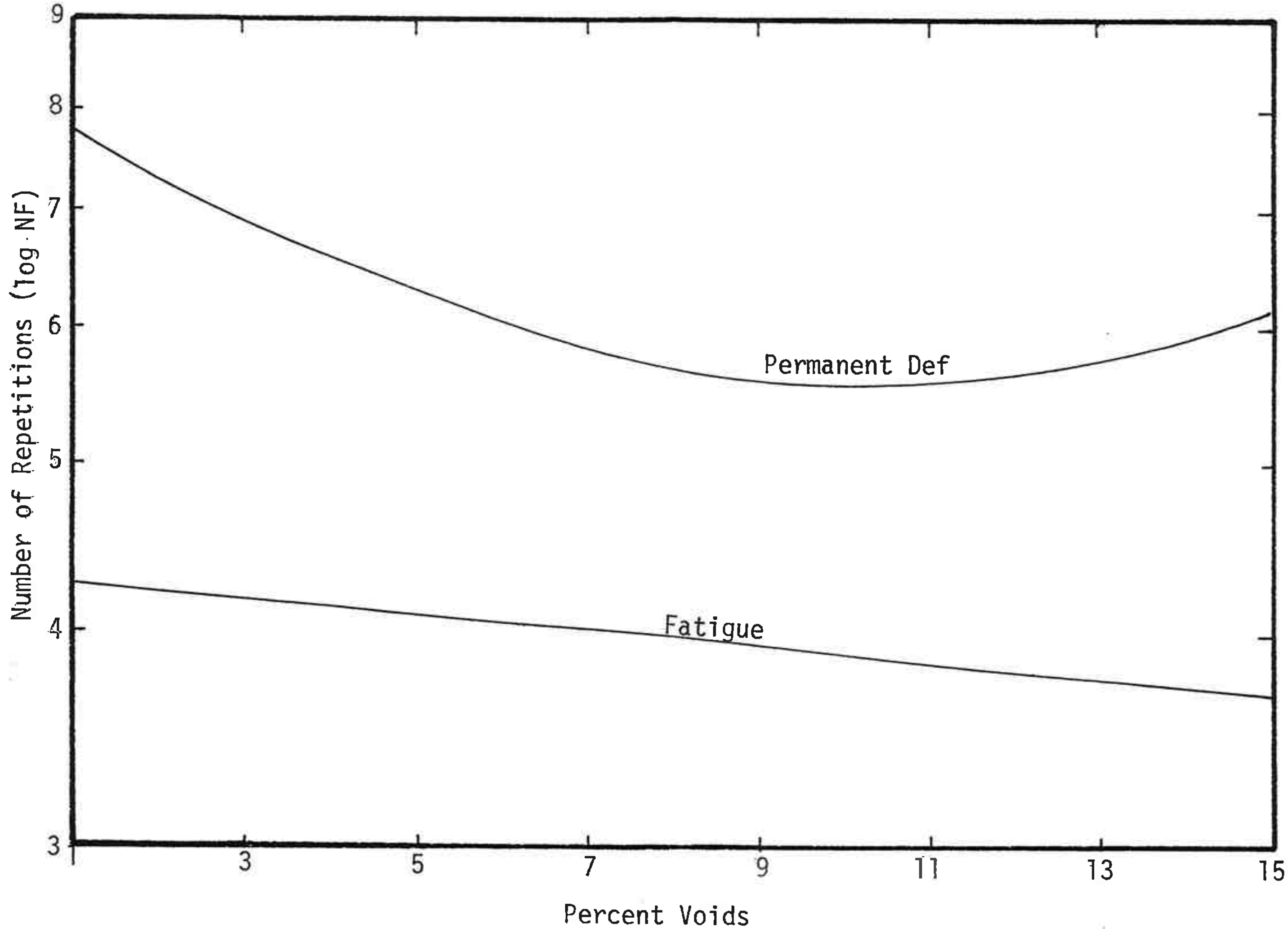


Figure 4-3 The Permanent Deformation and Fatigue Curves as a Function of Percent Voids for Warren-Scappoose Project at 100 microstrain

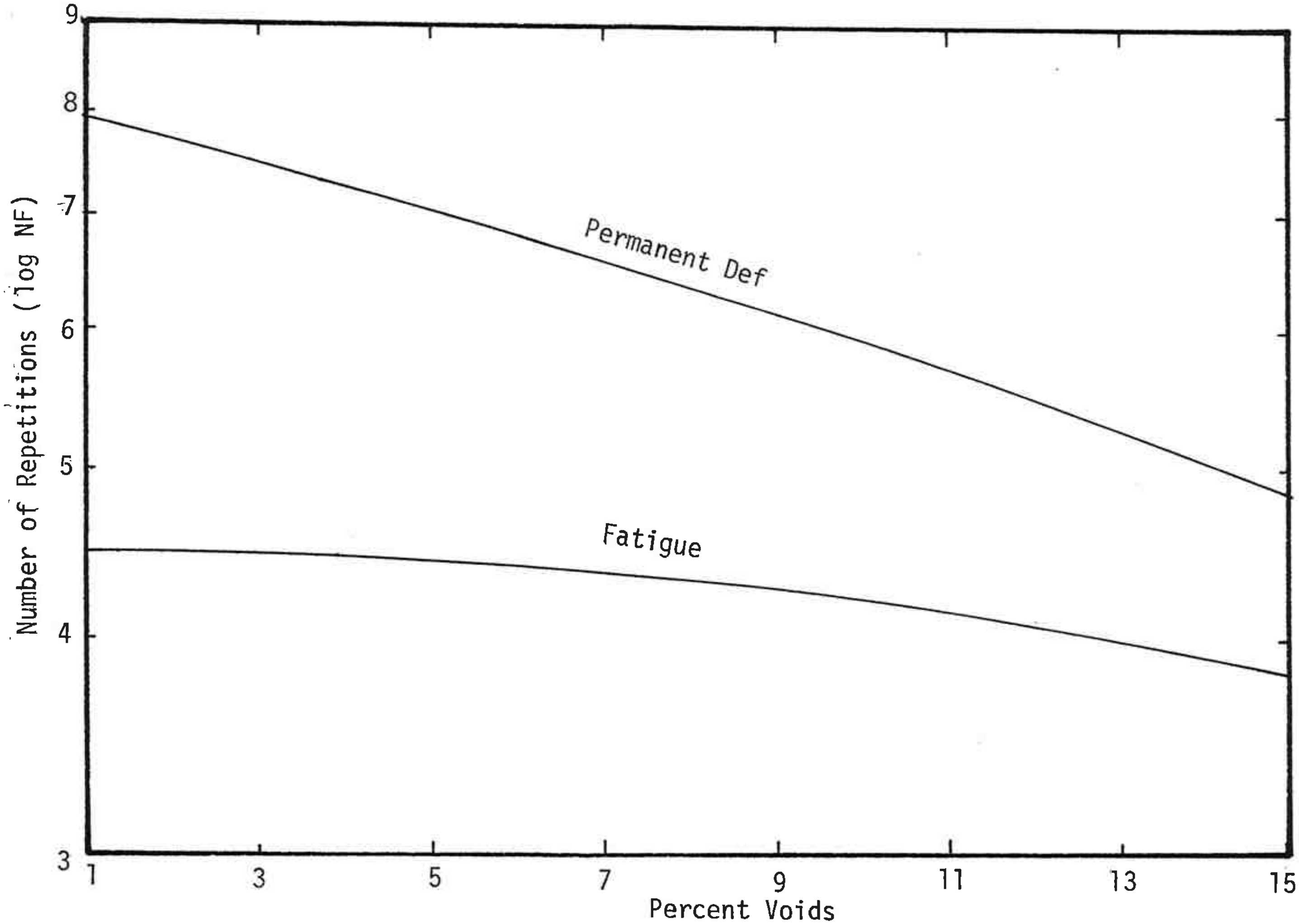


Figure 4-4 The Permanent Deformation and Fatigue Curves as a Function of Percent Voids for All Projects at 100 microstrain

deformation and fatigue cracking. As void content increases its effect on occurrence of rutting in comparison to cracking is greater. It should be noted that the fatigue and deformation results are for the same temperatures and although the number of load repetitions to failure is greater for deformation at this temperature, changes in temperature would alter the relative performance. Also, the laboratory results have not been adjusted to field performance curves, and, therefore, the same relationship is assumed between laboratory and field performance for both fatigue and deformation.

The permanent deformation and fatigue curves as a function of asphalt content are presented in Figures 4-5 through 4-8. The change in asphalt content from the design optimum significantly changes the pavement performance life, due to permanent deformation. This problem might be an effect of aggregate quality used in this project. Analyses of laboratory data for all projects indicates that at asphalt contents higher than optimum the main problem would be rutting. But because of the test temperatures used, fatigue cracking was still the most critical condition.

The impact of amount of fines on pavement performance life is shown in Figures 4-9 through 4-12. The results indicate the effect of fines on mix behavior is similar to asphalt content. When the optimum value is exceeded, the number of repetitions to failure for permanent deformation drops sharply and, again, on the Castle Rock-Cedar Creek project this effect is clearly indicated. The variation in the amount of fines appears to have minimal effect on fatigue life; however, fatigue cracking is still the controlling criterion with regard to the amount of fines in the mix.

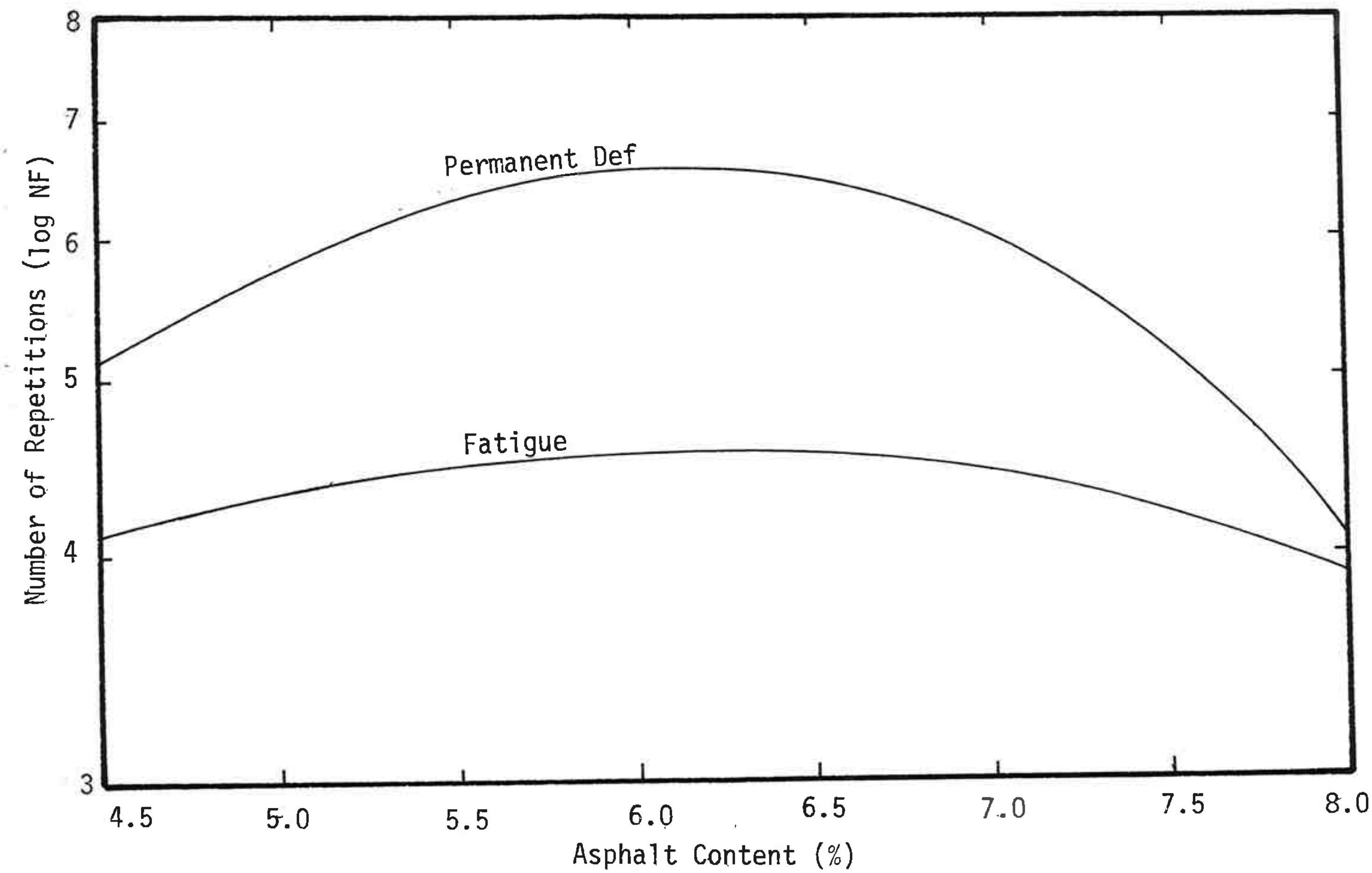


Figure 4-5 The Permanent Deformation and Fatigue Curves as a Function of Asphalt Content for N. Oakland-Sutherlin Project at 100 microstrain

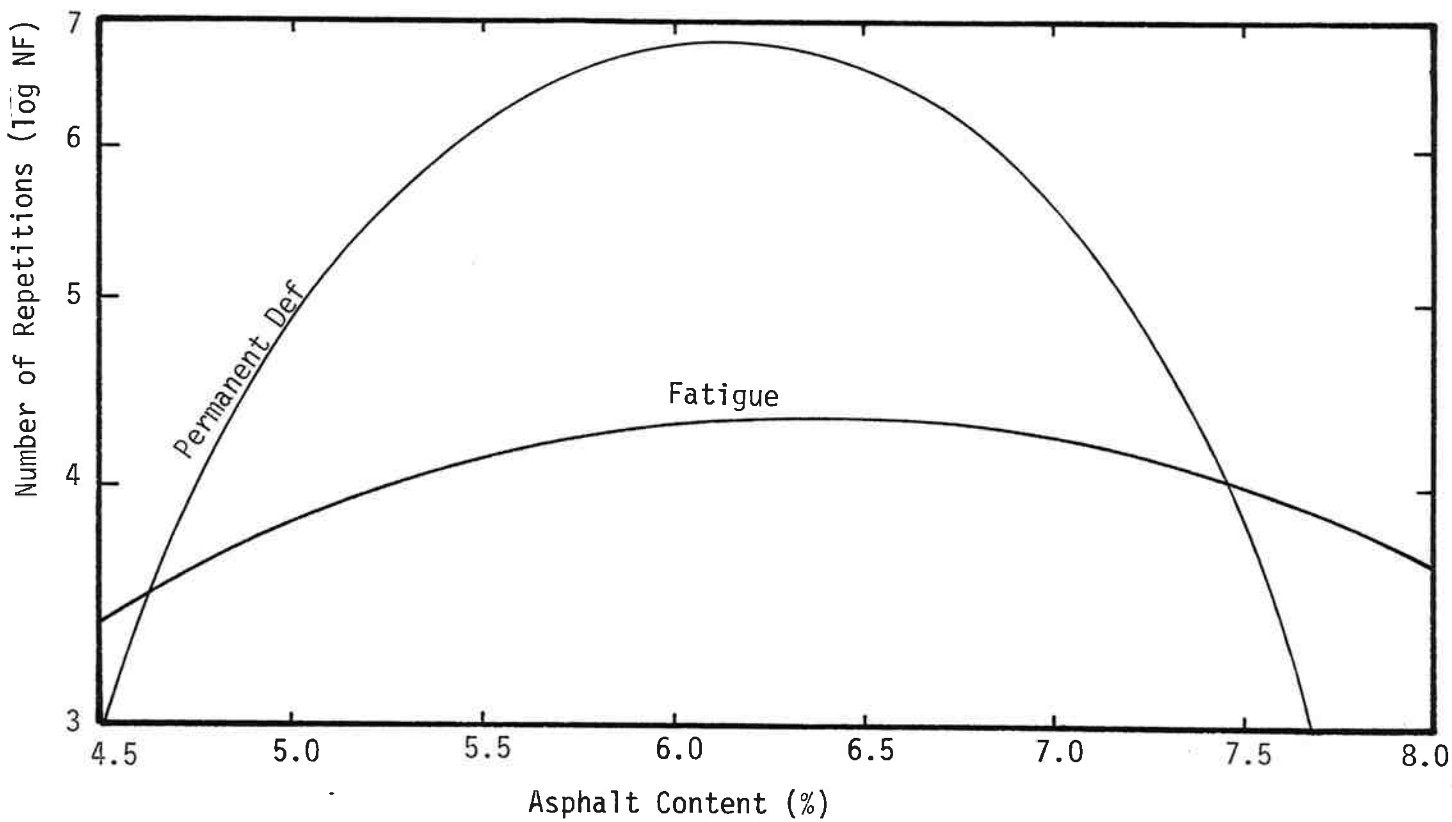


Figure 4-6 The Fatigue and Permanent Deformation Curves as a Function of Asphalt Content for Castle Rock-Cedar Creek Project at 100 microstrain

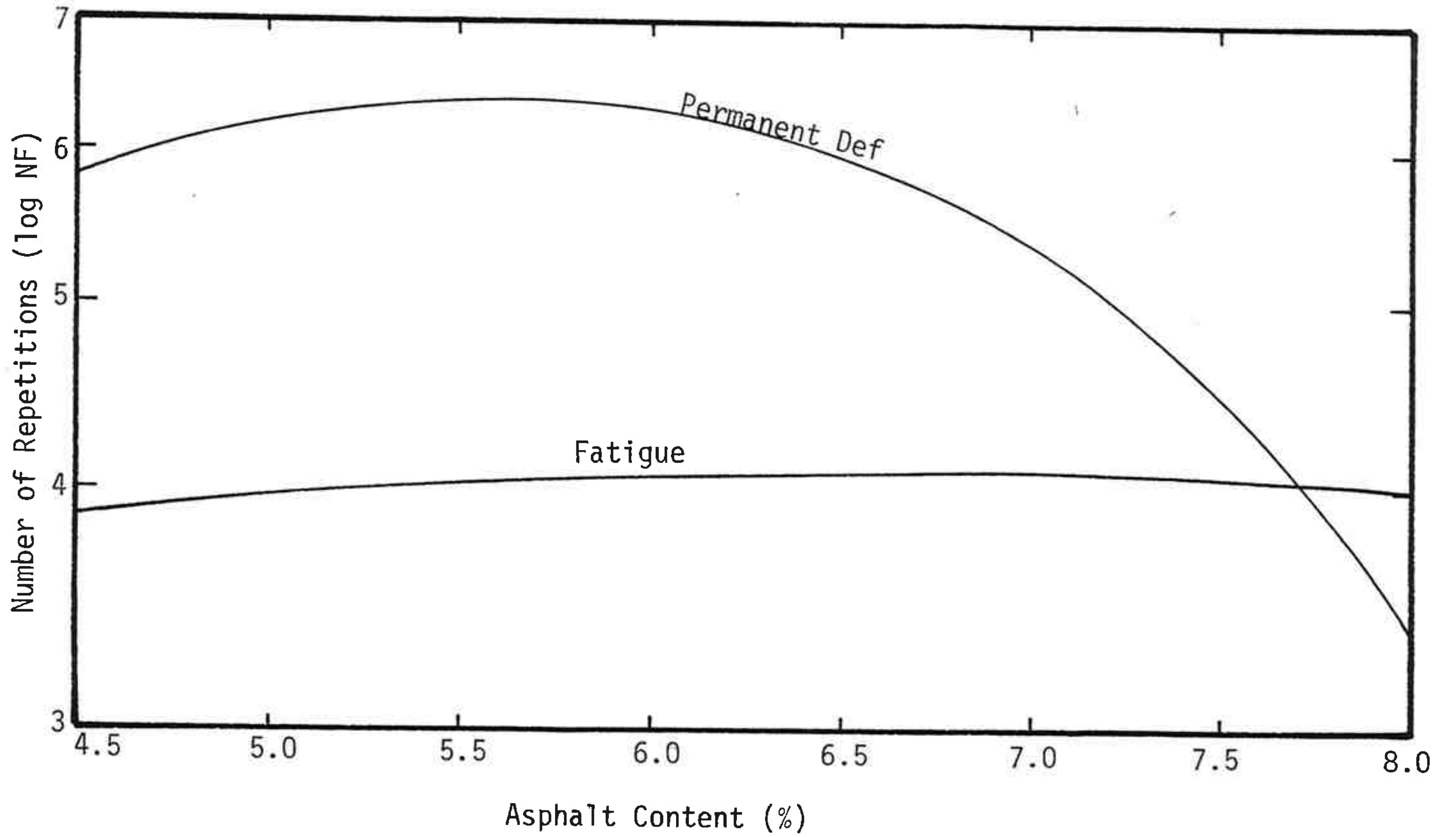


Figure 4-7 The Permanent Deformation and Fatigue Curves as a Function of Asphalt Content for Warren-Scappoose Project at 100 Microstrain

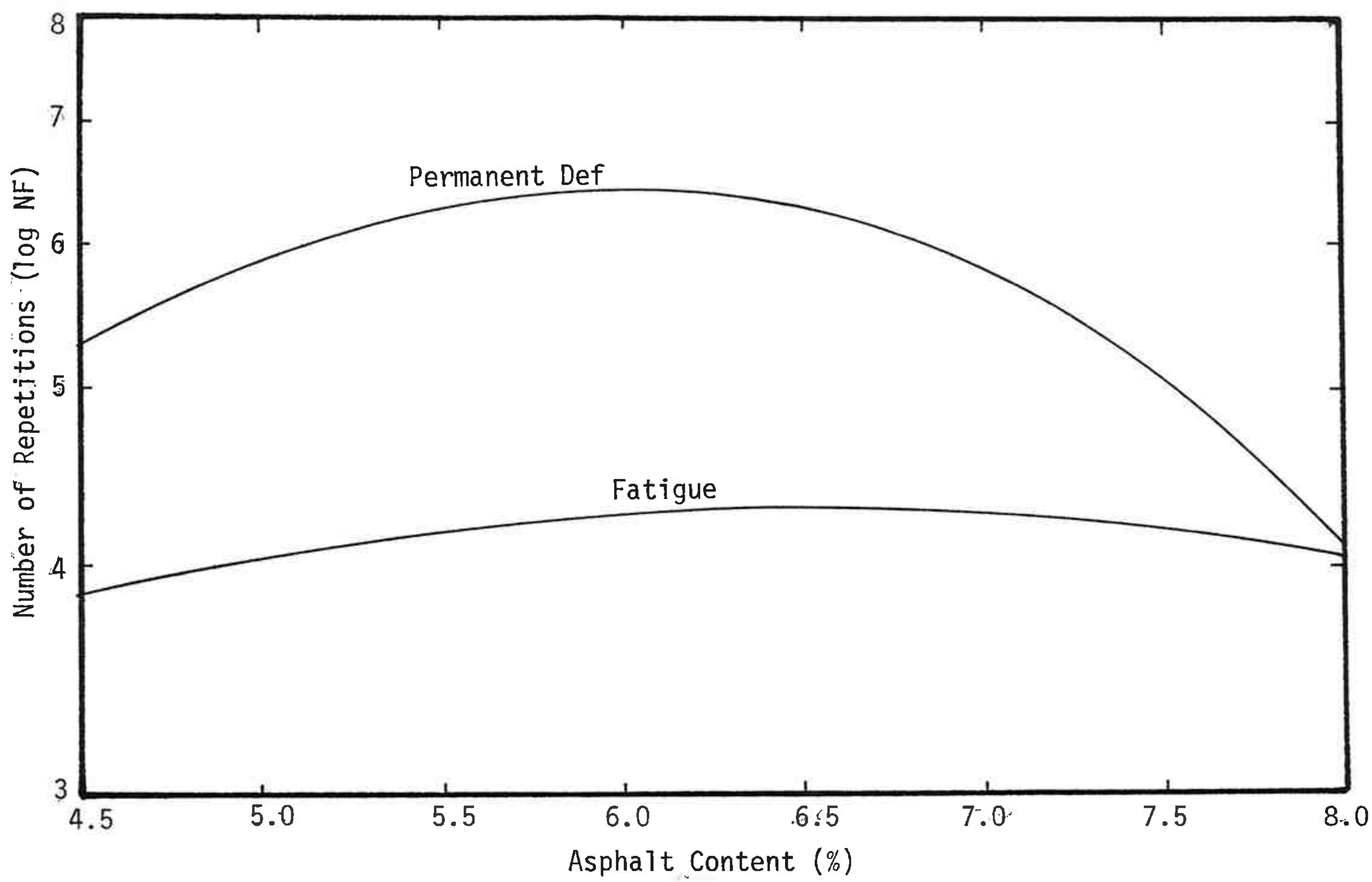


Figure 4-8 The Permanent Deformation and Fatigue Curves as a Function of Asphalt Content for All Projects at 100 microstrain

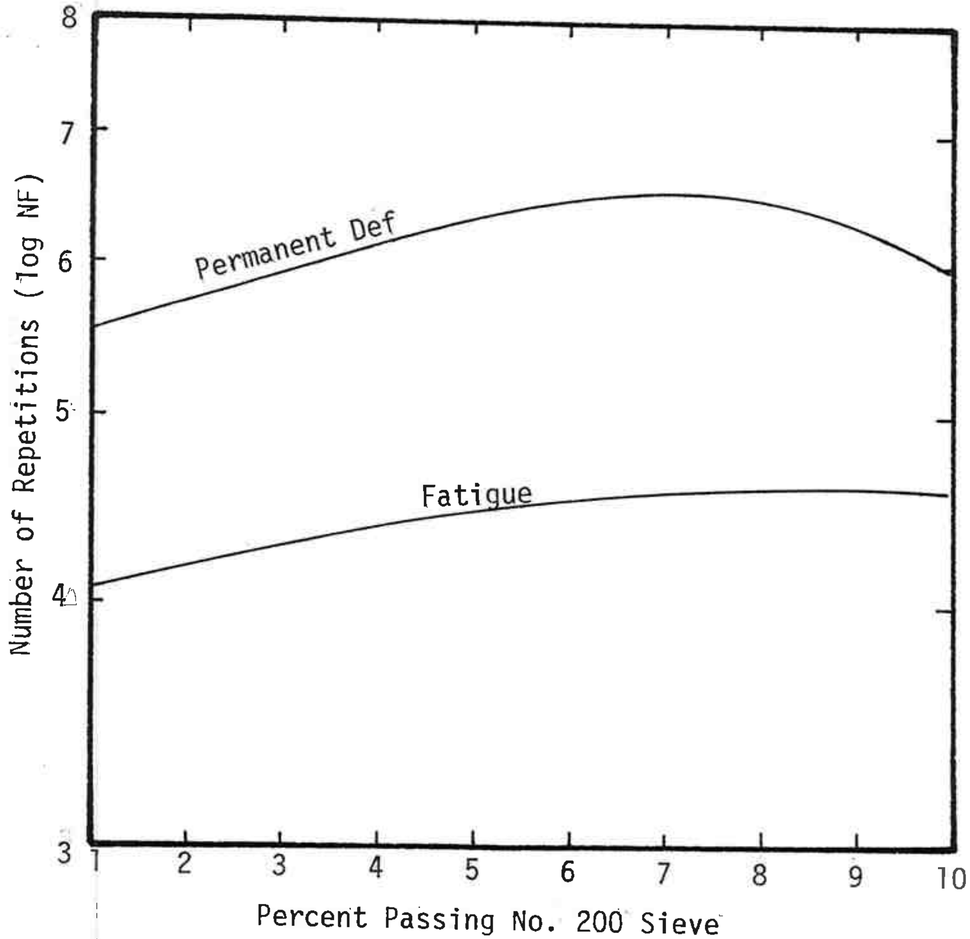


Figure 4-9 The Permanent Deformation and Fatigue Curves as a Function of Percent Passing #200 Sieve for N. Oakland-Sutherlin Project at 100 Microstrain

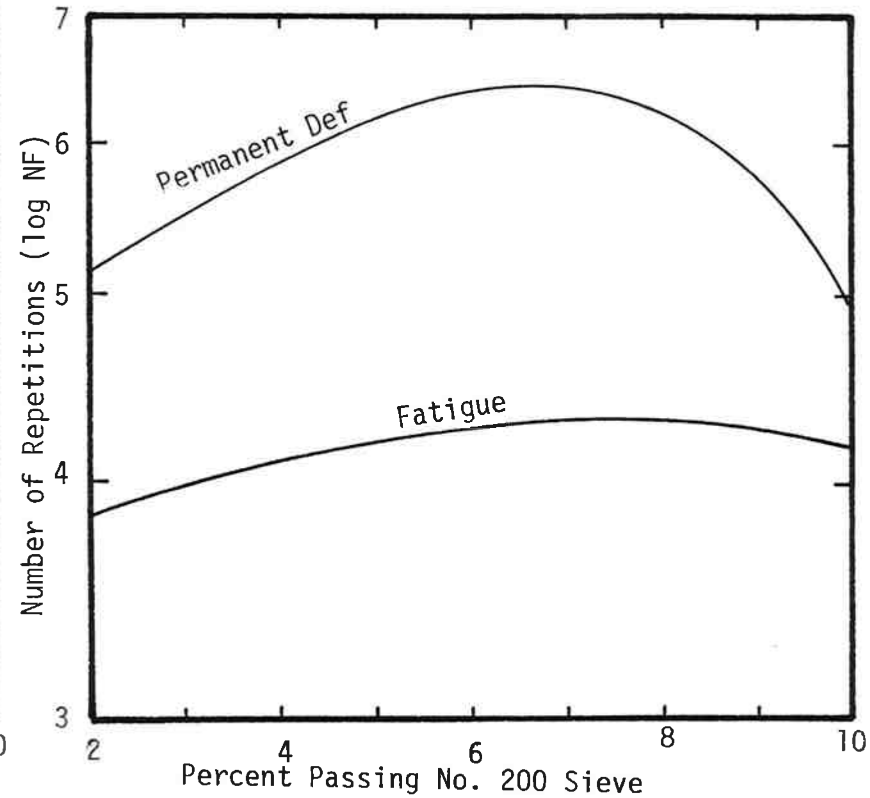


Figure 4-10 The Fatigue and Permanent Deformation Curves as a Function of Percent Passing #200 Sieve for Castle Rock-Cedar Creek Project at 100 Microstrain

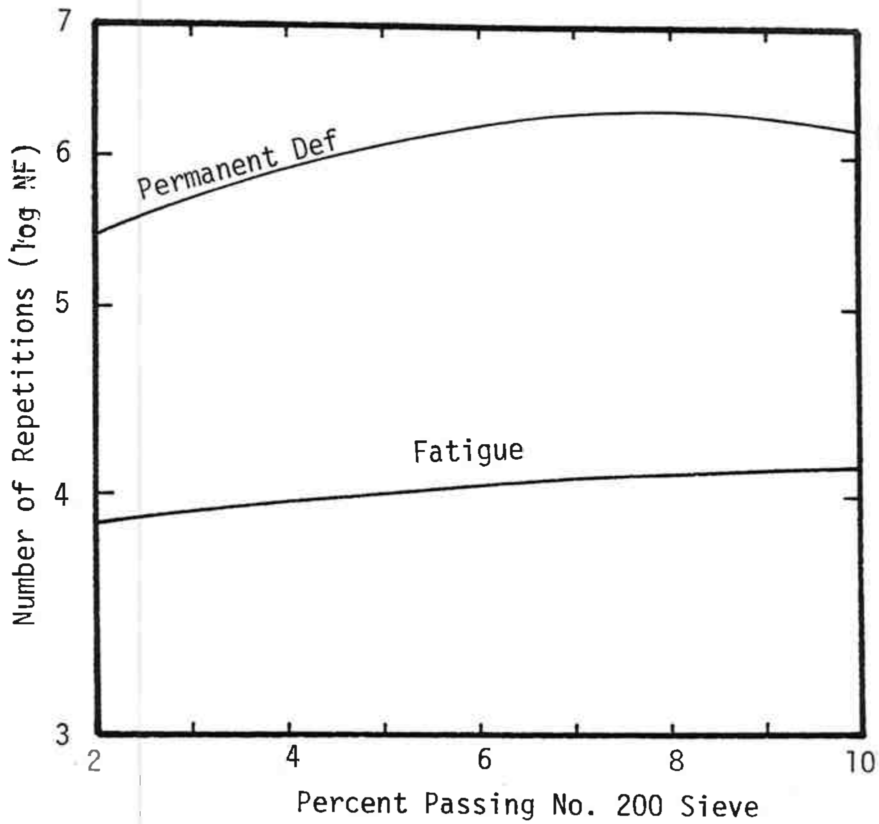


Figure 4-11 The Fatigue and Permanent Deformation Curves as a Function of Percent Passing #200 Sieve for Warren-Scappoose Project at 100 Microstrain

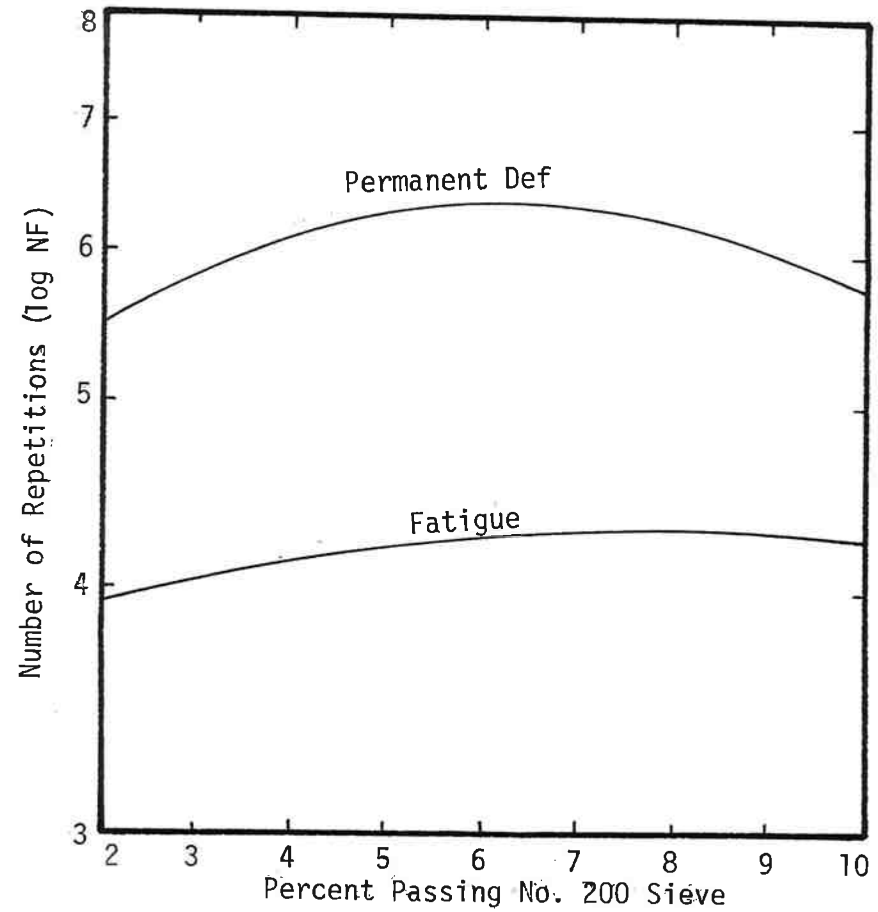


Figure 4-12 The Permanent Deformation and Fatigue Curves as a Function of Percent Passing #200 Sieve for All Projects at 100 Microstrain

The effect of variation of percent passing the No. 10 sieve was studied only on the North Oakland-Sutherlin project. Increasing the amount of percent passing No. 10 sieve caused a reduction in fatigue life but increased rutting resistance. The results from analysis again indicated that fatigue cracking is the critical factor in controlling pavement performance as far as percent passing No. 10 sieve is concerned (Fig. 4-13).

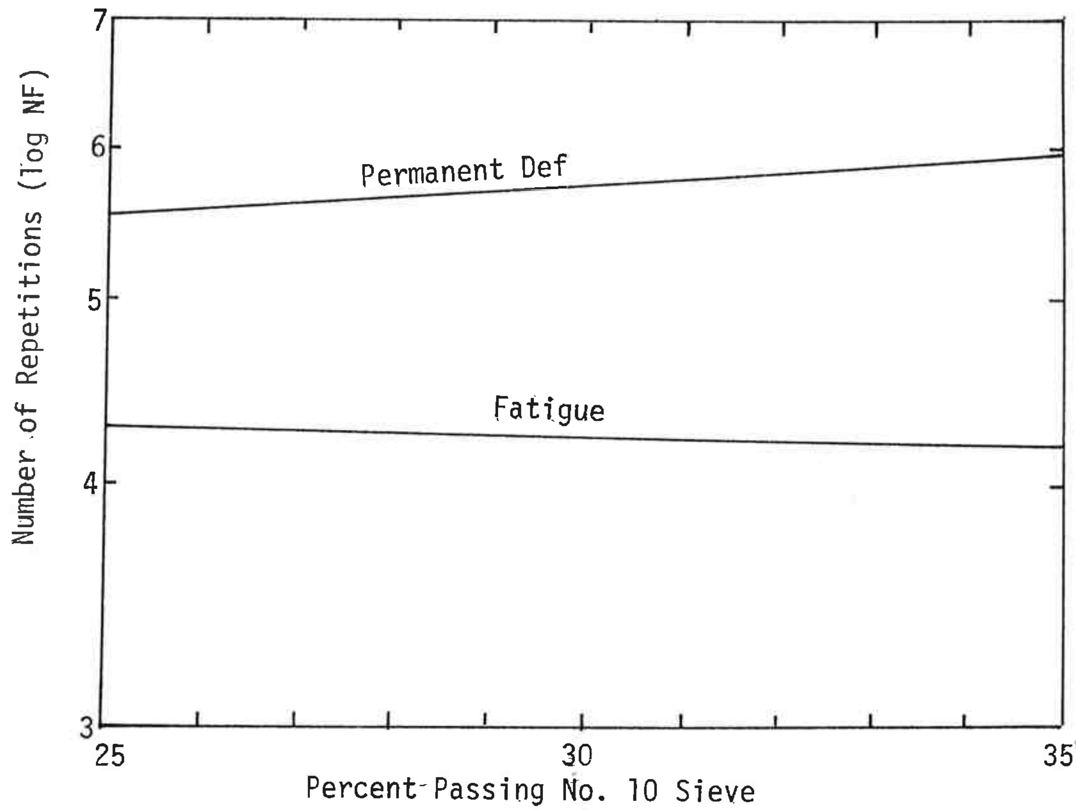


Figure 4-13. The permanent deformation and fatigue curves as a function of percent passing #10 Sieve for North Oakland-Sutherlin Project (100 Microstrain).

The results of pavement life concerning mix properties as described indicate that fatigue life is shorter than permanent deformation life. Therefore, the permanent deformation data are not included in the development of pay adjustment factors. Hence, based on the predictive model of fatigue life, the pay adjustment factors are developed to show variations in mix performance resulting from changes in percent voids, asphalt content, percent passing No. 10 and No. 200 sieve, and the minor effect of aggregate type quality.

4.3 PAY ADJUSTMENT FACTORS

The fatigue life predictive models at 100 microstrain were used in this study to develop pay adjustment factors. This was for two major reasons. First, this strain level represents an average condition typical of most roads in Oregon. Second, fatigue criteria were used because it is the most prevalent type of distress found in Oregon. Had permanent deformation criteria been used, the resulting penalties would have been much greater. Thus, the predictive models (shown in Table 10 in Appendix C) have been used to estimate the performance life of the pavement in this study. For given mix properties (asphalt content, voids, % passing No. 200, aggregate type), the design fatigue life (N_f) can be found from the predictive model. At three different confidence intervals (90%, 95%, 99%) for mean response of fatigue life models, the estimated fatigue life of a proposed pavement is obtained and evaluated to determine the reduction in pavement performance. The proposed pay adjustment factors are based on the concept that pavements constructed with design mix specifications are accepted with full payment or a pay adjustment factor of 100. A deviation from the mix specification causes a change in pavement fatigue life. Therefore, the pay adjustment factor is defined as the ratio of the fatigue life of the constructed pavement to the fatigue life of the standard proposed pavement.

$$\text{Pay Adjustment Factor} = \frac{N_f \text{ (of constructed)}}{N_f \text{ (of design standard)}} \times 100$$

The bonus payment would be granted to a greater fatigue life and a reduction of payment to shorter life compared to the design standard pavement life.

Pay adjustment factors have been developed for each mix variable (% AC, voids, % passing No. 200). The effect of aggregate type has been considered in developing the pay adjustment factor in all projects. The details are described in the following sections.

4.3.1 North Oakland-Sutherlin Pay Adjustment Factors

The pay adjustment factors shown in Table 4-1 are based on the predictive model that relates pavement fatigue life to air void content:

$$\log N_f = 5.1758 - 0.07973 \text{ (VOIDS)}$$

The high coefficient of multiple determination (R^2 = 0.8082) indicates that void content effects pavement life very significantly. The smaller the value of percent voids in the mix, the greater the pavement life. The zero voids content might be the most desirable mix to obtain the highest fatigue life, but excess deformation would result from this condition. As shown in Figure 4-4, at the void content below five percent, the fatigue life is not significantly different. It is generally believed that five percent voids might be the most desirable target design value in mix design. Thus, pay adjustment factors in this project have been established at five percent voids, eight percent voids, ten percent voids and twelve percent voids as shown in Table 4-1.

The pay adjustment factors based on asphalt content as shown in Table 4-2 were derived from the fatigue curve at optimum asphalt content level. The low coefficient of multiple determination (R^2 = 0.1327)

Table 4-1. Pay Adjustment Factors for Percent Voids for North Oakland-Sutherlin Project Using Fatigue Criteria.

DESIGN TARGET VALUE %	SPECIFICATIONS LIMITS %	NUMBER OF REPETITIONS log NF	PAF %
	Model: log NF = 5.1758 - 0.07973 (VOIDS) $R^2 = 0.8082$		
12	< 9.70	> 4.4021	152
	10.42 - 9.70	4.3449 - 4.4021	143
	11.59 - 10.42	4.2516 - 4.3449	120
	12.41 - 11.59	4.1866 - 4.2516	100
	13.58 - 12.41	4.0934 - 4.1866	83
	14.29 - 13.58	4.0361 - 4.0934	70
	> 14.29	< 4.0361	66
10	< 8.21	> 4.5211	139
	8.77 - 8.21	4.4765 - 4.5211	132
	9.60 - 8.77	4.4110 - 4.4765	116
	10.41 - 9.60	4.3460 - 4.4110	100
	11.23 - 10.41	4.2806 - 4.3460	86
	11.79 - 11.23	4.2360 - 4.2806	76
	> 11.79	< 4.2360	72
8	< 6.05	> 4.6933	143
	6.66 - 6.05	4.6447 - 4.6933	135
	7.60 - 6.66	4.5705 - 4.6447	117
	8.41 - 7.60	4.5055 - 4.5705	100
	9.34 - 8.41	4.4313 - 4.5055	85
	9.95 - 9.34	4.3827 - 4.4313	74
	> 9.95	< 4.3827	70
5	< 1.88	> 5.0256	177
	2.86 - 1.88	4.9479 - 5.0256	162
	4.59 - 2.86	4.8097 - 4.9479	126
	5.40 - 4.59	4.7447 - 4.8097	100
	7.14 - 5.40	4.6065 - 4.7447	79
	8.12 - 7.14	4.5288 - 4.6065	62
	> 8.12	< 4.5288	56

Table 4-2. Pay Adjustment Factor for Asphalt Content for North Oakland-Sutherlin Project Using Fatigue Criteria.

OPTIMUM VALUE %	SPECIFICATIONS LIMITS %	NUMBER OF REPETITIONS log NF	PAF %
Model:	$\log NF = -0.1773 + 1.1435\,(AC) - 0.00997\,(AC)^3$		
	$R^2 = 0.1327$		
	> 7.79	< 4.0187	31
	7.51 - 7.79	4.1866 - 4.0187	37
	7.38 - 7.51	4.2561 - 4.1866	48
	6.70 - 7.38	4.4855 - 4.2561	68
6.2	6.70 - 5.70	4,4855 - 4.4943	100
	4.91 - 5.70	4.2561 - 4.4943	68
	4.75 - 4.91	4.1866 - 4.2561	48
	4.42 - 4.75	4.0187 - 4.1866	37
	< 4.42	< 4.0187	31

indicates that the large variation in fatigue life with changes in asphalt content. It appears that asphalt content is not the major factor in controlling fatigue life of pavement.

The pay adjustment factors based on percent passing No. 200 sieve shown in Table 4-3 were obtained at the optimum value from the following predictive model:

$$\log N_f = 3.9438 + 0.1031\ (\#200) - 0.00042\ (\#200)^3$$

Even though the amount of fines is one of the factors controlling fatigue life, it is not the most critical factor. The low coefficient of multiple determination ($R^2 = 0.2747$) of this model indicates that only 27.5% of the variation in pavement life is explained by the amount passing No. 200 sieve. The pay adjustment factors have been developed based on the optimum amount of fines at nine percent as the design target value. One optimum amount of fines for permanent determination is much lower, about 5 to 6 percent (see Figure 4-12). These optimum values would most likely change with test temperature..

As presented in Table 4-4, the pay adjustment factors are based solely on percent passing No. 10 sieve. The coefficient of multiple determination ($R^2 = 0.6827$) of this predictive model is quite acceptable. But it does not imply that this mix variable is extremely critical to fatigue life because only one sample was considered at each percent passing level. The design target value is set at the observation value again. According to a few observations, the variation in fatigue life from this model is very high. Therefore, it is recommended that further research be conducted with more replication.

Table 4-3. Pay Adjustment Factors for Percent Passing #200 Sieve for North Oakland-Sutherlin Project Using Fatigue Criteria.

OPTIMUM VALUE %	SPECIFICATIONS LIMITS %	NUMBER OF REPETITIONS log NF	PAF %
Model: $\log NF = 3.9438 + 0.1031\ (\#200) - 0.00042\ (\#200)^3$			
$R^2 = 0.2747$			
	> 14.91	< 4.0889	34
	13.94 - 14.91	4.2428 - 4.0889	40
	13.47 - 13.94	4.3066 - 4.2423	51
	9.18 - 13.47	4.5634 - 4.3066	74
9	9.18 - 8.82	4.5634 - 4.5634	100
	3.73 - 8.82	4.3066 - 4.5633	74
	3.01 - 3.73	4.2428 - 4.3066	51
	1.42 - 3.01	4.0889 - 4.2428	40
	< 1.42	< 4.0889	34

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* Optimum for permanent deformation is about 5 to 6% for the test temperature.

Table 4-4. Pay Adjustment Factors for Percent Passing #10 Sieve for North Oakland-Sutherlin Project Using Fatigue Criteria.

DESIGN TARGET VALUE %	SPECIFICATIONS LIMITS %	NUMBER OF REPETITIONS log NF	PAF %
Model: log NF = 4.5129 - 0.00855 (#10) R^2 = 0.6827			
	> 29	< 4.2650	92
25	21 - 29	4.3333 - 4.2650	100
	< 21	> 4.3333	108
	> 34	< 4.2222	92
30	26 - 34	4.2906 - 4.2222	100
	< 26	> 4.2906	108
	> 39	< 4.1795	92
35	31 - 39	4.2479 - 4.1795	100
	< 31	> 4.2479	108

In the statistical selection procedure for the best predictive fatigue model, all mix variables were considered including polynomial effect and interaction between variables. The void content is always the first mix variable entering the model. It has a high t-Statistic value at the 95 percent confidence interval in the t-Direct search procedure. The asphalt content appears to be the second important mix variable in controlling fatigue life entering the predictive model. The final model for fatigue life as a function of all possible effects at 0.05 significant level is given by:

$$\log N_f = 6.0333 - 0.1199\ (AC) - 0.0943\ (VOIDS)$$

The fatigue predictive model indicates that either increasing or decreasing asphalt content from an optimum level and increasing voids causes reduction in pavement life. This statement is strongly supported by the high coefficient of determination (R^2 = 0.8997) of this model. The pay adjustment factors for this predictive model were developed and reported as the family of pay adjustment factors shown in Figure 4-14. The pay adjustment factors curves are based on standard mix at ten percent voids and 6.2 percent optimum asphalt content.

4.3.2 Castle Rock-Cedar Creek Pay Adjustment Factors

The pay adjustment factors of the Castle Rock-Cedar Creek project have been developed for each individual mix variable and for all possible effects. Pay adjustment factors presented in Tables 4-5 through 4-7 are based solely on percent voids, asphalt content and percent passing No. 200 sieve, respectively. The effect of void content appears to be the most dominant factor for all mix properties (R^2 = 0.9583) in estimating fatigue life. Asphalt content and the amount of fines are

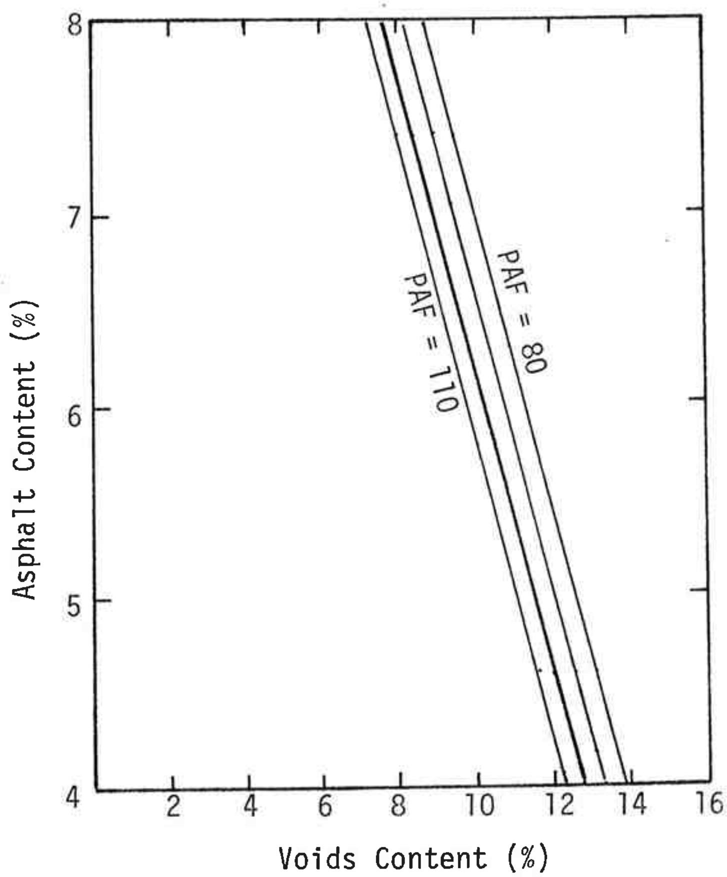

Figure 4-14 Pay Adjustment Factor Curves for Fatigue Life for N. Oakland-Sutherlin Project

Table 4-5. Pay Adjustment Factor for Percent of Voids for Castle Rock-Cedar Creek Using Fatigue Criteria.

DESIGN TARGET VALUE %	SPECIFICATIONS LIMITS %	NUMBER OF REPETITIONS log NF	PAF %
Model: $\log NF = 4.9118 - 0.00514\ (VOIDS)^2$ $R^2 = 0.9583$			
	< 11.28	> 4.2570	122
	11.51 - 11.28	4.2301 - 4.2570	118
	11.67 - 11.51	4.2187 - 4.2301	113
12	12.38 - 11.61	4.2132 - 4.2187	100
	12.47 - 12.38	4.1117 - 4.1232	88
	12.67 - 12.47	4.0848 - 4.1117	85
	> 12.67	< 4.0848	82
	< 8.94	> 4.5005	127
	9.28 - 8.94	4.4682 - 4.5005	122
	9.43 - 9.28	4.4545 - 4.4682	116
10	10.54 - 9.43	4.3401 - 4.4545	100
	10.67 - 10.54	4.3264 - 4.3401	86
	10.96 - 10.67	4.2941 - 4.3264	82
	> 10.96	< 4.2941	78
	< 6.16	> 4.7165	136
	6.79 - 6.16	4.6746 - 4.7165	130
	7.04 - 6.79	4.6567 - 4.6746	121
8	8.86 - 7.04	4.5082 - 4.6567	100
	9.05 - 8.86	4.4904 - 4.5082	83
	9.49 - 9.05	4.4485 - 4.4904	77
	> 9.49	< 4.4485	73
	< 1.28	> 4.9034	132
	2.48 - 1.28	4.8801 - 4.9034	128
5	6.62 - 2.48	4.6862 - 4.8801	100
	6.96 - 6.62	4.6629 - 4.6629	78
	7.68 - 6.96	4.6082 - 4.6629	71
	> 7.68	< 4.6082	67

Table 4-6. Pay Adjustment Factor for Asphalt Content for Castle Rock-Cedar Creek Using Fatigue Criteria.

OPTIMUM VALUE %	SPECIFICATIONS LIMITS %	NUMBER OF REPETITIONS log NF	PAF %
Model: $\text{Log NF} = -6.8514 + 3.5035\,(AC) - 0.2733\,(AC)^2$			
$R^2 = 0.3804$			
	> 7.80	< 3.8470	30
	7.80 - 7.56	3.8470 - 4.0189	36
	6.91 - 6.41	4.0189 - 4.3093	61
6.41	5.91 - 6.91	4.3092 - 4.3093	100
	5.26 - 5.91	4.0189 - 4.3092	61
	5.02 - 5.26	3.8470 - 4.0189	36
	< 5.02	< 3.8470	30 or reject

Table 4-7. Pay Adjustment Factor for Percent Passing #200 Sieve for Castle Rock-Cedar Creek Using Fatigue Criteria at 100 Microstrain.

OPTIMUM VALUE %	SPECIFICATIONS LIMITS %	NUMBER OF REPETITIONS log NF	PAF %
Model: $\log NF = 3.5307 + 0.1583\,(\#200) - 0.00094\,(\#200)^2$			
$R^2 = 0.2263$			
	> 12.13	< 3.7636	28
	11.36 - 12.13	3.7636 - 3.9438	34
	7.60 - 11.36	3.9438 - 4.3191	65
7.45	7.60 - 7.30	4.3191 - 4.3188	100
	2.73 - 7.30	3.9438 - 4.3188	65
	1.49 - 2.73	3.7636 - 3.9438	34
	< 1.49	< 3.7636	28

less important and are not found to be statistically significant in terms of the overall effect. Thus, the overall predictive model for this project is given by

$$\log N_f = 4.9118 - 0.00514\ (\text{VOIDS})^2$$

Pay adjustment factors reported in Table 4-5 are given for design air voids of five percent, eight percent, ten percent and twelve percent.

4.3.3 Warren-Scappoose Pay Adjustment Factors

The regression predictive models for estimating fatigue life used to compute pay adjustment factors are reported in Table 10 in Appendix C. The fatigue life model as a function of voids content indicates it is the most crucial factor over all other mix properties. The logarithm of fatigue life in this project can be illustrated by the following:

$$\log N_f = 4.3154 - 0.0401\ (\text{VOIDS})$$

Since the coefficient of multiple determination is high ($R^2 = 0.7605$) it can be concluded that 76.05 percent of the variation in logarithm of fatigue life is explained by void content. Increasing percent voids certainly decreases fatigue life of pavement. The pay adjustment factors for the effect of voids are developed at design target values of five percent, eight percent, ten percent and twelve percent, as shown in Table 4-8. The pay adjustment factors for asphalt content and percent passing No. 200 sieve, which are reported in Tables 4-9 and 4-10, are based on the design target value at optimum level. Even though these mix variables are minor controlling factors, deficiency in these two factors would cause severe detrimental effects on pavement life also.

Table 4-8. Pay Adjustment Factors for Percent Voids for Warren-Scappoose Project Using Fatigue Criteria at 100 Microstrain.

TARGET VALUE % VOIDS	SPECIFICATIONS LIMITS % VOIDS	NUMBER OF REPETITIONS log NF	PAF %
	Model: log NF = 4.3154 - 0.0401 (VOIDS)		
	R^2 = 0.7605		
	< 8.24	> 3.9847	141
	9.41 - 8.24	3.9376 - 3.9847	134
	9.92 - 9.41	3.9176 - 3.9376	124
12	14.08 - 9.92	3.7508 - 3.9176	100
	14.57 - 14.08	3.7308 - 3.7508	81
	15.74 - 14.57	3.6837 - 3.7308	75
	> 15.74	< 3.6837	71
	< 7.18	> 4.0271	130
	8.06 - 7.18	3.9918 - 4.0271	125
	9.84 - 8.06	3.9209 - 3.9918	110
10	10.16 - 9.84	3.9079 - 3.9209	100
	11.92 - 10.16	3.8369 - 3.9079	91
	12.80 - 11.92	3.8017 - 3.8369	80
	> 12.80	< 3.8017	77
	< 5.70	> 4.0865	124
	6.42 - 5.70	4.0578 - 4.0865	120
	7.87 - 6.42	3.9999 - 4.9578	108
8	8.13 - 7.87	3.9893 - 3.9999	100
	9.57 - 8.13	3.9314 - 3.9893	92
	10.28 - 9.57	3.9027 - 3.9314	84
	> 10.28	< 3.9027	81
	< 2.18	> 4.2279	130
	3.06 - 2.18	4.1926 - 4.2279	125
	4.84 - 3.06	4.1214 - 4.1926	110
5	5.16 - 4.84	4.1084 - 4.1214	100
	6.93 - 5.16	4.0372 - 4.1084	91
	7.81 - 6.93	4.0019 - 4.0372	80
	> 7.81	< 4.0019	77

Table 4-9. Pay Adjustment Factor for Asphalt Content for Warren-Scappoose Project Using Fatigue Criteria at 100 Microstrain.

OPTIMUM VALUE %	SPECIFICATIONS LIMITS %	NUMBER OF REPETITIONS log NF	PAF %
	Model: $\log NF = 1.9253 + 0.6535\ (AC) - 0.04879\ (AC)^2$		
	$R^2 = 0.3561$		
	> 9.63	< 3.6945	38
	9.10 - 9.63	3.6945 - 3.8303	45
	7.20 - 9.10	4.1009 - 3.8303	71
6.70	7.20 - 6.20	4.1009 - 4.1014	100
	4.29 - 6.20	3.8303 - 4.1014	72
	3.77 - 4.29	3.6945 - 3.8303	45
	< 3.77	< 3.6945	38

Table 4-10. Pay Adjustment Factor for Percent Passing #200 Sieve for Warren-Scappoose Project Using Fatigue Criteria at 100 Microstrain.

OPTIMUM VALUE %	SPECIFICATIONS LIMITS %	NUMBER OF REPETITIONS log NF	PAF %
Model: $\log NF = 3.7795 + 0.0429\ (\#200) - 0.00007\ (\#200)^3$ $R^2 = 0.3108$			
14.32	< 14.03	< 4.1898	62
	14.61 - 14.03	4.1898 - 4.1898	100
	14.61 - 32.11	4.1898 - 2.8443	21
	> 32.11	< 2.8443	Reject

4.3.4 Combined Pay Adjustment Factors

As discussed earlier, fatigue life of pavement varies widely with several mix properties. The major ones are percent voids, asphalt content and percent passing No. 200 sieve. A factor which directly affects these mix variables is aggregate type and quality. As shown in Table 10 in Appendix C, all the fatigue predictive models had their coefficients of multiple determination (R^2) very much reduced when aggregate type was not included in the model. Reducing the R-squared means more variation in fatigue life associated with the use of the set of mix variables. As exhibited in Tables 4-11 through 4-13, the pay adjustment factors are based solely on voids contents, asphalt content, and the amount of fines, respectively. Table 4-14 presents the pay adjustment factors considering all possible mix properties but not including aggregate type effect. The model as shown in Table 4-14 only explains 33 percent of the variation in logarithm of fatigue life as controlled by void content. Hence, all these predictive models clearly illustrate the effect of aggregate type on variation of mix characteristics. Differences in aggregate type used result in different mix characteristics, which directly affect fatigue life and, of course, pay adjustment factors.

4.4 COMPARISON WITH EXISTING PAY ADJUSTMENT FACTORS

The successful implementation of pay adjustment factors is dependent upon their reliability and ease of use. While the format should be relatively simple, it must be based on statistical and sound engineering principles to ensure good performance. The pay adjustment factors developed in this (Puangchit et al.) are compared to those currently in

Table 4-11. Pay Adjustment Factors for Percent Voids of the Three Projects Using Fatigue Criteria.

DESIGN TARGET VALUE %	SPECIFICATIONS LIMITS %	NUMBER OF REPETITIONS log NF	PAF %
Model: $\log NF = 4.5072 - 0.00295\ (VOIDS)^2$			
$R^2 = 0.3608$			
10	< 7.38	> 4.3467	136
	8.14 - 7.38	4.3119 - 4.3467	131
	8.48 - 8.14	4.2951 - 4.3119	123
	11.31 - 8.48	4.1298 - 4.2951	100
	11.57 - 11.31	4.1129 - 4.1298	81
	12.06 - 11.57	4.0783 - 4.1129	76
	> 12.06	< 4.0783	73
8	< 3.29	> 4.4754	144
	4.96 - 3.29	4.4348 - 4.4754	137
	7.79 - 4.96	4.3282 - 4.4348	116
	8.20 - 7.79	4.3089 - 4.3282	100
	10.16 - 8.20	4.2024 - 4.3089	87
	10.82 - 10.16	4.1618 - 4.2024	74
	> 10.82	< 4.1618	70
5	< 4.54	> 4.4464	103
	5.41 - 4.54	4.4207 - 4.4464	100
	8.82 - 5.41	4.2781 - 4.4207	82
	9.81 - 8.82	4.2238 - 4.2781	66
	> 9.81	< 4.2238	62

Table 4-12. Pay Adjustment Factor for Percent Asphalt Content of the Three Projects Using Fatigue Criteria.

OPTIMUM VALUE %	SPECIFICATIONS LIMITS %	NUMBER OF REPETITIONS log NF	PAF %
Model:	$\log NF = 0.7705 + 0.8131\,(AC) - 0.00625\,(AC)^3$		
	$R^2 = 0.2248$		
	> 7.84	< 4.1317	62
	7.67 - 7.84	4.1858 - 4.1317	66
	7.10 - 7.67	4.2121 - 4.1858	72
6.6	6.10 - 7.10	4.2121 - 4.2121	100
	6.10 - 5.43	4.2121 - 4.1858	72
	5.43 - 5.23	4.1858 - 4.1317	66
	< 5.23	< 4.1317	62

Table 4-13. Pay Adjustment Factor for Percent Passing #200 Sieve of the Three Projects Using Fatigue Criteria.

OPTIMUM VALUE %	PERCENT PASSING #200 SIEVE %	NUMBER OF REPETITIONS log NF	PAF %
Model: $\log NF = 3.6938 + 0.1456\ (\#200) - 0.00863\ (\#200)^2$			
$R^2 = 0.1543$			
	> 13.50	< 4.0867	60
	12.80 - 13.50	4.1441 - 4.0867	64
	8.67 - 12.80	4.3074 - 4.1441	83
8.5	8.67 - 8.33	4.3074 - 4.3078	100
	8.33 - 4.08	4.3078 - 4.1441	83
	4.08 - 3.37	4.1441 - 4.0867	64
	< 3.37	< 4.0867	60

Table 4-14. Pay Adjustment Factor for Combined Mixture Properties of the Three Projects Using Fatigue Criteria.

DESIGN TARGET VALUE %	REQUIRED VALUE %	NUMBER OF REPETITIONS log NF	PAF %
Model: $\log NF = 4.6875 - 0.05103$ (VOIDS)			
$R^2 = 0.3300$			
	< 7.29	> 4.3155	138
	7.99 - 7.29	4.2797 - 4.3155	132
	8.33 - 7.99	4.2623 - 4.2797	124
10	11.67 - 8.33	4.0919 - 4.2623	100
	12.01 - 11.67	4.0919 - 4.1771	91
	12.71 - 12.01	4.0388 - 4.0746	76
	> 12.71	< 4.0388	73
	< 5.08	> 4.4280	147
	5.84 - 5.08	4.3895 - 4.4280	135
	7.82 - 5.84	4.2886 - 4.3895	115
8	8.19 - 7.82	4.2698 - 4.2886	100
	10.16 - 8.19	4.1680 - 4.2698	87
	10.92 - 10.16	4.1303 - 4.1688	74
	> 10.92	< 4.1303	71
	< 0.70	> 4.6521	166
	1.87 - 0.70	4.5920 - 4.6521	155
	4.73 - 1.87	4.4462 - 4.5920	122
5	5.27 - 4.73	4.4184 - 4.4462	100
	8.19 - 5.27	4.2694 - 4.4184	82
	9.31 - 8.19	4.2125 - 4.2694	64
	> 9.31	< 4.2125	60

use in Tables 4-15 through 4-17. Those shown for this study were developed by plotting, as a continuous function, the pay adjustment factor versus the indicated deviation from the specification limit. The factors shown for density resulted from the data in Table 4-11 for a design void content of 8 percent. Tables 4-12 and 4-13 were used to develop the pay factors for asphalt content and percent passing the No. 200 sieve, respectively. The pay adjustment factors developed in this study (Puangchit et al.) are compared to those currently in use in Tables 4-15 through 4-17.

The pay factors relating to density (Table 4-15) are comparable to those of the Oregon Quality Assurance specification (R=4), except that a higher bonus is recommended for good compaction and a greater penalty recommended for poor compaction, reflecting the significant effect that compaction has on pavement life. Density was shown to be the most significant variable and regarded as the only one to which a bonus payment should be applied; this is not the case with the Oregon Quality Assurance specifications where bonus payments are possible for mixes having constituents close to the target values.

The pay factors relating to asphalt content (Table 4-16) are significantly more severe than the others shown, with the exception of the FHWA specification which appears to penalize material within the normally accepted range of ± .5% of the target value and allows for rejection outside that range. This is because the FHWA specification recognizes that when a mean value from five tests results is used, there is less probability of that value differing from the target value by a certain amount than if fewer tests were used.

The pay factors based on percent passing No. 200 sieve (Table 4-17) are slightly more severe than the others shown, with the exception of the FHWA specification for similar reasons to those stated above for asphalt content.

Table 4-15. Pay Factor Comparison for Compaction.

Percent Compaction		Oregon Standard	Washington DOT	Puangchit et al.	Oregon Q.A. R = 1***	Oregon Q.A. R = 2***	Oregon Q.A. R = 4***
95.0+	=	-	-	110	-	-	-
92.0 - 94.9	=	-	-	100	-	-	-
94.0+	=	-	-	-	102	102	102
92.0+	=	-	100	-	100	100	100
91.5+	=	100	-	-	-	-	-
91.0 - 91.9	=	-	95	94	**96.7	**95.4	**92.7
91.0 - 91.4	=	*99.8	-	-	-	-	-
90.0 - 90.9	=	*99.6	90	87	**92.7	**91.4	**88.7
89.0 - 89.9	=	*99.3	80	80	**88.7	**87.4	(84.7)
Below 89.0	=	-	(50 (maybe)	70 (reject)	-	-	-
88.0 - 88.9	=	*98.8	-	-	(84.7)	(83.4)	(80.7)
87.0 - 87.9	=	*98.2	-	-	(80.7)	(79.4)	(76.7)
Below 87.0	=	less pay	-	-	(76.7)	(75.4)	(72.7)
Target Density, % Voids	=	1 - 3	0.0	0.0	1 - 3	1 - 3	1 - 3

* Values calcuated using current guidelines for pavement 16' wide, 2" thickness, $\frac{1}{2}$ mile length, 140#/ft^3 and $23/ton.

** Values calculated at the midpoint of % compaction range.

Values in parentheses are in excess of the 15% maximum payment reduction for Oregon Quality Assurance specification. Engineer may accept at 15% reduction to no payment.

*** R = range.

Table 4-16. Pay Factor Comparison - Asphalt Content

Tolerance for Percent Asphalt	Oregon* Standard	FHWA (6-15-80)	Puangchit*** et al.	Oregon Q.A.**** R = 0.1†	Oregon Q.A.**** R = 0.2†	Oregon Q.A.**** R = 0.4†
± 0.35	100.0	100.0	100.0	100.0	100.0	100.0
± 0.40	100.0	95.0	100.0	100.0	100.0	100.0
± 0.45	100.0	85.0	100.0	100.0	100.0	100.0
± 0.50	100.0	**	100.0	100.0	100.0	100.0
± 0.60	99.2	**	96.5	97.1	96.3	94.9
± 0.70	98.4	**	93.0	94.9	94.1	92.7
± 0.80	97.6	**	89.0	92.7	91.9	90.5
± 0.90	96.8	**	85.5	90.5	89.7	88.3
± 1.00	96.0	**	80.5	88.3	87.5	86.1
± 1.10	95.2	**	76.0	86.1	85.3	83.9
± 1.20	94.4	**	71.0	83.9	83.1	81.7
± 1.30	93.6	**	66.0	81.7	80.9	79.5
± 1.40	92.8	**	60.0	79.5	78.7	77.3

* For an individual field test with testing of a backup sample for failing samples to confirm results.

** May be accepted at a pay factor of 70% or rejected. For an average of five tests.

*** Based on fatigue criteria on specimens outside the ODOT tolerance of ± 0.5%. For an average of five tests.

**** Oregon 1982 Quality Assurance specification.

† R = range.

Table 4-17. Pay Factor Comparison - Gradation Pass No. 200 Sieve.

Tolerance for Percent Pass No. 200	Oregon* Standard	FHWA (6-15-80)	Puangchit*** et al.	Oregon Q.A.**** R = 0.1†	Oregon Q.A.**** R = 0.2†	Oregon Q.A.**** R = 0.4†
± 1.7	100.0	100.0	100.0	100.0	100.0	100.0
± 1.8	100.0	95.0	100.0	100.0	100.0	100.0
± 1.9	100.0	95.0	100.0	100.0	100.0	100.0
± 2.0	100.0	85.0	100.0	100.0	100.0	100.0
± 2.1	99.9	85.0	99.1	99.2	98.6	97.0
± 2.2	99.8	**	98.1	98.6	98.0	96.4
± 2.3	99.6	**	97.0	98.0	97.4	95.8
± 2.4	99.5	**	95.8	97.4	96.8	95.2
± 2.5	99.4	**	94.5	96.8	96.2	94.6
± 2.6	99.2	**	93.1	96.2	95.6	94.0
± 2.7	99.1	**	91.6	95.6	95.0	93.4
± 2.8	99.0	**	90.0	95.0	94.4	92.8
± 2.9	98.9	**	88.3	94.4	93.2	92.2
± 3.0	98.8	**	86.5	93.8	93.2	91.6

* For an individual field test with testing of a backup sample for failing samples to confirm results.

** May be accepted at a pay factor of 70% or rejected. For an average of five tests.

*** Based on fatigue criteria on specimens outside the ODOT tolerance of ± 2.0%. For an average of five tests.

**** Oregon 1982 Quality Assurance specification.

† R = range.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The impact of variations in material quality, construction and environment cause uncertainty in the performance of asphalt concrete pavements. The influence of variations in asphalt concrete mix characteristics on pavement performance was studied on the North Oakland-Sutherlin, Castle Rock-Cedar Creek and Warren-Scappoose projects. The results from a computer statistical analysis led to the following conclusions.

1. Of the variables considered, mix density (or air void content) had the most important influence on predicted performance. An increase in void content is associated with a decrease in performance life. Thus, for fatigue criteria a small air void content mix (within limits) is most desirable in asphalt concrete pavement construction. Quality control efforts should be focused on obtaining adequate compaction and low air void content.
2. Results of analyses suggest that a maximum performance life occurs at an optimum asphalt content. It is interesting to note that an optimum content required for maximum permanent deformation resistance is always less than for highest fatigue life on all projects. It is also obvious that performance life is decreased as asphalt content deviates from an optimum value.
3. The amount of fines also affected mix behavior. Similar to asphalt content, the best performance is obtained at

an optimum value. Deviations from this value caused reductions in mix performance life. The permanent deformation required less fines to reach the highest performance than fatigue cracking. Thus, the maximum allowable percent fines is controlled by deformation limits.

4. Aggregate type exhibited an indirect affect on mix performance. Aggregate type is important in that it controls the amount of asphalt required and limits the percent air voids that is incorporated in the mix. At the same air void and asphalt content the mix performance still varies with the aggregate type used.
5. The pay adjustment factors are based on the concept that the performance life at design target value obtained from the predictive model are accepted with full payment. The other pay adjustment factors are obtained by comparing the performance to a standard design mix.
6. The pay adjustment factors developed compare favorably with those used by other agencies in the Pacific Northwest.

It should be emphasized that all conclusions are based on fatigue criteria and on results of tests conducted at ambient temperatures (22°C ± 2°C). The results would be expected to change at higher temperatures (critical for permanent deformation) as well as at lower temperatures (critical for fatigue cracking). Further, had permanent deformation criteria been used in the development of the pay adjustment factors, the reductions in pavement life (associated with mix variations) would have been greater than that reported herein.

5.2 RECOMMENDATIONS

Based on the analyses of data in Chapter 4, it is recommended that consideration initially be given to the adoption of a compaction specification along the line of that presented in Table 4-11 using the eight percent voids as a maximum design value. The upper limit on the bonus should be ten percent maximum as initial void contents lower than five percent may lead to other problems such as permanent deformation. Further, it is felt that if higher compaction requirements are imposed, contractors will need better control of asphalt content and gradation in order to achieve the higher density values.

6.0 REFERENCES

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APPENDIX A

Number of Repetitions to Failure by

Fatigue and Permanent Deformation Criteria

for Variation in Each Mix Property

[Source References 29,30,31]

Figure 1. 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
100%					X				
96%					X				
92%	X		X	X	X	X	X		X
91%									

Figure 2. Range of Mix Variables Considered in This Study
(Crossed Boxes)
Castle Rock-Cedar Creek 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
100%					X				
97%					X				
92%	X		X	X	X	X	X		X
90%					X				

Figure 3. Range of Mix Variables Considered in This Study
(Crossed Boxes)
Warren-Scappoose Project

LEVEL OF COMPACTION	2% PASSING NO. 200			6% PASSING NO. 200			10% PASSING NO. 200		
	ASPHALT CONTENT			ASPHALT CONTENT			ASPHALT CONTENT		
	4.5	5.5	6.5	4.5	5.5	6.5	4.5	5.5	6.5
100%					X				
96%					X				
91%	X		X		X	X	X		X
90%					X				

Table 1. Number of Repetitions to Fatigue Failure and Associated Pay Factors for Four Levels of Mix Density, North Oakland-Sutherlin Project [29].

	LEVEL OF COMPACTION	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 $\mu\varepsilon$	STRAIN LEVEL 100 $\mu\varepsilon$	STRAIN LEVEL 125 $\mu\varepsilon$
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 2. Number of Repetitions to Fatigue Failure and Associated Pay Factors for Three Levels of Asphalt Content at 6% Passing No. 200, North Oakland-Sutherlin Project [29].

	ASPHALT CONTENT	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 με	STRAIN LEVEL 100 με	STRAIN LEVEL 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

Table 3. Number of Repetitions to Fatigue Failure and Associated Pay Factors for Three Levels of Percent Passing No. 200 at 5% Asphalt Content, North Oakland-Sutherlin Project [29].

	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 4. Number of Repetitions to Fatigue Failure and Associated Pay Factors for Three Levels of Percent Passing No. 200 at 7% Asphalt Content, North Oakland-Sutherlin Project [29].

	PERCENT P_{200}	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 με	STRAIN LEVEL 100 με	STRAIN LEVEL 125 με
PAVEMENT LIFE	Standard 6% P_{200}	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% P_{200}	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 5. Number of Repetitions to Fatigue Failure and Associated Pay Factors for Three Levels of Percent Passing No. 10, North Oakland-Sutherlin Project [29].

	PERCENT P_{10}	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 $\mu\varepsilon$	100 $\mu\varepsilon$	125 $\mu\varepsilon$
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

Table 6. Number of Repetitions to Permanent Deformation Failure and Associated Pay Factors for Four Levels of Mix Density, North Oakland-Sutherlin Project [29].

	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 7. Number of Repetitions to Permanent Deformation Failure and Associated Pay Factors for Three Levels of Asphalt Content, North Oakland-Sutherlin Project [29].

	ASPHALT CONTENT	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 με	STRAIN LEVEL 100 με	STRAIN LEVEL 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 8. Number of Repetitions to Permanent Deformation Failure and Associated Pay Factors for Three Levels of Percent Passing No. 200 at 5% Asphalt Content, North Oakland-Sutherlin Project [29].

	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 9. Number of Repetitions to Permanent Deformation Failure and Associated Pay Factors for Three Levels of Percent Passing No. 200 at 7% Asphalt Content, North Oakland-Sutherlin Project [29].

	PERCENT P_{200}	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 $\mu\varepsilon$	100 $\mu\varepsilon$	125 $\mu\varepsilon$
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 10. Number of Repetitions to Permanent Deformation Failure and Associated Pay Factors for Three Levels of Percent Passing No. 10, North Oakland-Sutherlin Project [29].

	PERCENT P_{10}	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 με	100 με	125 με
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

Table 11. Number of Repetitions to Fatigue Failure and Associated Pay Factors for Mix Density, Castle Rock-Cedar Creek Project [30].

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

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

Table 12. Number of Repetitions to Fatigue Failure and Associated Pay Factors for Asphalt Content Effect at 6% Passing No. 200 and 92% Compaction, Castle Rock-Cedar Creek Project [30].

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

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

Table 13. Number of Repetitions to Fatigue Failure and Associated Pay Factors for Percent Passing No. 200 Effect at 5% Asphalt Content, Castle Rock-Cedar Creek Project [30].

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

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

Table 14. Number of Repetitions to Fatigue Failure and Associated Pay Factors for Percent Passing No. 200 at 7% Asphalt Content, Castle Rock-Cedar Creek Project [30].

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

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

Table 15. Number of Repetitions to Permanent Deformation Failure and Associated Pay Factors for Mix Density Effect, Castle Rock-Cedar Creek Project [30].

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

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

Table 16. Number of Repetitions to Permanent Deformation Failure and Associated Pay Factors for Asphalt Content Effect at 6% Passing No. 200 and 92% Compaction, Castle Rock-Cedar Creek Project [30].

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

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

Table 17. Number of Repetitions to Permanent Deformation Failure and Associated Pay Factors for Percent Passing No. 200 at 5% Asphalt Content, Castle Rock-Cedar Creek Project [30].

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

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

Table 18. Number of Repetitions to Permanent Deformation Failure and Associated Pay Factors for Percent Passing No. 200 at 7% Asphalt Content, Castle Rock-Cedar Creek Project [30].

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

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

Table 19. Number of Repetitions to Fatigue Failure and Associated Pay Factors for Mix Density, Warren-Scappoose Project [31].

	LEVEL OF COMPACTION	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 $\mu\varepsilon$	100 $\mu\varepsilon$	150 $\mu\varepsilon$
PAVEMENT LIFE	Standard 96%	2.39	A.C.	9.66×10^4	1.29×10^4	3.99×10^3
			C.	9.87×10^4	1.57×10^4	5.37×10^3
	100%	2.45	A.C.	1.16×10^5	1.40×10^4	4.07×10^3
			C.	1.75×10^5	1.78×10^4	4.66×10^3
	91%	2.29	A.C.	4.48×10^4	1.04×10^4	4.41×10^3
			C.	3.91×10^4	1.09×10^4	5.13×10^3
	90%	2.20	A.C.	2.08×10^4	7.95×10^3	4.54×10^3
			C.	5.73×10^4	1.36×10^4	5.83×10^3
PAY FACTOR	Standard 96%	2.39	A.C.	1.0	1.0	1.0
			C.	1.0	1.0	1.0
	100%	2.45	A.C.	1.20	1.09	1.02
			C.	1.77	1.13	.87
	91%	2.29	A.C.	.46	.81	1.11
			C.	.40	.69	.96
	90%	2.20	A.C.	.22	.62	1.14
			C.	.58	.87	1.09

* A.C. - As Compacted
C. - Conditioned

Table 20. Number of Repetitions to Fatigue Failure and Associated Pay Factors for Asphalt Content Effect at 6% Passing No. 200 and 93% Compaction, Warren-Scappoose Project [31].

	PERCENT ASPHALT CONTENT	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 με	STRAIN LEVEL 100 με	STRAIN LEVEL 150 με
PAVEMENT LIFE	Standard 5.5%	2.29	A.C.	4.48×10^4	1.04×10^4	4.41×10^3
			C.	3.91×10^4	1.09×10^4	5.13×10^3
	4.5%	2.27	A.C.	1.96×10^4	7.79×10^3	4.54×10^3
			C.	4.25×10^4	1.19×10^4	5.63×10^3
	6.5%	2.30	A.C.	7.15×10^4	1.17×10^4	4.06×10^3
			C.	4.50×10^4	1.28×10^4	6.16×10^3
PAY FACTOR	Standard 5.5%	2.2	A.C.	1.0	1.0	1.0
			C.	1.0	1.0	1.0
	4.5%	2.27	A.C.	.44	.75	1.03
			C.	1.09	1.09	1.10
	6.5%	2.30	A.C.	1.60	1.13	.92
			C.	1.15	1.17	1.20

* A.C. - As Compacted
C. - Conditioned

Table 21. Number of Repetitions to Fatigue Failure and Associated Pay Factors for Percent Passing No. 200 at 4.5% Asphalt Content, Warren-Scappoose Project [31].

	PERCENT P_{200}	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 $\mu\varepsilon$	100 $\mu\varepsilon$	150 $\mu\varepsilon$
PAVEMENT LIFE	Standard 6%	2.27	A.C.	1.96×10^4	7.79×10^3	4.54×10^3
			C.	4.25×10^4	1.19×10^4	5.63×10^3
	2%	2.21	A.C.	1.19×10^4	6.89×10^3	5.01×10^3
			C.	1.65×10^4	9.38×10^3	6.76×10^3
	10%	2.30	A.C.	2.10×10^4	8.02×10^3	4.57×10^3
			C.	6.28×10^4	1.32×10^4	5.30×10^3
PAY FACTOR	Standard 6%	2.27	A.C.	1.0	1.0	1.0
			C.	1.0	1.0	1.0
	2%	2.21	A.C.	.61	.88	1.10
			C.	.39	.79	1.20
	10%	2.30	A.C.	1.07	1.03	1.01
			C.	1.48	1.11	.94

* A.C. - As Compacted
C. - Conditioned

Table 22. Number of Repetitions to Fatigue Failure and Associated Pay Factors for Percent Passing No. 200 at 6.5% Asphalt Content, Warren-Scappoose Project [31].

	PERCENT P_{200}	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 $\mu\varepsilon$	STRAIN LEVEL 100 $\mu\varepsilon$	STRAIN LEVEL 150 $\mu\varepsilon$
PAVEMENT LIFE	Standard 6%	2.30	A.C.	7.15×10^4	1.17×10^4	4.06×10^3
			C.	4.50×10^4	1.28×10^4	6.16×10^3
	2%	2.24	A.C.	2.00×10^4	7.79×10^3	4.49×10^3
			C.	2.80×10^4	1.13×10^4	6.64×10^3
	10%	2.39	A.C.	5.83×10^5	2.37×10^4	3.64×10^3
			C.	3.01×10^5	2.13×10^4	4.53×10^3
PAY FACTOR	Standard 6%	2.30	A.C.	1.0	1.0	1.0
			C.	1.0	1.0	1.0
	2%	2.24	A.C.	.28	.67	1.11
			C.	.62	.88	1.08
	10%	2.39	A.C.	8.15	2.03	.90
			C.	6.69	1.66	.74

* A.C. - As Compacted
C. - Conditioned

Table 23. Number of Repetitions to Permanent Deformation Failure and Associated Pay Factors for Mix Density, Warren-Scappoose Project [31].

	LEVEL OF COMPACTION	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 $\mu\varepsilon$	100 $\mu\varepsilon$	150 $\mu\varepsilon$
PAVEMENT LIFE	Standard 96%	2.39	A.C.	1.55×10^7	4.55×10^6	2.08×10^6
			C.	1.38×10^7	3.15×10^6	1.30×10^6
	100%	2.45	A.C.	1.42×10^8	4.51×10^7	2.04×10^7
			C.	2.08×10^9	4.87×10^8	1.83×10^8
	91%	2.29	A.C.	1.42×10^6	6.92×10^5	4.09×10^5
			C.	2.71×10^6	8.07×10^5	3.81×10^5
	90%	2.20	A.C.	3.67×10^5	2.56×10^5	1.88×10^5
			C.	1.03×10^6	3.21×10^5	1.49×10^5
PAY FACTOR	Standard 96%	2.39	A.C.	1.0	1.0	1.0
			C.	1.0	1.0	1.0
	100%	2.45	A.C.	9.15	9.92	9.81
			C.	151	154	141
	91%	2.29	A.C.	.09	.15	.20
			C.	.20	.26	.29
	90%	2.20	A.C.	.02	.06	.09
			C.	.07	.10	.12

* A.C. - As Compacted
C. - Conditioned

Table 24. Number of Repetitions to Permanent Deformation Failure and Associated Pay Factors for Asphalt Content Effect at 6% Passing No. 200 and 93% Compaction, Warren-Scappoose Project [31].

	ASPHALT CONTENT	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 με	100 με	150 με
PAVEMENT LIFE	Standard 5.5%	2.29	A.C.	1.42×10^6	6.92×10^5	4.09×10^5
			C.	2.71×10^6	8.07×10^5	3.81×10^5
	4.5%	2.27	A.C.	2.30×10^6	1.62×10^6	1.20×10^6
			C.	1.83×10^6	8.85×10^5	5.17×10^5
	6.5%	2.30	A.C.	1.77×10^6	4.84×10^5	2.25×10^5
			C.	1.22×10^6	5.29×10^5	2.94×10^5
PAY FACTOR	Standard 5.5%	2.29	A.C.	1.0	1.0	1.0
			C.	1.0	1.0	1.0
	4.5%	2.27	A.C.	1.62	2.34	2.93
			C.	.67	1.10	1.35
	6.5%	2.30	A.C.	1.25	.70	.55
			C.	.45	.66	.77

* A.C. - As Compacted
C. - Conditioned

Table 25. Number of Repetitions to Permanent Deformation Failure and Associated Pay Factors for Percent Passing No. 200 at 4.5% Asphalt Content, Warren-Scappoose Project [31].

	PERCENT P_{200}	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 $\mu\varepsilon$	100 $\mu\varepsilon$	150 $\mu\varepsilon$
PAVEMENT LIFE	Standard 6%	2.27	A.C.	2.30×10^6	1.62×10^6	1.20×10^6
			C.	1.83×10^6	8.85×10^5	5.17×10^5
	10%	2.30	A.C.	3.00×10^6	4.92×10^5	1.98×10^5
			C.	1.52×10^7	1.44×10^6	4.72×10^5
	2%	2.21	A.C.	6.93×10^5	3.34×10^5	1.98×10^5
			C.	1.49×10^6	6.33×10^5	3.49×10^5
PAY FACTOR	Standard 6%	2.27	A.C.	1.0	1.0	1.0
			C.	1.0	1.0	1.0
	10%	2.30	A.C.	1.30	.31	.17
			C.	8.30	1.62	.92
	2%	2.21	A.C.	.30	.21	.17
			C.	.81	.72	.68

* A.C. - As Compacted
C. - Conditioned

Table 26. Number of Repetitions to Permanent Deformation Failure and Associated Pay Factors for Percent Passing No. 200 at 6.5% Asphalt Content, Warren-Scappoose Project [31].

	PERCENT P_{200}	MIX BSG	TEST CONDITION*	STRAIN LEVEL 50 $\mu\varepsilon$	100 $\mu\varepsilon$	150 $\mu\varepsilon$
PAVEMENT LIFE	Standard 6%	2.30	A.C.	1.77×10^6	4.84×10^5	2.25×10^5
			C.	1.22×10^6	5.29×10^5	2.94×10^5
	10%	2.39	A.C.	1.79×10^7	5.46×10^6	2.46×10^6
			C.	7.00×10^7	6.69×10^6	1.94×10^6
	2%	2.24	A.C.	6.69×10^5	2.95×10^5	1.69×10^5
			C.	9.42×10^5	4.26×10^5	2.41×10^5
PAY FACTOR	Standard 6%	2.30	A.C.	1.0	1.0	1.0
			C.	1.0	1.0	1.0
	10%	2.39	A.C.	10.1	11.3	10.9
			C.	57.6	12.6	6.6
	2%	2.24	A.C.	.38	.61	.75
			C.	.78	.81	.82

* A.C. - As Compacted
C. - Conditioned

APPENDIX B

Regression Techniques

Coefficient of Multiple Determination

The coefficient of multiple determination, denoted by R^2, is defined as follows:

$$R^2 = \frac{SSR}{SSTO} = 1 - \frac{SSE}{SSTO}$$

It measures the proportionate reduction of total variation in Y associated with the use of the set of X variables $X_1, \ldots, X_{p-1}$.

$$0 \leqslant R^2 \leqslant 1$$

R^2 assumes the value 0 when all $b_k = 0 (K = 1, \ldots, p-1)$. R^2 takes on the value 1 when all observations fall directly on the fitted response surface, that is, when $Yi = \hat{Y}i$ for all i.

Inferences About Mean Responses

For given values of $X_1, \ldots, X_{p-1}$, denoted $X_{h1}, \ldots, X_{h,p-1}$, the mean response is denoted E(Y). To estimate this mean response is defined as follows:

$$X_h = \begin{bmatrix} 1 \\ X_{h1} \\ X_{h2} \\ \vdots \\ X_{h,p-1} \end{bmatrix}$$

The estimated mean response corresponding to X_h is denoted $\hat{Y}_h$:

$$\hat{Y}_h = X_h' b$$

This estimator is unbiased:

$$E(\hat{Y}_h) = E(Y_h) = X_h'\beta$$

and its variance is:

$$\sigma^2(\hat{Y}_h) = \sigma^2 X_h'(X'X)^{-1}X_h$$

The estimated variance $S^2(\hat{Y}_h)$ is given by:

$$S^2(\hat{Y}_h) = MSE\ (X_h'(X'X)^{-1}X_h)$$

The 1-α confidence interval for $E(Y_h)$ is:

$$\hat{Y}_h - t(1-\alpha/2;n-p)S(\hat{Y}_h) \leqslant E(Y_h) \leqslant \hat{Y}_h + t(1-\alpha/2;n-p)S(\hat{Y}_h)$$

APPENDIX C

The best set of mix variables obtained from regression analyses for asphalt concrete samples.

Where:

NF	=	number of repetitions to failure
AC	=	asphalt content
VOIDS	=	air void content
#200	=	percent passing No. 200 sieve
#10	=	percent passing No. 10 sieve
AG.T	=	aggregate type used where
AG.T1	=	1 for marginal quality aggregate
	=	0 for good quality aggregate
AG.T2	=	1 for crushed stone
	=	0 for crushed gravel
R^2	=	coefficient of multiple determination
t	=	t-student statistics
n	=	number of samples

Table 1. The Relationship Between Fatigue Life and Percent Passing No. 200 Sieve.

Projects	Strain Level $\times 10^{-6}$ $\mu\varepsilon$	Number of Samples n	R^2	Regression Models
North Oakland	50	10	0.0549	$\log NF = 4.7786 + 0.1255X - 0.00058X^3$
Castle Creek	50	10	0.2236	$\log NF = 3.6437 + 0.3885X - 0.02688X^2$
Warren	50	10	0.3156	$\log NF = 3.9093 + 0.1406X - 0.00027X^3$
All	50	30	0.1153	$\log NF = 4.1703 + 0.1724X - 0.00078X^2$
North Oakland	100	10	0.2199	$\log NF = 3.9938 + 0.1031X - 0.00042X^3$
Castle Creek	100	10	0.2263	$\log NF = 3.5307 + 0.1583X - 0.00095X^3$
Warren	100	10	0.3108	$\log NF = 3.7795 + 0.0429X - 0.00007X^3$
All	100	30	0.1543	$\log NF = 3.6938 + 0.1456X - 0.00863X^2$
North Oakland	125	10	0.3182	$\log NF = 3.9403 + 0.04165X - 0.00009X^3$
Castle Creek	125	10	0.2283	$\log NF = 3.4365 + 0.1283X - 0.00077X^3$
Warren	125	10	0.2787	$\log NF = 3.7554 + 0.0015X^2 - 0.00006X^3$
All	125	30	0.0969	$\log NF = 3.6734 + 0.0849X - 0.00502X^2$

Table 2. The Relationship Between Number of Repetition to Failure and Percent Passing No. 10 Sieve.

Projects	Strain Level x 10^{-6} $\mu\varepsilon$	Number of Samples n	R^2	Regression Models
North Oakland (Fatigue)	50	3	0.7341	log NF = 5.6056 - 0.0258X
	100	3	0.6827	log NF = 4.5129 - 0.00855X
	125	3	0.5644	log NF = 4.1585 - 0.00289X
North Oakland	100	3	0.9806	log NF = 4.4205 + 0.0446 (#10)

Table 3. The Relationship Between Number of Repetitions to Failure and Asphalt Content for Fatigue Criteria.

Projects	Strain Level $\times 10^{-6}$ $\mu\varepsilon$	Number of Samples n	R^2	Regression Models
North Oakland	50	10	0.1589	$\log NF = -5.1304 + 2.5367X - 0.0209X^3$
Castle Creek	50	10	0.3860	$\log NF = -12.8984 + 5.6380X - 0.4398X^2$
Warren	50	10	0.3749	$\log NF = -1.9143 + 2.0533X - 0.1529X^2$
All	50	30	0.2686	$\log NF = -2.6881 + 1.8108X - 0.01395X^3$
North Oakland	100	10	0.1327	$\log NF = -0.1173 + 1.1435X - 0.00997X^3$
Castle Creek	100	10	0.3804	$\log NF = -6.8514 + 3.5035X - 0.2733X^2$
Warren	100	10	0.3561	$\log NF = 1.9253 + 0.6535X - 0.04879X^2$
All	100	30	0.2248	$\log NF = 0.7705 + 0.8131X - 0.00625X^3$
North Oakland	125	10	0.1219	$\log NF = 2.4036 + 0.4229X - 0.00345X^3$
Castle Creek	125	10	0.3773	$\log NF = -4.8842 + 2.8091X - 0.2191X^2$
Warren	125	10	0.1918	$\log NF = 2.9259 + 0.2978X - 0.02476X^2$
All	125	30	0.1821	$\log NF = 1.8164 + 0.6286X - 0.04362X^2$

Table 4. The Relationship Between Number of Repetitions to Failure and Percent Voids for Fatigue Criteria.

Projects	Strain Level $x\ 10^{-6}$ $\mu\varepsilon$	Number of Samples n	R^2	Regression Models
North Oakland	50	10	0.7384	$\log NF = 7.1455 - 0.1911X$
Castle Creek	50	10	0.9606	$\log NF = 6.0252 - 0.008231X^2$
Warren	50	10	0.7720	$\log NF = 5.6046 - 0.1254X$
All	50	30	0.4368	$\log NF = 6.0773 - 0.1188X$
North Oakland	100	10	0.8082	$\log NF = 5.1758 - 0.07973X$
Castle Creek	100	10	0.9583	$\log NF = 4.9118 - 0.00514X^2$
Warren	100	10	0.7605	$\log NF = 4.3154 - 0.0401X$
All	100	30	0.3608	$\log NF = 4.5072 - 0.00295X^2$
North Oakland	125	10	0.8215	$\log NF = 4.5397 - 0.04036X$
Castle Creek	125	10	0.9570	$\log NF = 4.5520 - 0.00415X^2$
Warren	125	10	0.6019	$\log NF = 3.8905 - 0.01203X$
All	125	30	0.2643	$\log NF = 4.1146 - 0.00012X^3$

Table 5. The Number of Repetitions to Failure as a Function of Mix Variables for Fatigue Criteria.

Projects	Strain Level x 10^{-6} $\mu\varepsilon$	Number of Samples n	R^2	Regression Models
North Oakland	50	10	0.7384	log NF = 7.1455 - 0.1911 (VOIDS)
Castle Creek	50	10	0.9436	log NF = 6.8761 - 0.1758 (VOIDS)
Warren	50	10	0.7720	log NF = 5.6046 - 0.1254 (VOIDS)
All	50	30	0.5152	log NF = 4.5546 + 0.2353(AC) - 0.1030(VOIDS)
			0.8283	log NF = 5.8748 - 0.1614(VOIDS)+0.9897(AG.T1) +0.8332(AG.T2)
North Oakland	100	10	0.8997	log NF = 6.0333 - 0.1199(AC) - 0.0943(VOIDS)
Castle Creek	100	10	0.9419	log NF = 5.4439 - 0.1099 (VOIDS)
Warren	100	10	0.7605	log NF = 4.3154 - 0.04010 (VOIDS)
All	100	30	0.3300	log NF = 4.6875 - 0.05103 (VOIDS)
			0.8266	log NF = 4.5732 - 0.0744(VOIDS)+0.5525(AG.T1) +0.4585(AG.T2)
North Oakland	125	10	0.8215	log NF = 4.5397 - 0.04036 (VOIDS)
Castle Creek	125	10	0.9410	log NF = 4.9812 - 0.08862 (VOIDS)
Warren	125	10	0.6019	log NF = 3.8905 - 0.01203 (VOIDS)
All	125	30	0.2708	log NF = 3.6474 + 0.0911(AC) - 0.02187(VOIDS)
			0.7571	log NF = 4.1389 - 0.04511(VOIDS)+0.4457(AG.T1) +0.3372(AG.T2)

AG.T1 = Aggregate Type 1 used in North Oakland
AG.T2 = Aggregate Type 2 used in Castle Creek

Table 6. The Number of Repetitions to Failure as Considering Polynomial Effect of Mix Variables for Fatigue Criteria.

Projects	Strain Level $\times 10^{-6}$ $\mu\varepsilon$	Number of Samples n	R^2	Regression Models
North Oakland	50	10	0.7384	log NF = 7.1455 - 0.1911 (VOIDS)
Castle Creek	50	10	0.9606	log NF = 6.0252 - 0.00823 $(VOIDS)^2$
Warren	50	10	0.7720	log NF = 5.6046 - 0.1254 (VOIDS)
All	50	30	0.5152	log NF = 4.5546 + 0.2353(AC) - 0.1030(VOIDS)
			0.8283	log NF = 5.8748 - 0.1614(VOIDS)+0.9897(AG.T1) +0.8332(AG.T2)
North Oakland	100	10	0.8999	log NF = 5.6699 - 0.0937 (VOIDS) - $0.0099(AC)^2$
Castle Creek	100	10	0.9583	log NF = 4.9118 - 0.00515 $(VOIDS)^2$
Warren	100	10	0.7605	log NF = 4.3154 - 0.0401 (VOIDS)
All	100	30	0.3608	log NF = 4.5072 - 0.00295 $(VOIDS)^2$
			0.8539	log NF = 4.3039+0.5382(AG.T1)+0.4797(AG.T2) $-0.00426(VOIDS)^2$
North Oakland	125	10	0.8215	log NF = 4.5397 - 0.04036 (VOIDS)
Castle Creek	125	10	0.9570	log NF = 4.5521 - 0.00415 $(VOIDS)^2$
Warren	125	10	0.6019	log NF = 3.8905 - 0.01203 (VOIDS)
All	125	30	0.2643	log NF = 4.1146 - 0.00012 $(VOIDS)^3$
			0.8463	log NF = 3.9212+0.4332(AG.T1)+0.3706(AG.T2) $-0.00018(VOIDS)^3$

Table 7. The Number of Repetitions to Failure as Considering Interactions Effects of Mix Variables for Fatigue Criteria.

Projects	Strain Level $\times 10^{-6}$ $\mu\varepsilon$	Number of Samples n	R^2	Regression Models
North Oakland	50	10	0.7384	log NF = 7.1455 - 0.1911 (VOIDS)
Castle Creek	50	10	0.9436	log NF = 6.8761 - 0.1758 (VOIDS)
Warren	50	10	0.9787	log NF =4.2689-0.0089(VOIDS)+0.0295(#200)(AC) -0.0124(#200)(VOIDS)
All	50	30	0.8283	log NF =5.8748-0.1614(VOIDS)+0.9897(AG.T1) +0.8332(AG.T2)
North Oakland	100	10	0.8873	log NF = 5.3093 - 0.01596 (AC)(VOIDS)
Castle Creek	100	10	0.9789	
Warren	100	10	0.9148	log NF =3.8729-0.0011(VOIDS)+0.0098(#200)(AC) -0.0042(#200)(VOIDS)
All	100	30	0.9148	log NF = 4.3801 - 0.0045(VOIDS) + 0.8429(AG.T1) +0.0048(AG.T1)(#200)-0.0466(VOIDS)(AG.T2) -0.0082(AC)(VOIDS) + 0.1569(AC)(AG.T2) -0.0349(VOIDS)(AG.T1)
North Oakland	125	10	0.8363	log NF = 4.5906 - 0.0078 (AC)(VOIDS)
Castle Creek	125	10	0.9410	log NF = 4.9812 - 0.0886 (VOIDS)
Warren	125	10	0.6019	log NF = 3.8905 - 0.01203 (VOIDS)
All	125	30	0.9488	log NF = 3.8905 - 0.01203(VOIDS)+0.5462(AG.T1) +1.0611(AG.T2) + 0.0040(AC)(AG.T2) -0.0244(VOIDS)(AG.T1)-0.0761(VOIDS)(AD.T2)

Table 8. The Number of Repetitions to Failure as Considering All Possible Effects of Mix Variables for Fatigue Criteria.

Projects	Strain Level $\times 10^{-6}$ $\mu\varepsilon$	Number of Samples n	R^2	Regression Models
North Oakland	50	10	0.7384	$\log NF = 7.1455 - 0.1911\ (VOIDS)$
Castle Creek	50	10	0.9606	$\log NF = 6.0252 - 0.00823\ (VOIDS)^2$
Warren	50	10	0.9787	$\log NF = 4.2689 - 0.0090(VOIDS) + 0.0295(\#200)(AC) - 0.0124(\#200)(VOIDS)$
All	50	30	0.8283	$\log NF = 5.8748 - 0.1614(VOIDS) + 0.9897(AG.T1) + 0.8332(AG.T2)$
North Oakland	100	10	0.8873	$\log NF = 5.3093 - 0.01596\ (AC)(VOIDS)$
Castle Creek	100	10	0.9583	$\log NF = 4.9118 - 0.00515\ (VOIDS)^2$
Warren	100	10	0.9789	$\log NF = 3.8729 - 0.0011(VOIDS) + 0.0098(\#200)(AC) - 0.0042(\#200)(VOIDS)$
All	100	30	0.8769	$\log NF = 4.3217 + 1.2297(AG.T1) + 0.4998(AG.T2) - 0.1139(AC)(AG.T1) - 0.0045(VOIDS)^2$
North Oakland	125	10	0.8363	$\log NF = 4.5906 - 0.0078\ (AC)(VOIDS)$
Castle Creek	125	10	0.9570	$\log NF = 4.5521 - 0.00415\ (VOIDS)$
Warren	125	10	0.6019	$\log NF = 3.8905 - 0.01203\ (VOIDS)$
All	125	30	0.9439	$\log NF = 3.8278 + 0.5769(AG.T1) + 1.1183(AG.T2) + 0.0090(\#200)(AG.T1) - 0.0081(AC)(AG.T2) - 0.0271(VOIDS)(AG.T1) - 0.0748(VOIDS)(AG.T2) - 0.00004(VOIDS)^3$

Table 9. The Regression Models of Number of Repetitions to Failure as a Function of Mixture Properties for Permanent Deformation Criteria at 100 Microstrain Level.

Projects	Mix Properties Included	Number of Samples n	t	R^2	Regression Models
North Oakland-Sutherlin	VOIDS	10	3.395	0.8567	log NF = 11.4453 - 1.0158 (VOIDS) + 0.0444 (VOIDS)^2
	AC	10	-0.898	0.1066	log NF = -9.11857 + 3.8739 (AC) - 0.0348 (AC)^3
	#200 sieve	10	-0.994	0.1241	log NF = 5.5069 + 0.0621 (#200)^2 - 0.00579 (#200)^3
	#10 sieve	3	7.114	0.9806	log NF = 4.4205 + 0.0446 (#10)
	#200, AC, VOIDS	10	-3.2150	0.9078	log NF = 14.8476 - 0.1727 (#200) - 0.6530 (AC) - 0.3886 (VOIDS)
	Polynomial	10	3.395	0.8567	log NF = 11.4453 - 1.0158 (VOIDS) + 0.0444 (VOIDS)^2
	Interactions	10	-3.763	0.9161	log NF = 10.9743 - 0.1798 (#200) - 0.06612 (AC)(VOIDS)
	All	10	-4.270	0.9296	log NF = 10.3995 - 0.0641 (AC)(VOIDS) - 0.0145 (#200)^2
Castle Rock-Cedar Creek	VOIDS	10	3.516	0.9283	log NF = 16.1979 - 1.5508 (VOIDS) + 0.0534 (VOIDS)^2
	AC	10	-1.644	0.3130	log NF = -32.5274 + 9.5914 (AC)^2 - 0.0843 (AC)^3
	#200 sieve	10	-1.192	0.1769	log NF = 4.8149 + 0.1050 (#200)^2 = 0.0101 (#200)^3
	#200, AC, VOIDS	10	-3.537	0.9572	log NF = 17.1230 - 0.1797 (#200) - 0.6583 (AC) - 0.5326 (VOIDS)
	Polynomial	10	2.5060	0.9979	log NF = 14.7903 0 1.0286(VOIDS)+0.0267(VOIDS)^2-0.0029(AC)^3+0.0185(#200)^2 -0.0028(#200)^3
	Interactions	10	-2.426	0.9484	log NF = 15.9832 - 0.4848 (AC) = 0.5299 (VOIDS) - 0.0281 (#200)(AC)
	All	10	2.506	0.9979	log NF = 14.7903 - 1.0286(VOIDS)+0.2674(VOIDS)^2-0.0029(AC)^3+0.1848(#200)^2 -0.0028(#200)^3
Warren-Scappoose	VOIDS	10	2.579	0.8528	log NF = 8.3018 - 0.5205 (VOIDS) + 0.0249 (VOIDS)^2
	AC	10	-1.019	0.1357	log NF = -9.2822 + 5.6213 (AC) = 0.5039 (AC)^2
	#200 sieve	10	-0.805	0.1838	log NF = 4.995 + 0.2563 (#200) - 0.00134 (#200)^3
	#200, AC, VOIDS	10	-6.497	0.8588	log NF = 9.8565 + 0.3916 (AC) - 0.2150 (VOIDS)
	Polynomial	10	2.368	0.9297	log NF = 8.8501 + 0.4571(VOIDS) + 0.1818 (VOIDS)^2 - 0.0032(AC)^3
	Interactions	10	-6.786	0.8520	log NF = 7.6178 - 0.0383 (AC)(VOIDS)
	All	10	-6.786	0.8520	log NF = 7.6178 - 0.0383 (AC)(VOIDS)
All	VOIDS	30	-5.856	0.5505	log NF = 8.1935 - 0.2226 (VOIDS)
	AC	30	1.589	0.0998	log NF = -6.4484 + 3.2086 (AC) - 0.0295 (AC)^3
	#200 sieve	30	-1.929	0.1288	log NF = 4.4149 + 0.6149 (#200) - 0.0479 (#200)^2
	#200, AC, VOIDS	30	-5.856	0.5505	log NF = 8.1935 - 0.2226 (VOIDS)
	#200, AC, VOIDS, AG.T	30	-5.856	0.5505	log NF = 8.1935 - 0.2226 (VOIDS)
	Polynomial	30	-5.856	0.5505	log NF = 8.1935 - 0.2226 (VOIDS)
	Poly + AG.T	30	-4.272	0.8470	log NF = 12.2367-0.5534(AC)-0.3665(VOIDS)+1.1208(AG.T1)+1.6032(AG.T2) -0.00106(#200)^3
	Interactions	30	-5.856	0.5505	log NF = 8.1935 - 0.2226 (VOIDS)
	All Effects	30	-5.856	0.5505	log NF = 8.1935 - 0.2226 (VOIDS)

Table 10. The Regression Models of Number of Repetitions to Failure as a Function of Mixture Properties for Fatigue Criteria at 100 Microstrain Level.

Projects	Mix Properties Included	Number of Samples n	t	R^2	Regression Models
North Oakland-Sutherlin	VOIDS	10	-5.805	0.8082	$\log NF = 5.1758 - 0.07973\ (VOIDS)$
	AC	10	-0.925	0.1327	$\log NF = -0.1173 + 1.1435\ (AC) - 0.00997\ (AC)^3$
	#200 sieve	10	-0.687	0.2199	$\log NF = 3.9938 + 0.1031\ (\#200) - 0.00042\ (\#200)^3$
	#10 sieve	3	-1.467	0.6827	$\log NF = 4.5129 - 0.00855\ (\#10)$
	#200, AC, VOIDS	10	-2.528	0.8997	$\log NF = 6.0333 - 0.1199(AC) - 0.0943\ (VOIDS)$
	Polynomial	10	-2.533	0.8999	$\log NF = 5.6699 - 0.0937\ (VOIDS) - 0.0099\ (AC)^2$
	Interactions	10	-7.935	0.8873	$\log NF = 5.3093 - 0.01596\ (AC)(VOIDS)$
	All Effects	10	-7.935	0.8873	$\log NF = 5.3093 - 0.01596\ (AC)(VOIDS)$
Castle Rock-Cedar Creek	VOIDS	10	-13.562	0.9583	$\log NF = 4.9118 - 0.00514\ (VOIDS)^2$
	AC	10	-1.266	0.3804	$\log NF = -6.8514 + 3.5035\ (AC) - 0.2733\ (AC)^2$
	#200 sieve	10	-1.128	0.2263	$\log NF = 3.5307 + 0.1583\ (\#200) - 0.00094\ (\#200)^3$
	#200, AC, VOIDS	10	-11.385	0.9419	$\log NF = 5.4439 - 0.1099\ (VOIDS)$
	Polynomial	10	-13.652	0.9583	$\log NF = 4.9118 - 0.00515\ (VOIDS)^2$
	Interaction	10	-11.385	0.9419	$\log NF = 5.4439 - 0.1099\ (VOIDS)$
	All Effects	10	-13.562	0.9583	$\log NF = 4.9118 - 0.00515\ (VOIDS)^2$
Warren-Scappoose	VOIDS	10	-5.041	0.7605	$\log NF = 4.3154 - 0.0401\ (VOIDS)$
	AC	10	-0.503	0.3561	$\log NF = 1.9253 + 0.6535\ (AC) - 0.04879\ (AC)^2$
	#200 sieve	10	-0.201	0.3108	$\log NF = 3.7795 + 0.0429\ (\#200) - 0.00007\ (\#200)^3$
	#200, AC, VOIDS	10	-5.041	0.7605	$\log NF = 4.3154 - 0.04010\ (VOIDS)$
	Polynomial	10	-5.041	0.7605	$\log NF = 4.3154 - 0.04010\ (VOIDS)$
	Interactions	10	-5.620	0.9789	$\log NF = 3.8729 - 0.0011\ (VOIDS) + 0.0098\ (\#200)(AC) - 0.0042\ (\#200)(VOIDS)$
	All Effects	10	-5.620	0.9789	$\log NF = 3.8729 - 0.0011\ (VOIDS) + 0.0098\ (\#200)(AC) - 0.0042\ (\#200)(VOIDS)$
All	VOIDS	30	-3.975	0.3608	$\log NF = 4.5072 - 0.00295\ (VOIDS)^2$
	AC	30	-1.225	0.2248	$\log NF = 0.7705 + 0.8131\ (AC) - 0.00625\ (AC)^3$
	#200 sieve	30	-1.191	0.1543	$\log NF = 3.6938 + 0.1456\ (\#200) - 0.00863\ (\#200)^2$
	#200, AC, VOIDS	30	-3.714	0.3300	$\log NF = 4.6875 - 0.05103\ (VOIDS)$
	#200, AC, VOIDS, AG.T	30	6.329	0.8266	$\log NF = 4.5732 - 0.0744\ (VOIDS) + 0.5525\ (AG.T1) + 0.4585\ (AG.T2)$
	Polynomial	30	-3.975	0.3608	$\log NF = 4.5072 - 0.00295\ (VOIDS)^2$
	Poly + AG.T	30	7.147	0.8539	$\log NF = 4.3039 + 0.5382\ (AG.T1) + 0.4797\ (AG.T2) - 0.00426\ (VOIDS)^2$
	Interactions	30	-2.162	0.9148	$\log NF = 4.3801 - 0.0045\ (VOIDS) + 0.8429\ (AG.T1) + 0.0048\ (AG.T2)(\#200)$
	All Effects	30	-2.162	0.8769	$\log NF = 4.3217 + 1.2297(AG.T1) + 0.4998(AG.T2) - 0.1139(AC)(AG.T1) - 0.0045(VOIDS)^2$