EFFECTIVENESS OF ANTISTRIPPING ADDITIVES

VOLUME I

by

John W. Shute
Graduate Research Assistant
Oregon State University
Corvallis, OR 97331

R.G. Hicks
Professor of Civil Engineering
Oregon State University
Corvallis, OR 97331

James E. Wilson
Research Specialist
Oregon Department of Transportation
Salem, OR 97310

and

Lewis G. Scholl
Technical Studies Coordinator
Oregon Department of Transportation
Salem, OR 97310

May 1989

Technical Report Documentation Page

1. Report No. FHWA-OR-RD-89-03A	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle EFFECTIVENESS OF ANTISTRIPPING ADDITIVES Volume I		5. Report Date May 1989
		6. Performing Organization Code
7. Author(s) John W. Shute, et al.		8. Performing Organization Report No.
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 No. 5275
12. Sponsoring Agency Name and Address Materials and Research Division Oregon Department of Transportation Salem, OR 97310		13. Type of Report and Period Covered Feb. 1988 - May 1988
		14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

Volume I of this report summarizes the findings on the extent of stripping in 32 Oregon DOT projects and discusses the significant findings from a literature review. Of the 32 projects, 10 had not used an asphalt or aggregate antistripping additive, 19 contained lime-treated aggregate, and 3 contained asphalt treated with an amine-type antistripping additive. These projects range in age from 1 to 9 years. The percent of aggregate coated with asphalt, which was used to determine if a project was stripping, was established from breaking and visually evaluating core samples taken from each project site. Using the criteria that a coating less than 85% classified a project as stripping, the following results were obtained: 7 of the 10 projects without an additive, 5 of the 19 lime-treated aggregate projects, and 2 of the 3 amine-treated asphalt projects were identified as stripping. The conclusion is that lime treatment of aggregate has proven effective in reducing the moisture susceptibility of an asphalt mixture. In addition, a review of mix design test values for the index of retained strength (IRS) (AASHTO T-165) and resilient modulus (M_r) test indicates the IRS test does not reliably predict moisture susceptible mixtures. The M_r test did provide a better indication of moisture susceptible mixtures. This assessment is based on a limited amount of data and is not statistically based; therefore, the results may change with further information. Volume II presents the complete literature review.

17. Key Words Asphalt concrete, stripping, chemical additives, lime, moisture	18. Distribution Statement This document is available to the public through the National Technical Information Service, Springfield, VA 22161.		
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages	22. Price

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

ACKNOWLEDGEMENTS

The work presented in this report was conducted as part of a Highway Planning and Research (HP&R) Project funded through the U.S. Department of Transportation Federal Highway Administration (FHWA), and Oregon Department of Transportation (ODOT). The authors wish to thank the FHWA for their continued support of this and previous studies.

The authors would like to extend their appreciation to Glenn Boyle and Bud VanCleave of the Oregon Department of Transportation for their assistance. A special thanks is due also to the numerous field and laboratory personnel from the Oregon Department of Transportation for producing the field core data and to Laurie Dockendorf from the Engineering Experiment Station at Oregon State University for typing this document.

DISCLAIMER

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

1.1 Problem Statement

Stripping, the loss of adhesion between the asphalt and aggregate due to moisture, has received considerable attention in recent years from agencies throughout the United States including the Oregon Department of Transportation (ODOT). An example is a recent joint study by Oregon State University and Oregon DOT (1985) entitled, "Identification and Quantification of the Extent of Asphalt Stripping in Flexible Pavements in Oregon." A recommendation from this project was for the identification of the short- and long-term benefits of using antistripping additives. It was this lack of information that was the basis for this project.

Initiated in February 1988, this project consisted of a literature review and an evaluation on the presence of stripping in 32 project sites. Initially, 20 projects were to be selected for coring analysis. A pavement condition survey was performed at each project site to be used in selecting the 20 projects for coring. However, after completion of the pavement condition survey, it was determined by ODOT that enough funds were available to core all 32 project sites. In addition, a proposed Phase II was canceled and efforts were redirected into evaluating the data obtained from the coring, an analysis of the mix design test values, and the effect of stockpiling lime-treated aggregate.

1.2 Objectives

The project had three specific objectives;

1) conduct a literature review on the effectiveness of antistripping additives and moisture sensitivity testing procedures,
2) evaluate the effectiveness of lime or chemicals in controlling stripping on recent projects constructed in Oregon, and
3) evaluate the correlation of mix design test results with the field performance of the mixture.

1.3 Background

Experience in Oregon over the past 35 years has seen the need for the use of antistripping additives change from almost none prior to 1974 to 16, 20, and 26% of paving in 1975, 1976, and 1977, respectively. The need for the use of antistripping additives had became apparent. The index of retained strength (IRS test AASHTO T-165) at a criteria of 70% minimum was used to determine moisture susceptible mixes. Design mixtures with an IRS less than the 70% minimum were redesigned with the addition of an amine-type antistripping additive and/or a change in asphalt grade or source. In 1980, the ODOT laboratory developed the use of a 0.70 modulus ratio minimum as a design requirement on mixtures.

In 1982, a study (Hicks et al., 1983) was undertaken by ODOT for the Federal Highway Administration. The purpose of this study was to evaluate the effect of material sources, void content, and additive type on the Index of Retained Strength (IRS) or retained resilient modulus (IRM) after freeze-thaw conditioning. Furthermore, the study objectives were to identify and quantify the extent of asphalt stripping in flexible pavements. To satisfy the objectives of the study, a total of 20 projects were evaluated. The results of this study indicated that (1) present mix design procedures may not always detect problems from asphalt-aggregate stripping; (2) aggregate quality and asphalt sources appear to relate to low values for IRS and modulus ratio; (3) significant differences existed for IRS and modulus ratio values for construction mix design specimens, submitted mix specimens, and laboratory batched specimens; (4) level of compaction greatly affected the compressive strength; however, the IRS values show little change; (5) freeze-thaw conditioning greatly affected modulus and modulus ratios; and (6) the use of additives generally increases both the modulus ratio and IRS.

The study led in part to the following specification changes in 1984 to determine when to use antistripping additives to reduce moisture susceptibility in asphalt concrete mixtures:

1) mandatory use of lime-treated aggregate when project elevation is above 2500 ft,
2) projects are in problem pavement areas,
3) increasing the IRS from 70% to 75%, and
4) require the use of lime-treated aggregate when mix design testing has an IRS less than 80%.

As of April 1987, the above criteria were amended by ODOT to include mandatory lime treatment of aggregates when specific criteria are met as outlined in Chapter 3.

1.4 Study Approach

The study approach for this report entailed the following:

1) Conducting a literature review (Volume II) on stripping, the effectiveness of antistripping additives, and moisture sensitivity test procedures.
2) A review of methods currently employed by Oregon DOT concerning moisture sensitivity test methods and the use of antistripping additives.
3) An analysis of the pavement condition for the 32 projects selected. This analysis, in addition to the pavement condition survey and core data, included a review of the correlation between the results from ODOT's moisture sensitivity tests and the field performance of the asphalt concrete. Also, an evaluation of the effect of stockpiling lime-treated aggregate and a statistical analysis of the data were conducted.

2.0 EVALUATION OF LITERATURE

Stripping is defined as the physical separation of the asphalt from the aggregate. This loss of adhesion is caused by the action of water and/or water vapor and occurs because the aggregate has a greater attraction to water than asphalt. The rate at which stripping occurs is dependent upon factors such as the physical or chemical properties of the asphalt, the type or mineral composition of aggregate, and the exposure conditions of the asphalt pavement in service. Indications of stripping are surface flushing (or bleeding), raveling, random cracking, and potholes. Stripping leads to premature failure of the asphalt concrete pavement structure.

Failure within the asphalt concrete due to the presence of moisture can occur in two ways. One is due to a loss of adhesion between the asphalt and the aggregate. This type of failure is associated with stripping. Additives, such as lime and liquid antistripping agents, may change the chemical properties at the asphalt/aggregate interface and therefore improve the adhesive bond between the asphalt and aggregate. The second type of failure is due to a weakening of the cohesive strength of the asphalt. This process will not be addressed to any great extent in this report.

This chapter presents a summary of the literature review on (1) theories of adhesion, (2) factors influencing stripping, and (3) preventive treatments against stripping.

2.1 Theories of Adhesion

The loss of adhesion between the asphalt and aggregate surface has been explained by several theories. These include the Chemical Reaction Theory, Molecular Orientation Theory, Mechanical Theory, and Surface Energy Theory. Stripping cannot be fully described based on just one theory. Each theory has some contribution to the explanation of why moisture can break the adhesion between an asphalt and aggregate; however, a precise definition of one theory's significance over another theory has not clearly been established. Each of the theories are summarized below:

1) **Chemical Reaction Theory.** The chemical composition of the aggregate plays an important role. Basic aggregates, such as limestone, are less likely to strip than "acidic" rocks, such as sandstone (Mertson and Wright, 1959). The chemical reaction is not complete between acidic aggregate and asphalt since there are less receptive sites for the carboxylic acids in the asphalt to bond. In other words, there are less electrically positive sites on the aggregate to receive the negative components of the asphalt.

2) **Molecular Orientation Theory.** Contact between the aggregate and asphalt molecules can depend on their molecular orientation. Asphalt molecules orient themselves to satisfy energy demands of the aggregate (Mack, 1935). However, the number of receptive sites on the aggregate influences the overall strength of the bond.

3) **Mechanical Theory.** Adhesion is affected by surface texture, porosity, surface coating, surface area, and particle size. The rougher the surface texture, the greater the bond strength between the asphalt and aggregate. The greater the porosity (the pores being of sufficient size to allow the asphalt penetration), the greater the mechanical interlock (Zeisman, 1963). Dust blocks pore openings by forming small dams across the pore openings preventing penetration of asphalt into the pores (Held, 1986). This reduces the contact surface area between the asphalt and aggregate. The more fines in the mixture, the greater the surface area, and therefore a greater amount of asphalt is required to coat all the aggregates sufficiently.

4) **Surface Energy Theory.** The wetting power of a liquid indicates the ability of a liquid to coat or migrate across a surface (Hubbard, 1958). Water has a greater wetting power than asphalt because of its low viscosity. Water also has a greater adhesion tension than asphalt (Rice, 1958). Therefore, it will tend to displace asphalt from an aggregate surface.

The theories and mechanisms that have been reviewed are not satisfactory to completely explain stripping under all environmental conditions. The adhesion of asphalt cement to an aggregate is obviously a very complex phenomenon. The physio-chemical forces and reactions that exist and the interactions between the asphalt cement, aggregate, water, and air all relate to stripping. The adhesion of an asphalt cement to an aggregate surface may be controlled to a greater degree by the characteristics of the aggregate surface rather than the characteristics or chemistry of the asphalt cement. This is apparently why more agencies are leaning toward the use of lime (which changes the aggregate surface characteristics) rather than liquid chemicals (which affect the chemistry or wettability of the asphalt itself). Lime can affect the properties of asphalt or aggregate and in some cases affects both.

2.2 Factors Influencing Stripping

The factors that influence stripping may be discussed under three broad headings: (1) asphalt concrete characteristics, including aggregate, asphalt cement, and mix characteristics, (2) environmental considerations, including weather and traffic; and (3) construction practices. Each of these factors is discussed below.

2.2.1 Asphalt Concrete Characteristics

The asphalt concrete characteristics important to the stripping mechanisms include the nature of the aggregate, the nature of the asphalt cement, and the type of mix (Table 2.1). The aggregate characteristics identified as being important to stripping include the surface texture, porosity, mineralogy, surface moisture, surface coatings, and surface chemical composition. The asphalt cement viscosity is generally believed to be an important characteristic to consider; however, aggregate characteristics are the more important of the two. Finally, stripping is believed to occur more easily in open-graded mixes compared to dense graded mixes.

Aggregate Characteristics. Aggregates are composed of minerals. Each mineral has a characteristic chemical composition and crystalline structure. Rock types are identified based upon the mineralogical composition and the formation processes associated with the rock. More important to the stripping mechanism is the classification of an aggregate based upon its affinity for

Table 2.1. Summary of Factors Influencing Stripping.

Factor	Desirable Characteristics
1) Aggregate	
a) Surface Texture	Rough
b) Porosity	*
c) Mineralogy	*
d) Coatings	Clean
e) Surface Moisture	Low Moisture
f) Surface Chemical Composition	*
2) Asphalt Cement	
a) Viscosity	High
b) Surface Chemistry	*
c) Composition	*
3) Asphalt Concrete Mix	
a) Voids	Low
b) Gradation	Dense
4) Weather Conditions	
a) Temperature	Warm
b) Rainfall During Construction	None
c) Rainfall Following Construction	Minimal
d) Freeze-Thaw Following Construction	Minimal

*No consensus of opinion

water. Aggregates that are hydrophilic have a greater affinity (enhanced attraction) for water compared to asphalt cement. Aggregates that are hydrophobic have a greater attraction for asphalt cement than water. In general, it may be stated that hydrophilic aggregates are acidic and have a high silica content; hydrophobic aggregates are generally basic and have low silica contents (Majidzadeh and Brovold, 1968). Limestone and other carbonaceous rocks are generally categorized as hydrophobic aggregates. Hydrophobic aggregates, of course, are believed to provide greater resistance to stripping than hydrophilic aggregates. However, an acidic quartzite has been found to be less susceptible to stripping than most basic aggregates (Majidzadeh and Brovold, 1968), whereas a limestone aggregate mix was recently observed to strip (Maupin, 1983). Conglomerate-type aggregate containing minerals of clay compounds which cause degradation are more susceptible to stripping than aggregates of more uniform composition.

Asphalt Characteristics. It is not possible to make a general statement as to which asphalt characteristics are most important to stripping. However, most investigators have identified the fact that high viscosity asphalt cements resist displacement by water to a greater degree than low viscosity asphalt cements. Unfortunately, high viscosity asphalt cements do not have the wetting power of low viscosity cements. Schmidt and Graf (1972) note that if asphalt cements have the same viscosity, the chemical composition of the asphalt appears to have negligible effect on stripping. Others report that asphalt chemistry can be an important factor influencing stripping. Compounds contained in asphalts, such as carboxylic acids and certain sodium compounds have been found to be more susceptible to stripping.

Type of Mix. Brown et al. (1959) indicate that dense-graded hot mixes should not strip unless there are excessive air voids, moisture, insufficient asphalt cement, inadequate compaction, or unless the aggregates have absorbed coatings. The primary benefit in a hot mix, however, may be the drying of the aggregates in those mixtures (Thelen, 1958). Good resistance to stripping in open-graded cold mix paving mixtures has been observed in the state of Oregon (Takallou et al., 1985). This resistance may, however, be due to the anti-stripping agents which are contained in the emulsions used for these mixes.

2.2.2 Weather During Construction

Weather conditions during construction of an asphalt concrete pavement have a pronounced influence on the susceptibility of the pavement to stripping. If the weather is cool and wet during construction, stripping of the pavement is more likely to occur; generally speaking, these conditions are prevalent in the late fall. Improper or poor compaction is another construction practice that can lead to stripping because the high air void content that allows water penetration into the asphalt mixture.

2.2.3 Environmental Effects After Construction

Environmental considerations that affect stripping include both climatic and traffic loadings. Hindermann (1968) notes that temperature fluctuations, freeze-thaw cycles, and wet-dry cycles can all affect pavement stripping. Obviously, water is at the root of any stripping problem in an asphalt concrete mix. At least two mechanisms of stripping (pore pressure and hydraulic scouring) are associated with damage due to cyclic loading of the asphalt concrete by traffic. Consequently, it may be stated that, all other factors being equal, increased traffic loading (in terms of number of cycles) would accelerate stripping.

2.3 Corrective Treatments for Stripping

Several control measures may be employed in an attempt to minimize stripping. These include the use of good aggregate, the use of pavement surface sealants, pretreatment of aggregates, and the use of antistripping agents. These control measures are briefly discussed in the following paragraphs. The use of lime is discussed in more detail in a later section.

2.3.1 General

In general, to minimize stripping, aggregates should be chosen which have porosities of approximately 0.5% and a rough, clean surface (Krebs and Walker, 1971). Rounded aggregates should be crushed to produce a rougher texture and coated aggregates should be cleaned through initial processing.

The entrance of water into a pavement structure can be substantially reduced by closing the surface pores. To accomplish this, a variety of pavement surface sealants have been applied to asphalt concrete surfaces. Perhaps the most common is "fog sealing" of a pavement structure. This technique consists of spraying a light application of a liquid asphalt (typically an asphalt emulsion) without mineral aggregate filler to the pavement surface. Other commercially available sealants are also used.

Pretreatment of aggregates involves modifying the surface properties of the aggregate prior to construction. In general, the pretreatment techniques seek to replace the aggregate surface ions that are likely to be removed by water or to cause weak bonding with the asphalt. They also seek to promote a strong bond between the asphalt cement and the aggregate surface. Certainly the most common pretreatment material employed is hydrated lime. The lime can be applied using either a wet or dry process. It is generally believed that the lime produces a sharp decrease in the interfacial tension between the asphalt cement and water, thus resulting in good adhesion. Portland cement and flyash have also proven to be fairly effective in pretreatment applications. Plancher et al. (1977) suggest that hydrated lime improves stripping resistance as a result of the interaction between the lime and acids in the asphalt that are readily adsorbed onto the surface of an aggregate. Schmidt and Graf (1972) note that the mechanism associated with hydrated lime improving stripping resistance cannot be completely explained by the reaction of the asphaltic acids with the lime. They note that lime, in general, provides calcium ions which can replace hydrogen, sodium, potassium, and other cations on the aggregate surface.

The majority of chemical antistripping additives are surface active agents which reduce the surface tension of the asphalt cement and, therefore, promote greater adhesion to the aggregate. When chemical antistripping agents are added to asphalt concrete the amount is usually 0.5–1.0% by weight of asphalt cement. The improved asphalt cement/aggregate adhesion is associated with the fact that the antistripping agents give the asphalt cement an electrical charge that is opposite to that of the aggregate surface. The properties of asphalt cements containing antistripping agents can vary greatly.

2.3.2 Lime

Based on the review of available literature, lime appears to act on the aggregate surface in the following manner. With most siliceous aggregate, it forms a calcium hydroxy silicate crust on the surface of the aggregate. This crust, which forms a strong bond to the aggregate, has sufficient porosity to allow penetration of the asphalt. The carboxylic acids and 2-quinolenes components of the asphalt are then absorbed by the lime and form an insoluble calcium salt. In siliceous aggregate there exists acidic (SiOH) groups on the surface. These groups form hydrogen bonds with the carboxylic groups from the asphalt and play a major role in the adhesion between the asphalt and aggregate. However, in the presence of water, the two groups (SiOH and carboxylic acids) dissociate and each associates with water molecules forming strong hydrogen bonds. If the lime forms calcium salts with the carboxylic and 2-quinolenes, then the SiOH groups must bond with another molecule. Petersen et al. (1987) have proposed that the bond is with nitrogen groups in the asphalt (which are basic) and these groups form strong bonds which promote adhesion.

Addition of Lime. The manner by which lime can be added varies considerably in practice. Four ways to add lime and some features about each are discussed below (Kennedy, 1984):

1) **Dry Lime.** Adding dry lime to dry aggregate is the least effective method. A major problem with this method is retaining the lime on the aggregate surface. In a batch plant, the lime can be added in a couple of places. If it is added in the weigh box or pugmill, the amount of loss due to dust is minimized and the lime can be mixed with the aggregates prior to adding the asphalt. If it is added prior to this point, the chance of lime loss is increased.

   In drum mix plants, the lime can be added to aggregate on the plant cold feed aggregate or with the asphalt. The loss of lime is minimized when baffles are utilized inside the drum mixer.

2) **Hydrated Lime Slurry.** Lime can also be introduced using a slurry mixture of lime and water. The slurry is generally added on the cold feed or in a premixing pugmill in a ratio of 3:1 by weight of water to lime. The major disadvantage of this procedure is the removal of the moisture and the energy costs necessary to accomplish this task. An advantage is that the slurry will adhere to the aggregate surface upon contact.

3) **Dry Lime with Water.** This method involves adding dry lime to pre-moistened aggregate. In a drum plant, hydrated lime may be added to moist aggregate at the dryer headchute or the conveyor slinger. With this method, when the lime is added there will be some loss of lime to the atmosphere. A major advantage is that only the lime needs to be controlled. The required moisture level of the aggregates is at or just above saturated surface dry condition.

   In a batch plant the lime can be added in a couple of locations. A positive premixing pugmill or a tumble mixer may be used to obtain a homogeneous mixture.

4) **Hot (Quick Lime) Slurry.** A slurry mixture of Quick Lime (CaO) and water can also be used. The slurry is introduced in the same manner as the lime slurry mentioned earlier. The major drawback to this procedure is the volatile nature of the calcium oxide (CaO). An added benefit is the heat generated during the slaking process helps drive off moisture and the yield is approximately 25% greater (Kennedy, 1984).

Discussion. Stockpiling of the lime-treated aggregate can have mixed results. An advantage is eliminating the coordination of the lime treatment of aggregate with asphalt mixture production. Weather permitting, stockpiling also allows for the evaporation of moisture which would help reduce fuel costs. In addition, if the fines are highly plastic, stockpiling allows the lime to reduce the plasticity of the fines. However, if the lime is stock-piled too long, carbonation may occur which could render the lime ineffective. The rate of carbonation depends on climatic conditions.

The introduction of lime onto the aggregate requires the presence of moisture. When hydrated lime contacts the water the reaction is considered instantaneous. The addition of hydrated lime onto aggregates without surface moisture has not consistently provided the desired benefits; the problem is with retaining the lime on the aggregate surface. It appears as if the addition of hydrated lime onto moist aggregates provides the desired results in most cases while introducing the least amount of moisture.

Tunnicliff and Root (1986), in addition to studying the different methods of adding lime, found that mixtures were not highly sensitive to lime loss during construction. Also, they are of the opinion that the location of lime introduction, unlike chemical additives, does not play a significant role in the mixture performance. Therefore, adding the lime to the aggregate at different locations, e.g., a lime slurry on the cold feed or in a premixing pugmill, did not significantly change the effectiveness of the lime treatment.

The following list summarizes some key points concerning the use of lime as an antistripping additive:

1) Lime is not sensitive to asphalt or aggregate types,
2) Various methods of lime addition allow flexibility in the type of treatment for the aggregate,
3) All methods of addition require moisture to activate lime,
4) Additional energy is required to remove the additional moisture,
5) Application rates are approximately 1 to 2% by weight of aggregate, and
6) Lime addition significantly increases asphalt concrete mix costs, generally in the range of 3 to 10%.

2.3.3 Chemical Additives

Chemical additives commonly used to prevent stripping are cationic surfactants which change the surface charge of the asphalt to improve the compatibility of the asphalt and aggregate. The effectiveness of chemical additives is dependent upon the specific combination of additive, asphalt, and aggregate. Any change in either of the variables could cause a significant change in the moisture susceptibility of the mixture. If a chemical additive is not "heat stable," prolonged exposure to a high temperature during the construction process can render the additive ineffective even though lab results indicate the additive will reduce the moisture susceptibility of an asphalt concrete mix.

When using cationic surfactants, the non-polar end of the hydrocarbon attaches to the asphalt while the amine group forms ammonium salts with the hydrogen ions in the aggregate. These cationic surfactants also encourage the formation of emulsified asphalt which may form at the asphalt/aggregate interface due to the presence of water and traffic forces.

The effectiveness of chemical additives can depend significantly on how they are applied. When the additive is mixed with the asphalt cement, the chemicals migrate through the asphalt to the asphalt/aggregate interface. When the mix is still hot and the viscosity of the asphalt cement is low the amount of chemicals able to migrate is great. However, when the asphalt cools and the viscosity increases, the rate of migration decreases. The normal time of 3 hours for migration is not always enough for a high percentage of liquid antistripping agent to reach the asphalt/aggregate interface. A time of 12 hours is generally needed to allow a sufficient amount of chemical to reach the interface.

Additional methods have been studied to increase the adhesive bond between the asphalt and aggregate. Al-Ohaly and Terrel (1988) studied the effects of microwaves on asphalt mixes. They found the use of microwave treatment can enhance the resistance to stripping of an asphalt mixture by: 1) decreasing the viscosity of the asphalt and allowing it to redeposit in impermeable voids, 2) increasing the migration of chemical additives to the asphalt/aggregate interface by forced polarization, and/or 3) reducing the random orientation of the molecules at the interface. These mechanisms are illustrated in Figure 2.1.

One difference between chemical additives and lime is their reaction to an increase in the pH of the surrounding water. Chemical additives were found to be susceptible to the increase in pH of the surrounding water. Two liquid antistripping agents showed a decrease in their effectiveness when the pH was increased while the effectiveness of the lime remained unchanged (Yoon and Tarrer, 1988). The study also showed that storing the mix at 300°F for a few hours increases the effectiveness of the chemical additives even with an increase in the pH of the water. This was attributed to the formation of a polymerized asphalt on the aggregate surface (Held, 1986). Also, to maintain the asphalt at a high temperature for a longer period of time would enhance the migration of a liquid antistripping agent to the asphalt/aggregate interface.

The following list summarizes some key points on the use of chemical additives as an antistripping additive:

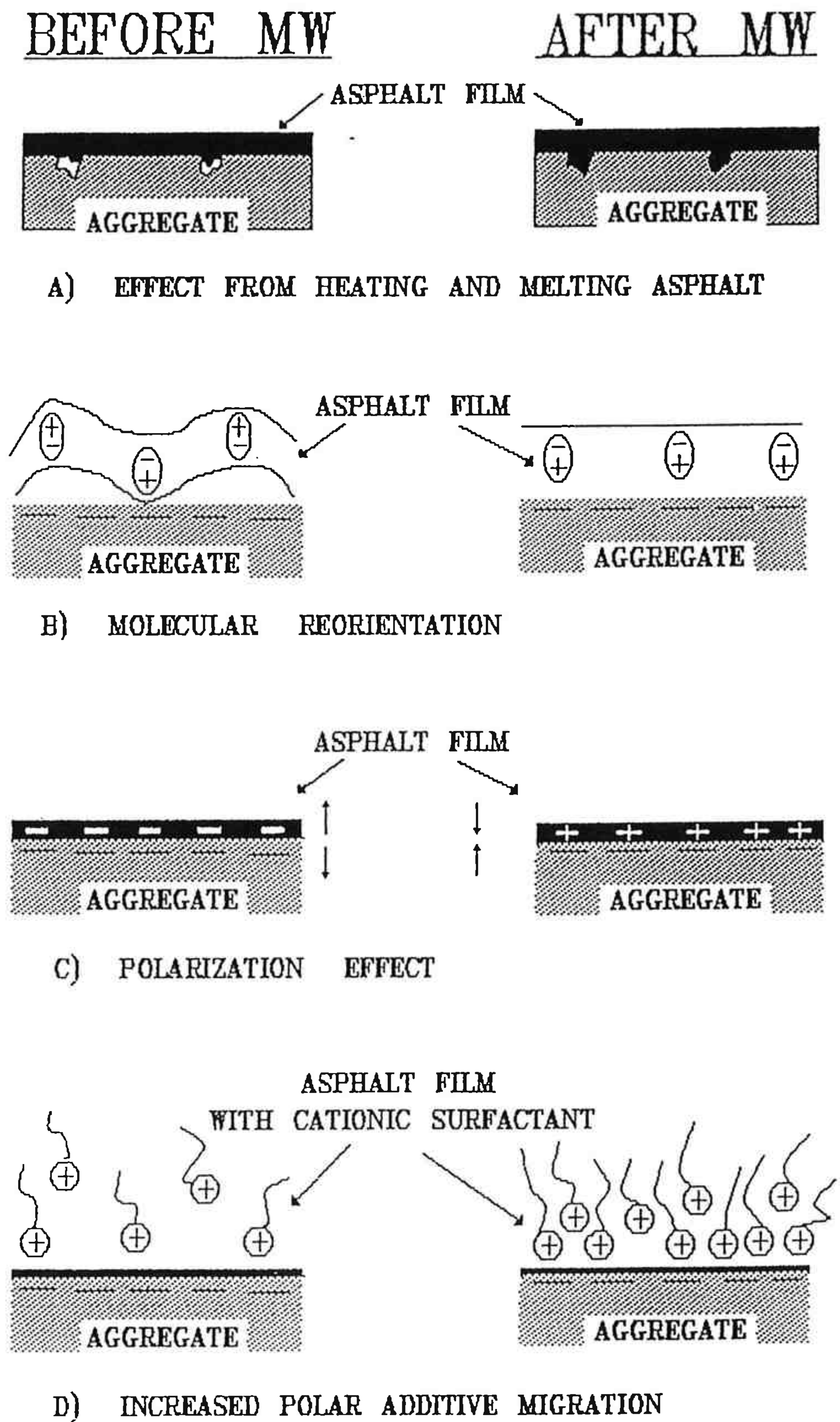


Figure 2.1. Mechanisms of Asphalt Adhesion Improvement with Microwave Energy Treatment (after Al-Ohaly and Terrel, 1988).

1) Low cost
2) Easy to blend with asphalt
3) Added directly to the asphalt prior to mixing with aggregate
4) Sensitive to asphalt and aggregate types
5) May proportionally increase the strengths of unconditioned samples more than conditioned samples, thereby decreasing ratios even though conditioned strengths are higher
6) Application rate of approximately 0.5 to 1.0% by weight of asphalt.

2.4 Testing Procedures

A review of the literature indicates that the following five test procedures have received the most attention and acceptance during laboratory studies. They are:

1) NCHRP 246 Indirect Tension Test and/or Modulus Test with Lottman Conditioning
2) NCHRP 274 Indirect Tensile Test with Tunnicliff and Root Conditioning
3) Boiling Water Tests
4) Texas Freeze-Thaw Pedestal Test
5) Immersion Compression Tests

An outline of the test procedures along with some advantages and disadvantages of each procedure is listed in Tables 2.2 to 2.6. The conditioning procedures vary for each test. A test that has produced 100% correlation to field data has not yet been produced. Coplantz (1987) has shown that conditioning by saturation alone may not be severe enough to induce moisture damage over a short period of time. Moisture damage will occur in a mixture from saturation over a period of time. Busching et al. (1986) found damage from saturation after 60 days. Also, while at least one freeze-thaw cycle appears to be needed to simulate moisture damage, it may not be enough to determine long term performance. The use of approximately 5 cycles of freeze-thaw may provide a better indication of the mixtures long term performance (Coplantz, 1987).

Table 2.2. NCHRP 246 – Indirect Tension Test and/or Modulus Test with Lottman Conditioning

Specimens	9 samples divided into 3 groups Size: 4-in. diameter by 2.5-in. height	
Compaction	ASTM Methods: D1559 or D1561 or D3387	
Air Voids (%)	Expected Field Level or Mix Design Value	
Procedure	Group I:	• Water bath (in jars) for 5 hours → Test
	Group II & III:	• Vacuum saturation @ 26 in. Hg for 30 min • Atmospheric pressure, submerged, for 30 min
	Group II:	• Water bath @ test temperature for 3 hours → Test
	Group III:	• Freeze @ 0.0°F for 15 hours • Water bath @ 140°F for 24 hours • Water bath @ test temperature for 3 hours → Test
	Note: Tests can be run at 55°F or 73°F	
Damage Analysis	Ratios: Diametral Resilient Modulus Test Diametral Tensile Strength Test $\frac{\text{Group II}}{\text{Group I}}$ Short Term (saturation) $\frac{\text{Group III}}{\text{Group I}}$ Long Term (accelerated)	
Advantages	• Conducted on lab mixes, field mixes, or core samples • Severe test • Can differentiate between additive levels • Good correlation with field performance • Does not give biased results toward lime or liquid additive	
Disadvantages	• Time consuming (about 3 days for one freeze-thaw cycle) • Amount and type of equipment required is not always readily available	

Table 2.3. NCHRP 274 - Indirect Tensile Test with Tunnicliff and Root Conditioning

Specimens	6 samples – 2 groups of 3 Size: 4–in. diameter x 2.5 in. height (for aggregate ≤ 1 in.)
Compaction	ASTM Methods: D1559 or D1561 or D3387
Air Voids (%)	6 to 8 or expected field level
Procedure	Sort into groups so average air voids are approximately equal
	Group I: • Store dry at room temperature • Prior to testing, soak 20 min. @ 77°F → Test Group II: • Obtain a 55% to 80% saturation level (20 in. Hg for about 5 min in distilled water) • Reject if saturation is > 80% • Soak 24 hours @ 140°F • Soak 1 hour @ 77°F • Test
Damage Analysis	• Diametral Tensile Strength (ASTM D 4123) • Visual
Advantages	• Can use lab, plant, or field mixes; also cores from existing pavements • Mixtures with or without additives • Time required is moderate • Initial indications show good correlation (based on 80% retained strength)
Disadvantages	• May require trial specimens to obtain air void level • May not be severe enough

Table 2.4. Boiling Tests.

Specimens	Field mixture representation @ design AC or Individual aggregate size (coat aggregate with AC)
	Note: • Use of 200 or 300 gram sample is common • Use of agitation is agency dependent • Specific evaluation techniques vary among agencies
Compaction	None - Loose mix sample
Air Voids (%)	Not applicable
Procedure	• Place 500 ml of distilled water in 1000 ml beaker • Heat to boil, then add mixture • Boil 10 min, stirring 3 times with glass rod • Skim asphalt off surface • Cool to room temperature, dry on paper towel
Damage Analysis	• Visual assessment • Texas Boiling Test < 70% retained indicates moisture susceptibility
Advantages	• Can be used for initial screening • Minimum amount of equipment required • Can be used to test additive effectiveness • May be used for quality control • Can use lab mix or field mix
Disadvantages	• Subjective analysis • Uncompacted mix • Water purity can affect coating retention • Assessment of stripping in fines is difficult

Table 2.5. Texas Freeze-Thaw Pedestal Test.

Specimens	3 to 5 briquets 1-5/8-in. diameter x 3/4-in. height AC @ 5% > optimum Aggregate: Passes No. 20 (0.850 mm), Retained No. 35 (0.500 mm) sieve
Compaction	In mold under 6200 lb for 20 min
Air Voids (%)	Not specified
Procedure	• Cure briquets @ 75°F for 3 days • Place specimens on stress pedestal in water bottle • Freeze @ 10°F for 15 hours • Place in warm water 75°F (room temperature) for 45 min • Place in 120°F oven for 9 hours • Repeat, beginning at freeze, if cracking is not present
Damage Analysis	• Visual observation • If crack develops in < 10 cycles, moisture susceptible > 20-25 cycles, resists moisture damage
Advantages	• Used to test additive effectiveness
Disadvantages	• Uses only a small portion of the mix • Only fair correlation between field and lab results • Measures only cohesion • Takes time, 1 day for each cycle

Table 2.6. Immersion-Compression Tests (ASTM D-1075, AASHTO T-165).

Specimens	6 samples - 2 groups of 3 4 in. x 4 in.	
Compaction	Double plunger - pressure 3000 psi for 2 min (ASTM)	
Air Voids (%)	6	
Procedure	Group I:	Air cured for ≥ 4 hours @ 77°F → Test
	Group II:	• Water bath @ 120°F for 4 days • Water bath @ 77°F for 2 hours → Test (Alternate) • Water bath @ 140°F for 24 hours • Water bath @ 77°F for 2 hours → Test
Damage Analysis	• Visual assessment • Unconfined compression @ 77°F and 0.2 in./min	
Advantages	• Uses actual mix	
Disadvantages	• Time required can be extensive • Poor reproducibility • Air void level plays significant role • Water quality (ions and salts) can affect moisture sensitivity • Equipment may not be readily available	

The NCHRP 246 procedure with Lottman conditioning provides a high correlation to field data. One drawback to the test is the time required (about 3 days) to run the test. However, the flexibility of being able to test lab mixes, field mixes, or core samples is important. The NCHRP 246 Procedure is also sensitive to additive levels. The use of the nondestructive Diametral Resilient Modulus Test is another key consideration in using this procedure.

3.0 REVIEW OF ODOT PRACTICE

3.1 Overview

A presentation of practices currently used by ODOT is appropriate now that a background has been provided from the literature review. This section reviews the procedures used to determine a moisture susceptible mix and the type of treatments used to reduce the moisture susceptibility of a mix. The testing procedures used by ODOT have been incorporated into the section on chemical additives because these tests are only used to determine if a chemical antistripping agent is required and if the additive is effective. Testing procedures are not currently used to determine when lime treatment is required. The criteria used to determine when the aggregate should be treated with lime are related to geographical and other experience factors as discussed later in this chapter. When a mix meets the requirement for lime treatment of the aggregate and the requirement for a chemical additive, then both are administered to the asphalt mixture.

3.2 Testing Procedures and Chemical Additives

An index of retained strength (IRS), following AASHTO procedures T-165 (Effect of Water on Cohesion of Compacted Bituminous Mixtures), and T-167 (Compressive Strength of Bituminous Mixtures), criteria is used for evaluating the need and effectiveness of chemical antistripping additives. The alternate method of conditioning, as outlined by AASHTO in method T-165, is used by ODOT and is summarized below:

1) Immerse specimens in a water bath for 24 hours at 60° ± 1°C (140° ± 1.8°F).
2) Transfer to water bath at 25° ± 1°C (77° ± 1.8°F) for 2 hours.
3) Test following AASHTO T-167 procedures

$$\text{IRS} = \frac{(\text{Compressive Strength of Conditioned Specimen})(100)}{(\text{Compressive Strength of Unconditioned Specimen})}$$

To determine mix design asphalt contents, five specimens are prepared at increments of 0.5% asphalt. For IRS determination, the specimens are selected to represent the low, medium, and high range of asphalt content used in the mix design process. Interpolation is used to establish an IRS at the asphalt content selected for the design mix. When the IRS is less than 75%, at the design asphalt content (70% at the minimum asphalt content prior to 1984), a chemical antistripping agent is required along with an IRS test to establish whether the mix with the chemical additive meets the 75% retained strength criterion. The selection of the brand and grade of chemical additive is made by the asphalt supplier. Oregon DOT currently does not have specific criteria that must be met by the chemical additive other than the IRS improves to 75% or greater.

In an effort to enhance their prediction of moisture susceptible mixes, ODOT began utilizing a resilient modulus test (M_r) in addition to IRS testing. The M_r test remains at this date a secondary testing procedure and is used only on selected projects. Initial resilient modulus testing and evaluation was started in 1980. Beginning in 1983, the resilient modulus test was used when a project required 15,000 tons or more of asphalt concrete mixture. In 1986, the criteria was changed to all projects requiring 10,000 tons or more of asphalt concrete mixture. A resilient modulus ratio less than 0.70 indicates a moisture susceptible mix and a chemical additive is required. The conditioning procedure outlined by Lottman in NCHRP 246 is followed and is shown in Figure 3.1.

3.3 Lime Treatment of Aggregate

The use of hydrated lime in Oregon's asphalt concrete paving mixtures was first experienced in 1967. Lime or portland cement has been added to all open-graded mixes to increase asphalt viscosity and reduce asphalt run off in thick film mixes. One to 2% lime as a filler is added at the pugmill or cold feed for drum mix plants. The benefit from lime addition to prevent pavement moisture damage was first experienced on a test section of a project in Central Oregon (Willamette Highway). On the section without lime, severe raveling developed during the first winter after paving and rapid loss of the open-graded surfacing resulted. Testing of the paving mix without lime addition had very low AASHTO T-165 IRS values.

In 1974, Oregon pavements began experiencing problems, in the form of excess surface raveling, asphalt stripping, surface flushing, tenderness, or rutting. During 1975 through 1977, pavements with and without problems were inspected and sampled by representatives of ODOT, Oregon Asphalt Paving Association and asphalt suppliers. It was found that the problem pavements

CONDITIONING SEQUENCE	
Day 1	
	Fabricate - cool 24 hours (in air bath)
Day 2	
	M_r test (I) @ 77°F (from air bath)
	Vacuum saturate for 30 minutes at 27 inches Hg
	Rest samples for 30 minutes
	Place in 77°F water bath for 3 hours
	M_r test (II)
	Vacuum saturate for 30 minutes
	Double wrap and place in freezer for 15 hours
Day 3	
	Remove from freezer
	Place in 140°F water bath for 30 minutes (with wrapping)
	Remove wrapping
	Replace in 140°F water bath for 24 hours
Day 4	
	Remove from 140°F water bath
	Place in 77°F water bath for 3 hours
	M_r test (III)
Report: M_r Ratio I: $(M_r II/M_r I)(100)$ M_r Ratio II: $(M_r III/M_r I)(100)$	Criteria: M_r Ratio II $\geq 70\%$

Figure 3.1. ODOT Conditioning Procedure for Resilient Modulus Test.

generally had extreme variation in asphalt content, gradation, compaction and aggregate quality with a high percentage of aggregate without asphalt coating. A review of our asphalt concrete mix design records indicated an increased need and use of asphalt antistripping additives. Prior to 1974, less than 1% of paving required additives, but by 1977 the use had grown to 26%. Several changes in specifications were made in 1978; included were improved control of aggregate gradation and quality, increased control of contractor's paving operation and changes in the asphalt cement specification requirements.

Initially, the 1978 specification changes resulted in pavement problems being reduced and several were eliminated. However, by 1983, following the specification changes, problems related to asphalt stripping returned to a severe level. From a study of Oregon's pavement problems and a survey of experiences of other organizations, it was decided by the ODOT specification committee that for selected projects lime treatment of the aggregate would be required in asphalt concrete paving mixtures.

An initial draft of Oregon's paving specifications required the addition of 1% lime at the paving plant aggregate cold feed with mixing in the paving plant. However, based on information and reports received from other agencies, it was decided that the final draft of the specification should require pugmill mixing of lime with each size aggregate followed by mellowing in a stockpile prior to use. The 1984 specification required mandatory lime treatment of paving aggregate on selected projects based on past experience with aggregate or pavement problems and on a required basis for projects when mix design testing for AASHTO T-165 IRS is less than an 80% level. Resulting from problems with the "when required" basis for lime addition in the 1984 specification, a 1985 specification was developed which required mandatory lime treatment of aggregate on selected projects.

Paving in 1984 used lime-treated aggregate on 15 projects which accounted for 18% of Oregon's total tonnage for the year. During the same year chemical antistripping additives were used on 8% of the paving. In 1985 paving, 33 projects used lime-treated aggregate which represents 46% of the total paving tonnage. Chemical additives were used on projects that amounted to 14% of the total paving tonnage. During more recent years the use of lime and chemicals has continued near the 1985 level.

The decision to lime treat aggregate is based not on the results of test procedures, but on geographic location, historical knowledge, or on the type of facility. The criteria, as established by the ODOT 1987 mix design guide, requires the lime treatment of aggregates when one of these conditions are met:

1) The project elevation is above 2500 ft (MSL)
2) Freeze-thaw conditions exist
3) Known poor aggregates are to be used
4) Known poor pavement performance
5) Freeways facility (Interstate, Beaverton-Tigard, and Sunset) (Quinn et al., 1987)

According to ODOT's special provisions lime may be added by either of the following methods:

1) **Dry lime treatment**
   - Mix the hydrated lime, water, and aggregate thoroughly in a pugmill or other approved mechanical mixer,
   - Hold the moisture content to at least 5.0% for fine aggregate and 2.5% for coarse aggregate,
   - Age the mixed material for at least 5 calendar days before use.
2) **Lime slurry treatment**
   - Mix the slurry and aggregate thoroughly in a pugmill, or other approved mechanical mixer,
   - Be sure the water content of the slurry is at least 70% water by weight when added to the aggregate,
   - Age the mixed material for at least 24 hours before use.

3.4 Discussion

Oregon DOT currently employs two tests to determine if an asphalt concrete mixture is moisture susceptible. The IRS test requires 2 days before results are achieved whereas 4 days are required for the M_r test. This advantage may not be sufficient to warrant continued use of the IRS test. The

IRS test, as will be shown later in Chapter 4, does not provide a good indication of moisture susceptible mixes. The resilient modulus test did provide a better indication as presented from the data in this report.

Consideration should be made in regards to developing acceptance criteria of chemical additives in addition to the required improvement in IRS or M_r above acceptance criterion. Yoon et al. (1989), reported all six of the chemical antistripping additives they tested showed a decrease in concentration and effectiveness when stored at temperatures greater than 300°F. Possible steps include the development of a list of acceptable chemical additives or specific time limitations for the asphalt/additive mix at elevated temperatures during construction.

Specifications in Oregon require stockpiling of the lime-treated aggregate when either dry lime or lime slurry is used. The literature review (Volume II), however, indicates that stockpiling is not necessary for the lime treatment of the aggregate to be effective. However, as discussed later in Chapter 4, the stockpiling does not appear to have a detrimental effect on the moisture susceptibility of an asphalt mixture.

4.0 PROJECT ANALYSIS

This chapter describes the results of a one-week pavement condition survey conducted in April 1988, the analysis of core samples, and an evaluation of the asphalt concrete mixtures on 32 projects throughout the state of Oregon. The items discussed include project descriptions, pavement condition survey procedures, core evaluation procedures, historical data (lab test results and aggregate stockpiling times), and the presentation and discussion of the results.

Both the condition survey and the core evaluation can be used to determine the extent of stripping. Although the core evaluations would seem to be most closely related to processes which occur within the pavement, the validity of these evaluations is based on one assumption; it is assumed that initial coating at laydown was near 100% and that the coating decreased as pavements stripped from exposure to moisture, freeze-thaw cycles, and traffic loading.

It is assumed that two core locations represent an entire project. The percent coating observed in cores taken only a few feet apart sometimes show large differences. To some degree, the differences between samples might be explained by exposure to traffic, since one was taken in the wheel track and one taken at the fog-line. Observation of the data (Appendix), however, does not reveal that there is consistently any less coating in the wheel track. Note, however, that the exact sample location is not always recorded and those that were recorded were subject to question. This is due to the number of cores taken from the fog-line which have lower air void contents than core samples taken from the wheel track. Thus, there is no way to determine if traffic loading may be responsible for the differences between cores.

4.1 Project Description

The projects, selected by the Oregon Department of Transportation (ODOT), were from each of the following categories: asphalt concrete without additive treatment, asphalt concrete with lime-treated aggregate, and asphalt concrete with chemical additives introduced into the asphalt. Of the 32 initial projects selected, 10 did not receive additive treatment, 19 had lime-treated aggregate, and 3 had liquid chemical additives. The projects are listed on Table 4.1 and their locations shown in Figure 4.1.

Table 4.1. Project Description

Project Rank	Project Name	Date Constructed
	Without Additives	
1W	Baldock Slough South Baker Interchange	1979
8W	Bend S City Limits-Murphy Rd	1979
10W	Port of Morrow Interchange	1984
3W	Rickreall-Suver Junction	1984
6W	Moro-Grass Valley	1984
2W	Monroe-Crow Creek	1985
7W	Rondo-Blossom Lane	1985
4W	Sherwood-Rex Hill	1986
9W	Burlington-Willbridge	1986
5W	East Stayton-Mehama	1986
	Lime-Treated Aggregates	
2L	McKay Reservoir-MP 11.17	1984
6L	Marion County Line-Bugaboo Rd	1984
18L	Salt Creek Tunnel-Klamath County Line	1984
14L	Hermiston Hwy-Washington State Line	1985
19L	Lava Butte-Fremont Junction	1985
11L	Tower Rd Int.-Stanfield Int.	1985
4L	La Grande S City Limits-Hot Lake/Apt. Rd	1985
1L	Suttle Lake-Sisters	1985
8L	N. Santiam River-Lava Lake Meadow Rd	1985
10L	Haines-Pochontas Rd	1985
5L	Hermiston-Stanfield	1986
15L	Minam-Spring Creek	1986
7L	Meacham-Hilgard	1986
16L	North Powder-Haines	1986
9L	Sandy River-Corbett Interchange	1986
3L	Eagle Creek-Salt Creek Tunnel	1986
17L	McNary Hwy-Umatilla	1986
13L	Linn Co Line-Suttle Lake	1987
12L	Irrigon Jct-First St	1987
	Amine-Treated Asphalts	
1C	Plainview Rd-Deschutes River	1980
2C	Sandy River-Mitchell Point	1984
3C	South Baker-Durkee	1984

NOTE: Sites are ranked by the lowest retained coating obtained from the two cores for each project.

W - Projects without additives
L - Projects with lime-treated aggregate
C - Projects with an amine chemical additive

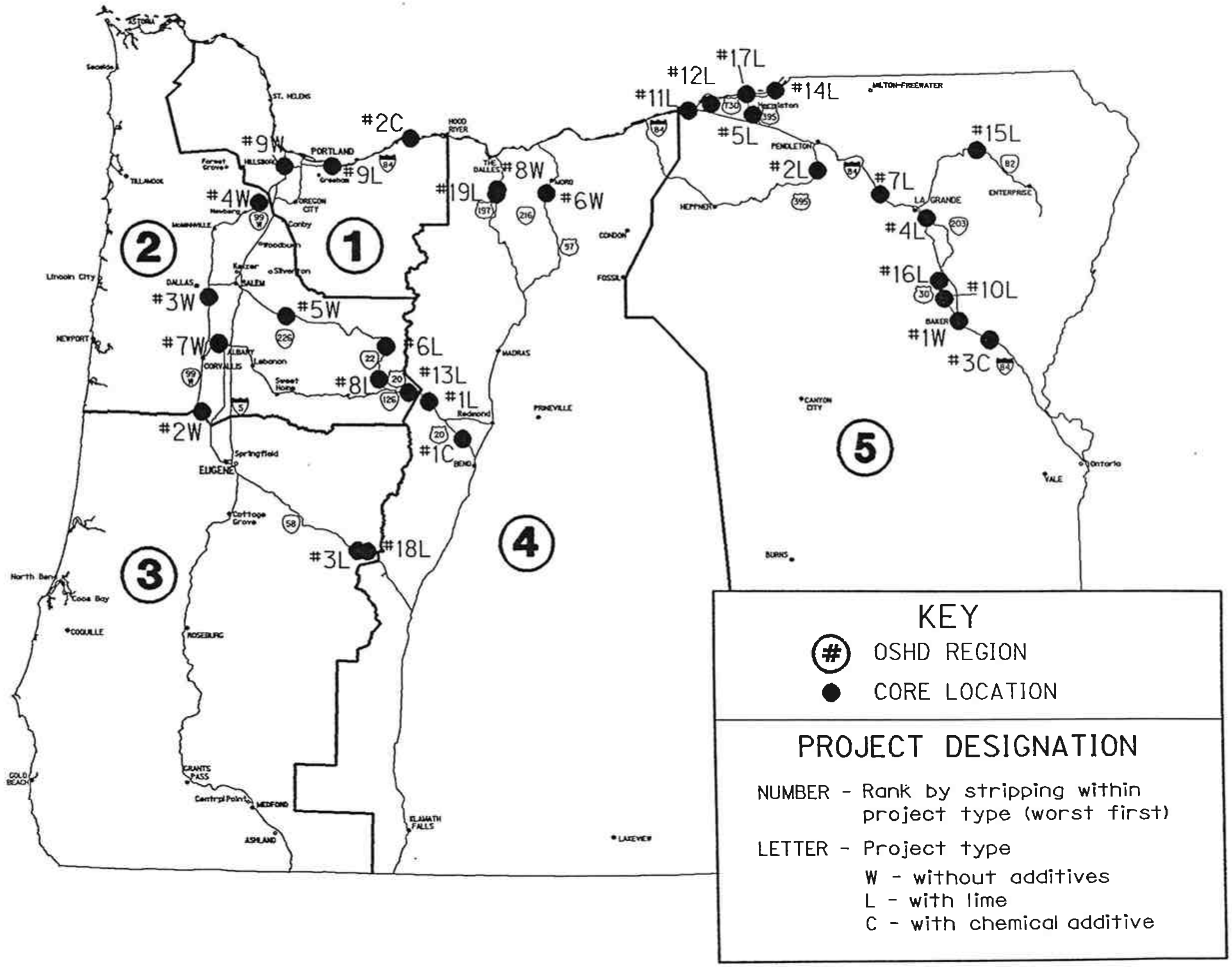


Figure 4.1. Project Site Locations.

For each of the projects, a pavement condition survey was conducted and two 6-inch diameter core samples were taken. The condition survey was used to visually evaluate the road for distress related to stripping. The core samples were evaluated to determine which projects were experiencing stripping. A comparison of the condition survey and core analysis results was conducted to assess their correlation.

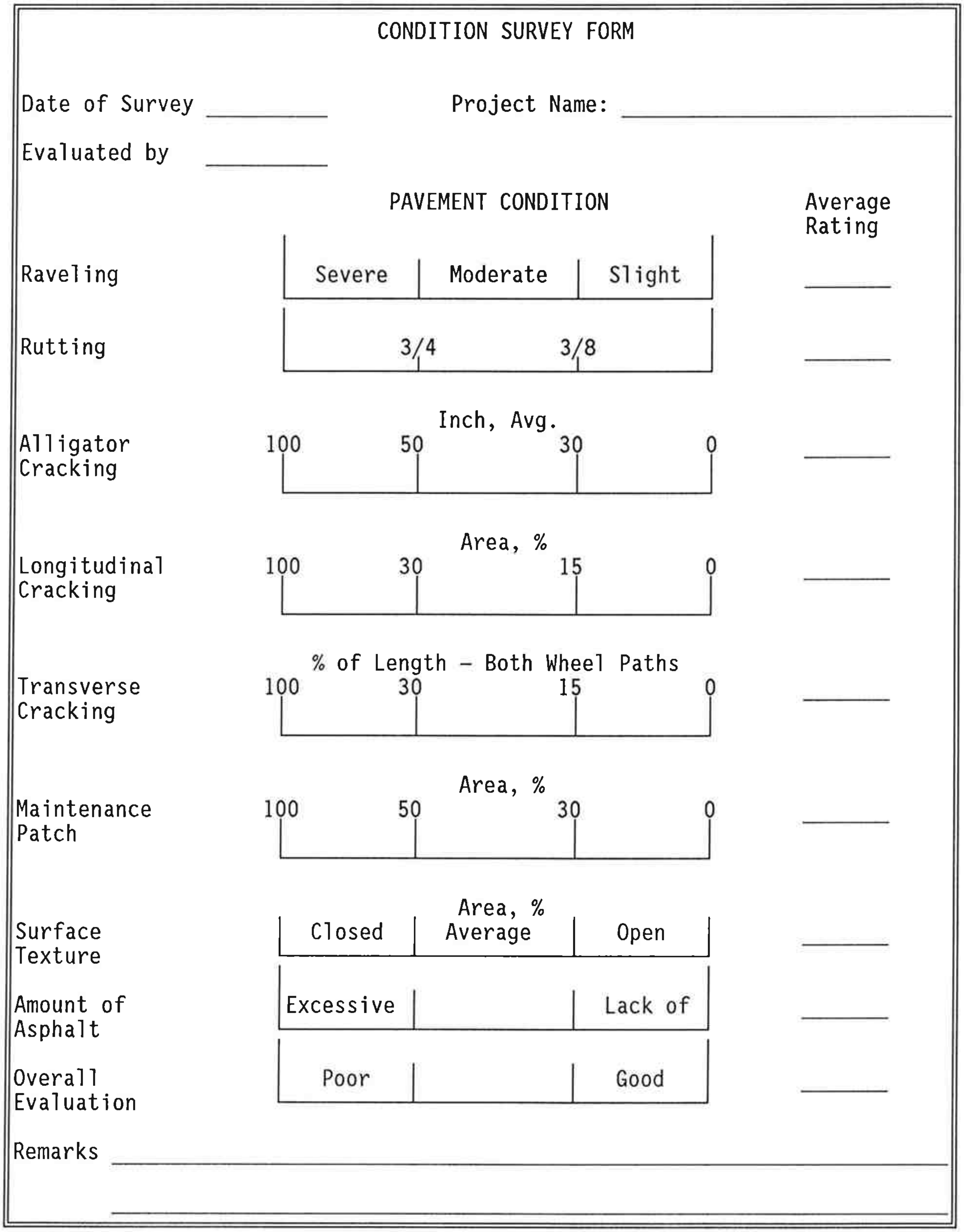
4.2 Condition Survey Procedure

The pavement condition survey results are a compilation of the scores from a minimum of four raters. The survey form used is shown in Figure 4.2. A riding score, which is sometimes incorporated into a condition survey, was not determined because a proper evaluation could not be made with the vehicle used for transportation. Each project, except for the Rondo to Blossom Lane project, was evaluated at two locations.

The results of the field survey are summarized in Tables 4.2 (for projects without additive treatment) and Tables 4.3 and 4.4 (for projects with additive treatment). The results for the condition survey are provided in tabular form in the Appendix.

4.3 Core Evaluation Procedure

Of the two 6-in. cores taken from each project site, one was taken in the wheel path and the other from between the outside wheel path and the shoulder. The cores were evaluated as outlined in Figure 4.3. The cores were visually inspected by personnel from both ODOT and Oregon State University (OSU) to determine the percentage of retained asphalt coating. A visual rating board was set up prior to evaluation of the cores to limit the subjective nature of visual evaluation. The samples were then laid out in order of percent coating. This direct comparison then allowed adjustments in the ranking. A numerical value was assigned representing the percent of asphalt coating on the aggregate. In addition, a rating of dry, sufficient, or thick was assigned to each core reflecting the asphalt thickness on the aggregate. The asphalt thickness rating is the same one used by ODOT for mix design evaluation. The independent results were discussed and a final decision on the condition of the asphalt concrete in the pavement structure was reached. The results of the coating evaluation and the air voids for the cores is presented in Tables 4.2, 4.3, and 4.4. In addition, the bulk specific gravity, rice



CONDITION SURVEY FORM

Date of Survey ________ Project Name: ____________________

Evaluated by ________

PAVEMENT CONDITION

Average Rating

Raveling | Severe | Moderate | Slight | ______

Rutting | 3/4 | 3/8 | ______

Inch, Avg.

Alligator Cracking | 100 | 50 | 30 | 0 | ______

Area, %

Longitudinal Cracking | 100 | 30 | 15 | 0 | ______

% of Length - Both Wheel Paths

Transverse Cracking | 100 | 30 | 15 | 0 | ______

Area, %

Maintenance Patch | 100 | 50 | 30 | 0 | ______

Area, %

Surface Texture | Closed | Average | Open | ______

Amount of Asphalt | Excessive | | Lack of | ______

Overall Evaluation | Poor | | Good | ______

Remarks ____________________

Figure 4.2. Example of Condition Survey Form.

Table 4.2. Projects without Additive Treatment.

Year Constructed	Project Name	Route #	Rank*	Visual Site Evaluation** Raveling	Visual Site Evaluation** Overall Eval.	Core Evaluation*** #1 % Coating	Core Evaluation*** #1 % Air Voids	Core Evaluation*** #2 % Coating	Core Evaluation*** #2 % Air Voids	Asphalt Brand	Asphalt Grade	IRS[1] Value	M_r[2] Value
1979	Baldock Slough-S. Baker Int.	I-84	1W	Moderate	Poor	40[4]	6.2	40	8.9	Shell	AR4000	100	---
1979	Bend South City Limits-Murphy Road	97	8W	Slight	Avg.	95	2.0	95	---	Chevron	AR4000	90	---
1984	Port of Morrow Interchange	I-84	10W	Slight	Good	95	6.0	95	---	Chevron	AR4000W	83	---
1984	Rickreall-Suver Junction	99W	3W	Slight	Good	60	6.6	95	4.4	Chevron	AR4000W	100+	0.78
1984	Moro-Grass Valley[3]	97	6W	Slight	Avg.	70	2.1	100	1.3	Chevron	AR4000W	100+	---
1985	Monroe-Crow Creek	99W	2W	None	Good	50	8.3	60	7.7	Chevron	AC 20	81	0.63
1985	Rondo Street-Blossom Lane	20	7W	Slight	Good	75	6.9	85	7.8	Chevron	AC 20	81	0.63
1986	Sherwood-Rex	99W	4W	Moderate	Avg.	70	7.4	90	6.0	McCall	AR4000	84	---
1986	Burlington-Willbridge	99W	9W	Slight	Good	95	3.0	95	3.5	McCall	AR4000	77	0.72
1986	East Stayton-Mehama	22	5W	Slight	Good	70	7.1	90	4.5	McCall	AR4000	86	0.64

*Sites ranked by evaluation of percent asphalt coating on core #1 (least % coating-#1W, greatest % coating-#10W). Core #2 and visual site evaluations were used when there was a tie.

**See Appendix for complete inspection data.

***See Appendix for complete test data.

[1]Index of Retained Strength (%).

[2]Resilient Modulus Ratio (M_r) test results from construction mix design records for freeze-thaw conditioned specimen.

[3]Identified as stripping from condition survey, see comment section, Table 1.0, Appendix.

[4]Core was microwaved for 2 minutes prior to breaking open for evaluation.

Table 4.3. Projects with Lime-Treated Aggregate.

Year Constructed	Project Name	Route #	Rank*	Visual Site Evaluation** Raveling	Visual Site Evaluation** Overall Eval.	Core Evaluation*** #1 % Coating	Core Evaluation*** #1 % Air Voids	Core Evaluation*** #2 % Coating	Core Evaluation*** #2 % Air Voids	Asphalt Brand	Asphalt Grade	# of Weeks Limed Agg. Stockpiled	IRS[1] Value	M_r[2] Value
1984	McKay Reservoir-Milepost 11.17	395	2L	Slight	Avg.	40	6.5	40	8.1	Chevron	AR2000	1	100+	0.71
1984	Marion Co. Line-Bugaboo Road	22	6L	Slight/Moderate	Poor	85	7.1	95	8.7	Chevron	AR2000	4	100+	---
1984	Salt Crk. Tunnel-Klamath Co. Line	58	18L	None	Avg.	100	4.5	100	---	Chevron	AR2000	1-2	100+	0.96
1985	Hermiston Hwy-Washington St Line	730 395	14L	None	Good	90	---	95	1.1	Chevron	AR2000	40	100+	1.32
1985	Lava Butte-Fremont Junction	197	19L	Slight	Good	100	3.4	100	---	Chevron	AC 20	2-8	99	1.00
1985	Tower Road Int.-Stanfield Int.	I-84	11L	Slight	Good	95	5.3	95	---	Chevron	AR4000W	3-4	87	0.72
1985	LaGrande-Hotlake/Airport Road	30	4L	Slight	Good	75	4.6	80	7.0	Sound	AR4000W	2	96	0.71
1985	Suttle Lake-Sisters	20	1L	None	Good	40	6.0	40	6.6	Chevron	AC 20	1-2	95	0.71
1985	N. Santiam River-Lava Lake Meadows Rd.	22	8L	Slight	Good	90	4.8	90	---	Chevron	AR4000W	40	100	---
1985	Haines-Pocohontas Road	30	10L	Slight	Good	90	3.5	100[3]	---	Sound	AR4000W	2	96	0.71
1986	Hermiston-Stanfield	395	5L	Slight	Good	80	---	90	7.5	Chevron	AC 20	40	94	0.83
1986	Minam-Spring Creek	82	15L	Slight	Good	95	3.6	95	---	McCall	AR4000	35	93	0.74
1986	Meacham-Hilgard	I-84	7L	Slight	Good	85	3.0	90	2.9	Koch	AC 20	1	83	0.71
1986	North Powder-Haines	30	16L	None	Good	95	11.1	95[3]	---	Sound	AR4000W	26-35	96	0.71
1986	Sandy River-Corbett Interchange	I-84	9L	Slight	Good	90	2.6	90	---	McCall	AR4000	4	100+	0.86

Table 4.3. Projects with Lime-Treated Aggregate (continued).

Year Constructed	Project Name	Route #	Rank*	Visual Site Evaluation**		Core Evaluation***				Asphalt		# of Weeks Limed Agg. Stockpiled	IRS[1] Value	M_r[2] Value
						#1		#2						
				Raveling	Overall Eval.	% Coating	% Air Voids	% Coating	% Air Voids	Brand	Grade			
1986	Eagle Creek-Salt Creek Tunnel	58	3L	Slight	Good	70	5.6	70	6.3	Witco	AR4000	6	89	---
1986	McNary Highway-Umatilla	730	17L	None	Good	95	2.2	95	---	Chevron	AC 20	2-3	100+	---
1987	Linn County Line-Suttle Lake	20	13L	None	Good	95	7.6	95	---	Witco	AR4000	43-52	97	0.87
1987	Irrigon Junction-First St. (Irrigon)	730	12L	Slight	Good	95	1.6	95	---	Idaho Asphalt	AR4000W	2-3	75	0.87

*Sites ranked by evaluation of percent asphalt coating on core #1 (least % coating-#1L, greatest % coating-#19L). Core #2 and visual site evaluation were used when there was a tie.

**See Appendix for complete inspection data.

***See Appendix for complete test data.

[1]Index of Retained Strength (%).

[2]Resilient Modulus Ratio (M_r) test results from construction mix design records for freeze-thaw conditioned specimen. All tests are believed to have been performed <u>with</u> lime-treated aggregate.

[3]Core was microwaved for 2 minutes prior to breaking open for evaluation.

Table 4.4. Projects with Chemical Anti-Stripping Agents.

Year Constructed	Project Name	Route #	Rank*	Visual Site Evaluation**		Core Evaluation***				Asphalt		Type and % of Added Chemical	IRS[1] Value	M_r[2] Value
						#1		#2						
				Raveling	Overall Eval.	% Coating	% Air Voids	% Coating	% Air Voids	Brand	Grade			
1980	Plainview Road–Deschutes River	20	1C	Slight	Good	40	6.9	40	7.7	Chevron	AR4000W	0.3% NSSC[3]	90	0.53
1984	Sandy River–Mitchell Point	I-84	2C	Moderate	Avg./Good	60	3.9	90	2.0	Chevron	AR4000W	0.3% PBS[4]	75	0.50
1984	South Baker–Durkee Interchange	I-84	3C	Slight	Avg.	100	2.8	100[5]	---	Chevron	AR4000W	0.5% PBS[4]	80	0.65

*Sites ranked by evaluation of percent asphalt coating on core #1 (least % coating-#1C, greatest % coating-#3C). Core #2 and visual site evaluation were used when there was a tie.

**See Appendix for complete inspection data.

***See Appendix for complete test data.

[1]Index of Retained Strength (%).

[2]Resilient Modulus Ratio (M_r) test results from construction mix design records for freeze-thaw conditioned specimen. All tests performed without additive.

[3]No Strip Super Concentrate (NSSC), chemical agent: SC-901.

[4]Pave Bond Special (PBS), chemical agent: amine.

[5]Core was microwaved for 2 minutes prior to breaking open for evaluation.

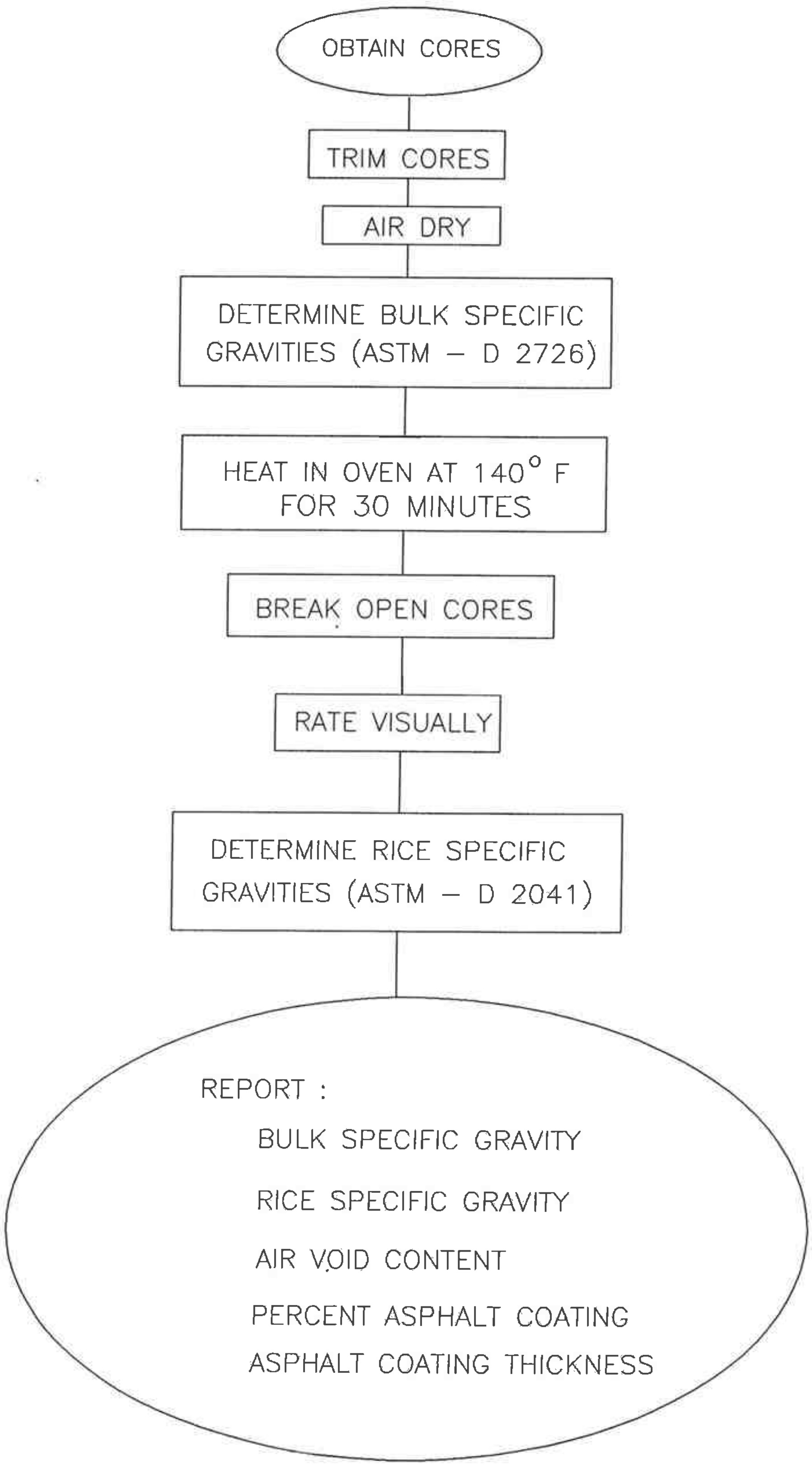


Figure 4.3. Core Analysis Procedure.

specific gravity, and coating thickness results from the core analysis are listed in Table 4.0 in the appendix.

Four cores, one from four separate projects, were inadvertently microwaved for two minutes prior to breaking open for evaluation. The projects affected are: Baldock Slough to South Baker Interchange, Haines to Pocohontas Road, North Powder to Haines, and South Baker to Durkee. As discussed in Chapter 2, Al-Ohaly and Terrel (1988) have shown that microwave treatment of an asphalt concrete mixture can improve adhesion between the aggregate and asphalt. The influence of the microwave radiation on the percent coating rated for these cores is unknown. After reviewing the percent coating rated for all the cores associated with these projects, it was believed the results and analysis were probably not seriously affected by this occurrence.

4.4 Evaluation of Results

The results from the core evaluations for retained coating were used to make the determination that a project had stripped. The cores provide a direct visual examination into the asphalt concrete pavement. The condition survey, which evaluates the pavement surface for signs of internal pavement distress, did not always correlate signs of stripping with the core evaluation that stripping was indeed present within the pavement.

Theoretically, if a core did not receive a rating of 100% retained coating, then stripping is occurring within the pavement. The lowest rated core for each project was used. However, the 100% criterion is too severe as only 3 of the 32 projects would be considered as not stripping using this guideline. The relation between the number of projects classified as stripping as the percent coating criterion is varied from 60 to 95% is shown in Figure 4.4. As can be seen from the graph, the number of projects with an asphalt (chemical) additive remains unchanged at 2 for the entire coating range on the graph. For projects without additives, 7 would be classified as stripping if 75, 80, 85, or 90% were used as the decision criterion. The lime-treated aggregate projects, 19 of the 32 projects evaluated, show the greatest variation in the figure due to the larger number of projects. In addition, coarse aggregate may be partially stripped without severe detrimental effects to the pavement structure. Therefore, the decision was reached to use a retained coating of 80% or less on at least one core to determine if a project should be classified as stripping.

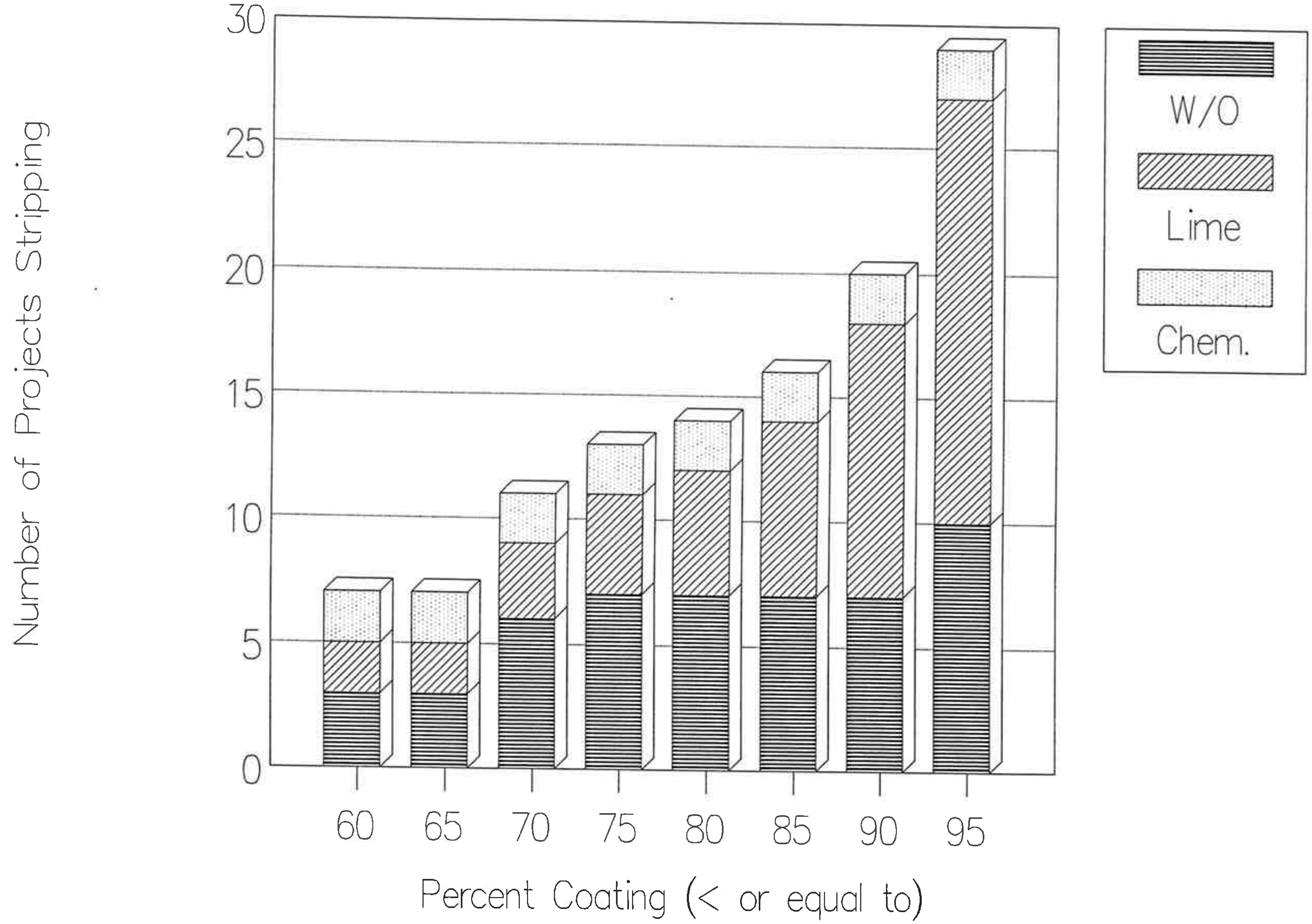


Figure 4.4. Number of Projects Classified as Stripping vs. Percent Asphalt Coating.

Of the 32 projects surveyed, 14 showed stripping in at least one of the cores and, of these, 7 had stripping in both core samples. Seventy percent of the projects that did not receive an additive treatment exhibited stripping (including 6 projects with only one core determined to be stripping). Of the 19 lime-treated aggregate projects, 4 had stripping occurring in both cores and one project had stripping in one core. Of the 3 amine-treated projects, 2 were classified as stripping, one in one core and the other in both cores.

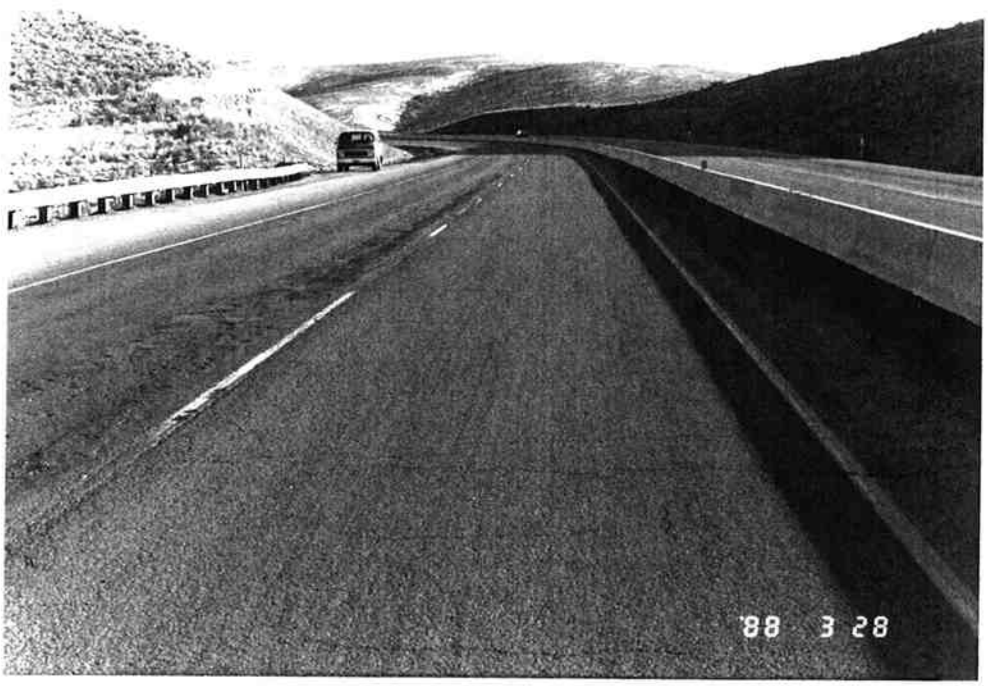
In order to evaluate Oregon projects for possible lime carbonation effects on the treated aggregate, information on the stockpiling time was compiled and is presented in Table 4.3. The values given account for construction time or in some cases the range also reflects a degree of uncertainty as the data was obtained from the memory of the project manager who worked on each job. Carbonation is further discussed in Section 4.4.2.

4.4.1 Projects without Additive Treatment

Projects without additive treatment range in age from 1 to 9 years and show varying levels of distress (Appendix). Presented in Table 4.2 are condition survey results for these projects which indicates only 2 of the 10 projects show signs of pavement failure: Baldock Slough to South Baker Interchange and Moro to Grass Valley. The results from the core analysis show 7 of the projects stripping. In addition to the projects mentioned above, Rickreall to Suver Junction, Monroe to Crow Creek, Rondo to Blossom Lane, Sherwood to Rex Hill, and East Stayton to Mehama are the projects identified as stripping.

Baldock Slough to South Baker Interchange, built in 1979, is the only project in this category with an overall condition survey evaluation of "poor." This project had received some chip seal maintenance. However, the outside lane at milepost 305.8 westbound did not receive a chip seal application. This outside lane is pitting in the wheel paths, as displayed in Figure 4.5a, and does not show signs of thermal cracking. However, the inside lane, with a chip seal, does not show signs of pitting, but thermal cracking is present as illustrated in Figure 4.5b. It appears the chip seal, acting as a barrier, has helped reduce the moisture damage in the outside lane by limiting water intrusion into the pavement structure. The aggregate in the mix has a coating of 40%, shown in Figure 4.6a, which indicates this project

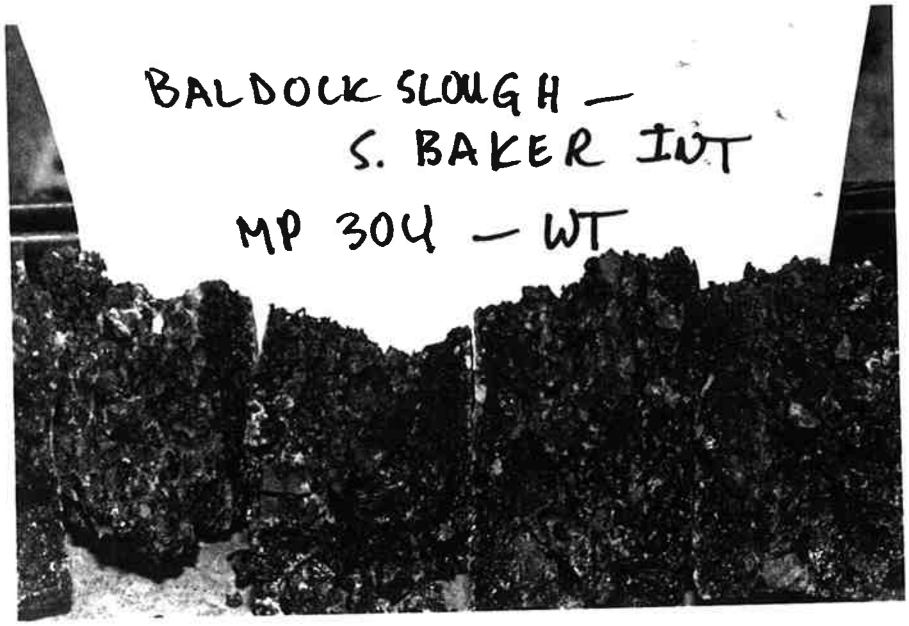
a) Milepost 305.8 Westbound



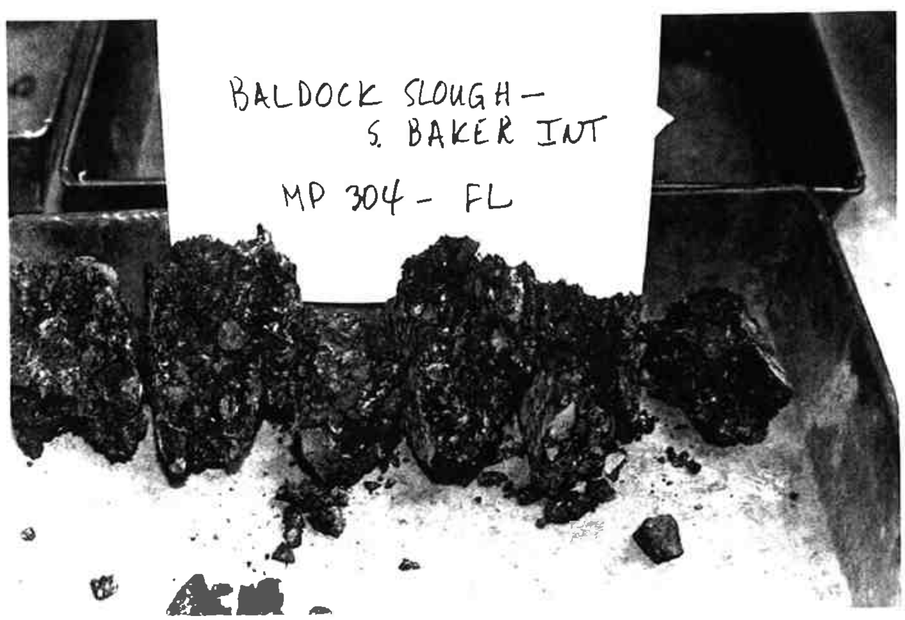
b) Milepost 305.8 Westbound

Figure 4.5. Baldock Slough–South Baker Interchange.

2



a) Wheel Track Core



b) Fog Line Core

Figure 4.6. Baldock Slough-South Baker Interchange.

2

3

and 4.6b, is stripping in the outside lane. Cores were not taken from the inside lane.

The Moro to Grass Valley project, built in 1984, has mixed results for the condition survey and the core analysis. The condition survey indicates the project is stripping as illustrated in the following sequence of figures. In Figure 4.7a, flushing is evident and Figure 4.7b shows the closed surface texture from the excessive asphalt on the pavement surface. A closer look reveals potholes have formed in the pavement structure as shown in Figure 4.8a. It is evident from Figure 4.8b that the adhesion between the asphalt and aggregate has severely deteriorated. This photo illustrates what can occur from the presence of moisture in an asphalt concrete pavement. The preferential wetting of the aggregate has displaced the asphalt binder from the aggregate. The core analysis for the other evaluation site shows one core with a thick coating over 100% of the aggregate and the other core, taken on the fog line, with a coating over 70% of the aggregate indicating that stripping is beginning at this location.

Rickreall to Suver Junction, Monroe to Crow Creek, and Rondo to Blossom Lane were evaluated as stripping from the core analysis while results of the condition survey indicates signs of stripping are not yet evident. The Monroe project is the only one of the three to have a retained asphalt coating less than 60% in both cores. The condition survey did not, however, indicate any sign of possible stripping in the pavement for the Monroe project.

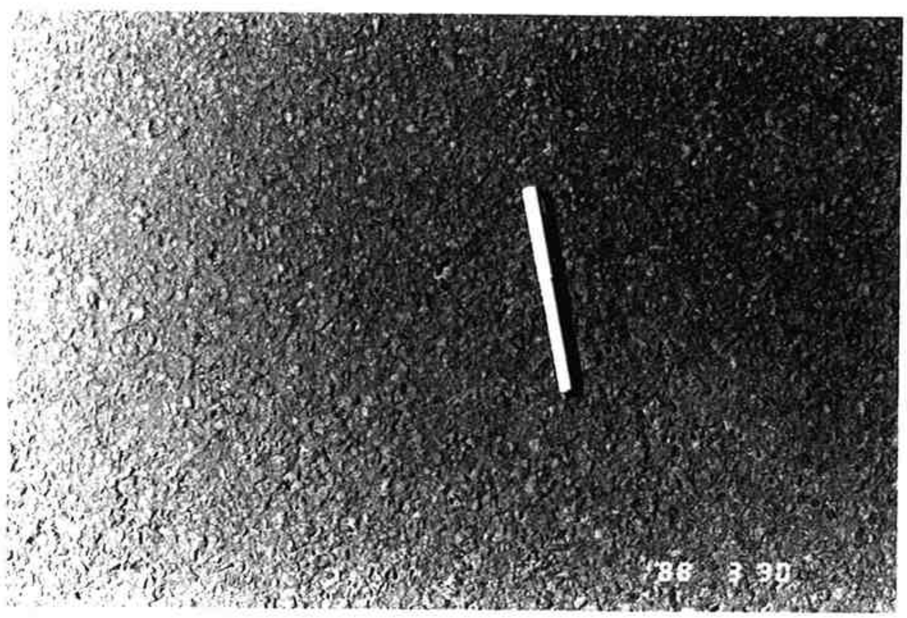
Of the other projects without additive treatment, two appear to be showing early signs of stripping. The projects are Sherwood to Rex Hill and East Stayton to Mehama. Both showed some initial signs of possible stripping from the condition survey and the core analysis. The Sherwood project was raveling in places, the East Stayton project was beginning to flush, and both had one core with a retained coating of 70%.

One project which, from the condition survey, had signs of possible stripping that was not confirmed from the core analysis. The Port of Moro Interchange project has a closed surface texture with excessive surface asphalt. However, the project had an asphalt coating of 95%. This could indicate the pavement contained an excessive amount of fine aggregate or asphalt in the mixture.

4

a) Overview

3



b) Close-up

Figure 4.7. Moro-Grass Valley

5

a) Overview

b) Close-up

Figure 4.8. Moro-Grass Valley

4

6

4.4.2 Projects with Lime-Treated Aggregate

Of the 19 projects with lime-treated aggregate, only three received an overall rating less than "good" from the condition survey as shown in Table 4.3. These projects include McKay Reservoir to Milepost 11.17, Marion County Line to Bugaboo Road, and Salt Creek Tunnel to Klamath County Line. All were constructed in 1984, the first year ODOT used lime-treated aggregate as an antistripping additive in asphalt concrete mixes. In addition to McKay Reservoir to Milepost 11.17, La Grande S. City Limits to Hot Lake/Apt. Rd, Hermiston to Stanfield, Suttle Lake to Sisters, constructed in 1985, and Eagle Creek to Salt Creek Tunnel, constructed in 1986, had asphalt coatings less than 85% and were therefore determined to be stripping.

The Marion County Line to Bugaboo Road project had the worst overall condition survey rating at "poor." However, it appears that the pavement distress producing this poor rating is not related to stripping. The alligator cracking, as shown in Figures 4.9a and 4.9b, is probably due to a base and/or subgrade failure. This is confirmed from the core analysis which shows an asphalt coating of 85% and 95%.

McKay Reservoir to Milepost 11.17 and Salt Creek Tunnel to Klamath County Line both received an overall rating of "average" from the condition survey. Figure 4.10a shows the pitting that is occurring at mile post #9 on the McKay Reservoir project which also has a closed surface texture at mile post #10 due to a high asphalt content for the mix gradation. The core analysis for McKay Reservoir indicates this project is stripping. The asphalt coating is 40%, as shown in Figure 4.10b, with a dry coating thickness for both the wheel track and fog line core locations. A major concern for this project is that there may be considerable variation from the mix design gradation and/or asphalt content during construction. Therefore, it could not be substantiated that the lime added to the McKay project met design criteria. The problem of a closed surface is also being experienced on the Salt Creek Tunnel project. However, the core results for this project show the aggregate is 100% coated with a thick film of asphalt.

Two other projects, constructed in 1986, Meacham to Hilgard and North Powder to Haines, showed signs of possible stripping. The condition survey found pitting in the Meacham to Hilgard project. The core analysis for this project shows 90% coating of the aggregate with a sufficient film of asphalt.

7

a) Overview

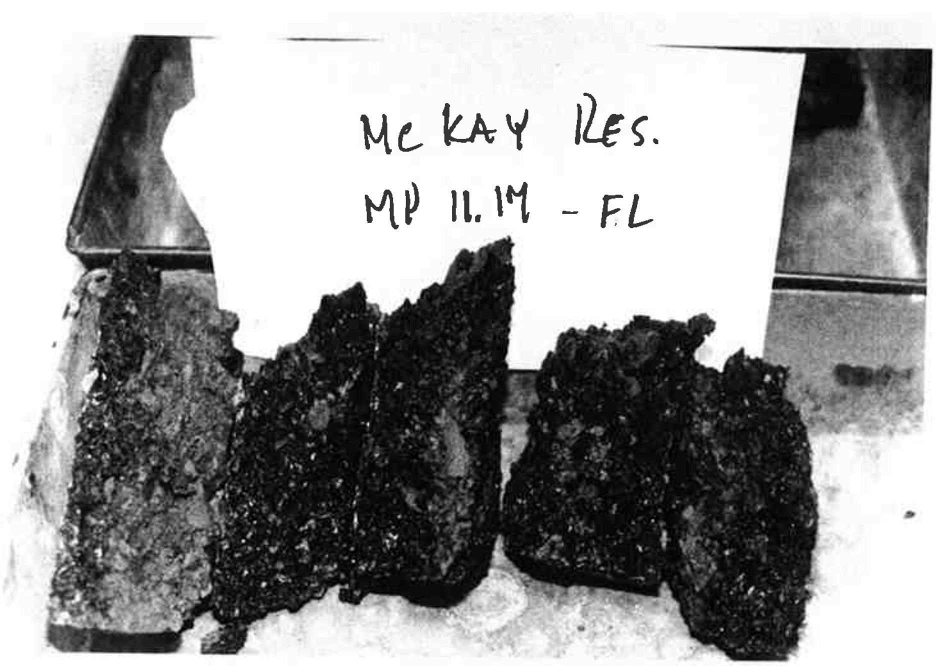
6

b) Closeup

Figure 4.9. Marion County Line–Bugaboo Road.

8

a) Overview



6

b) Close-up of Broken Cores

Figure 4.10. McKay Reservoir–Milepost 11.17.

9

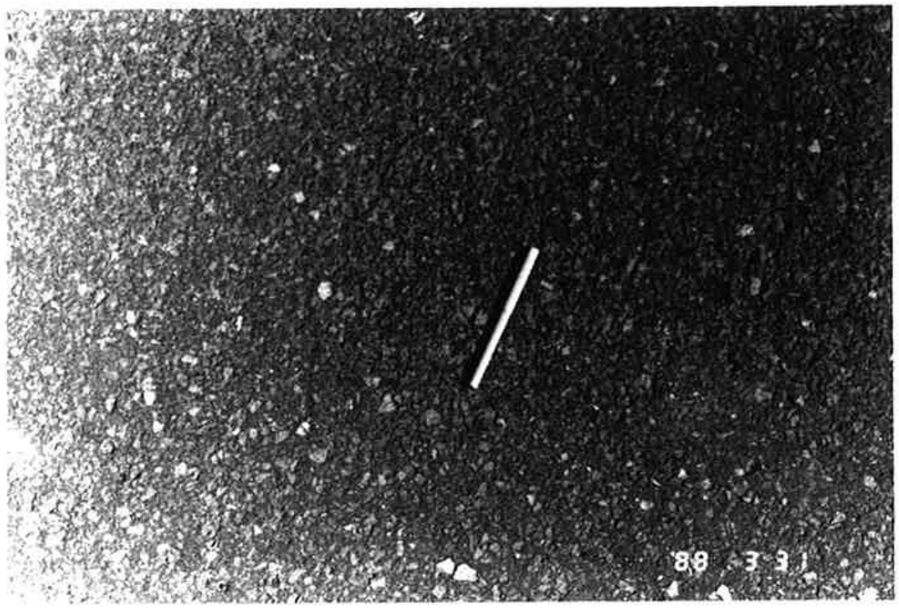
The North Powder to Haines project had a closed surface texture that may be due to excessive asphalt in the mix. The results from the core analysis show a 95% coating of the aggregate with a sufficient film of asphalt.

One project not identified from the pavement condition survey that is showing signs of stripping is Suttle Lake to Sisters. The core analysis rating was only 40% asphalt coating for both core samples with a sufficient thickness of asphalt. The project is pictured in Figure 4.11a and 4.11b. Again, there were questions during construction as to whether the lime was properly mixed in a pugmill with the aggregate prior to stockpiling. Another possible factor that could result in the low asphalt coating is the less than normal level of asphalt aging during paving plant mixing on the project.

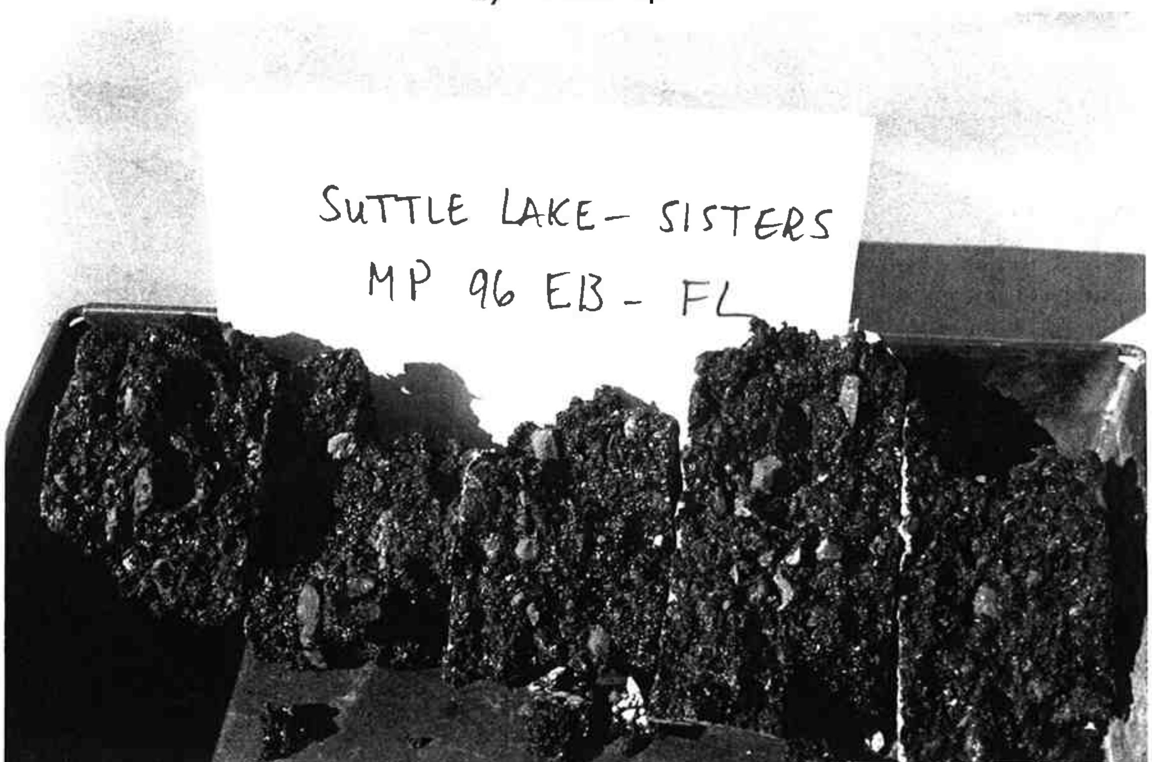
A review of the literature indicates that stockpiling of lime-treated aggregate prior to incorporating into an asphalt concrete mix is not required unless the fines are highly plastic. Current practice by ODOT requires stockpiling of the lime-treated aggregate prior to use in a mix. While ODOT does specify a minimum amount of time the aggregate must be stockpiled, a maximum time limit is not specified. Therefore, lime-treated aggregate may be stockpiled anywhere from one day to one year or more. It was the concern about the length of stockpiling and possible detrimental effects from carbonation that prompted an analysis of stockpiling time for the treated aggregate on the selected projects.

As discussed in Chapter 2, some researchers have suggested that the effectiveness of lime is reduced by stockpiling the treated aggregate for an excessively long period of time. When the lime reacts with atmospheric carbon dioxide or carbon dioxide dissolved in rainwater, it "carbonizes" or turns into calcium carbonate ($CaCO_3$). In this form it does not serve as an anti-stripping agent. The unknown factor is: How long does this chemical change take to occur inside of an aggregate stockpile? The literature review found no research that specifically addresses this question. It is known that the reaction occurs slowly and requires the presence of water and carbon dioxide. However, the conditions which allow water and CO_2 to come into contact with lime in the stockpile vary considerably. The only visual evidence that this reaction has occurred is "cementing" of the material. Observers from ODOT have described the cemented material, which is typical of material stockpiled for one winter, as being a thin crust no more than a few inches thick. None

10



a) Close-Up



7

b) Core Sample

Figure 4.11. Suttle Lake-Sisters.

11

of the projects in this study had lime-treated aggregate that was stockpiled more than one winter. During the course of this study, however, one project was found which was built with aggregate that was stockpiled over 2 winters (Lostine to Trout Creek - 1988). Severe cementing of aggregate on this job suggests that carbonation may affect the antistripping properties of the lime. This is documented here to encourage tracking of its performance in the future.

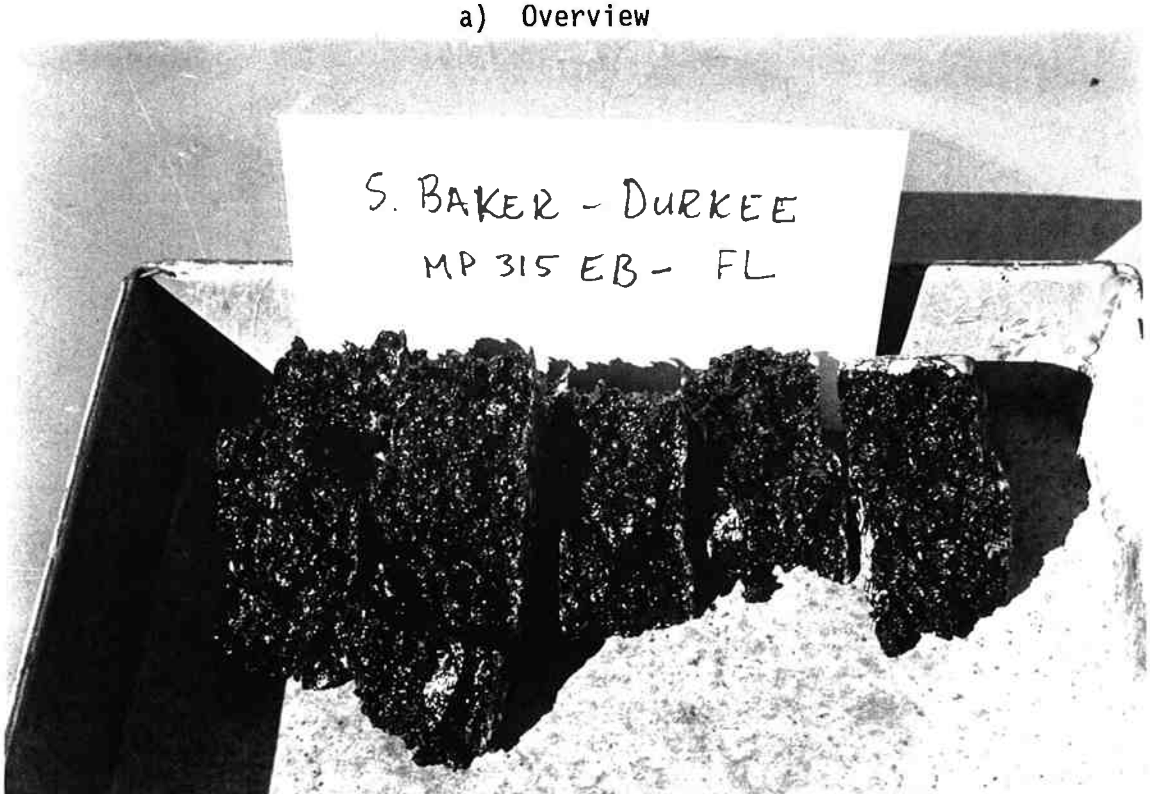
Of the five projects identified as stripping, 3 (McKay Reservoir to Milepost 11.17, LaGrande to Hot Lake/Apt. Rd., Suttle Lake to Sisters) used lime-treated aggregate stockpiled 2 weeks or less while the Eagle Creek-Salt Creek Tunnel project and the Hermiston to Stanfield project had stockpiling times of 6 and 40 weeks, respectively. It would appear from reviewing the length of time the aggregate was stockpiled for other projects, noting that six projects have times greater than 25 weeks, that the carbonation concerns appear to be unwarranted.

4.4.3 Projects with Amine-Treated Asphalt

Table 4.4 contains the three projects with amine-treated asphalt and they are: Plainview Road to Deschutes River, built in 1980; Sandy River to Mitchell Point and South Baker to Durkee, both built in 1984. The Plainview project received a 0.3% treatment with PaveBond Special while the Sandy River project and the South Baker project received 0.3% and 0.5%, respectively, by weight of asphalt, concentration of the same additive. The South-Baker to Durkee project, the only one not classified as stripping, has the highest concentration of additive.

Even though the Plainview project is four years older, its overall evaluation from the pavement condition survey was "good" compared to the "average" to "good" for the Sandy River project and the "average" rating received by the South Baker project. The Sandy River and South Baker to Durkee projects are showing signs of stripping-related distress. The Sandy River pavement has had frequent overlays and considerable rock loss. In the South Baker project, as shown in Figure 4.12a, the outside lane is flushed in the wheel paths. However, the results from the core analysis indicate the project is not stripping. Figure 4.12b shows the aggregate is 100% coated with a thick film of asphalt. The core analysis for the Sandy River to Mitchell point project

a) Overview



8

b) Core Sample

Figure 4.12. South Baker-Durkee.

13

indicates stripping is occurring. The distress noted in the Plainview project, shown in Figure 4.13a, is not stripping related. The thermal cracking radiating from the longitudinal crack along the center line is a reflection of the temperature extremes that occur in central Oregon. However, the core data indicates the project is stripping, the aggregate has an asphalt coating of 40% and a dry thickness of asphalt, as shown in Figure 4.13b.

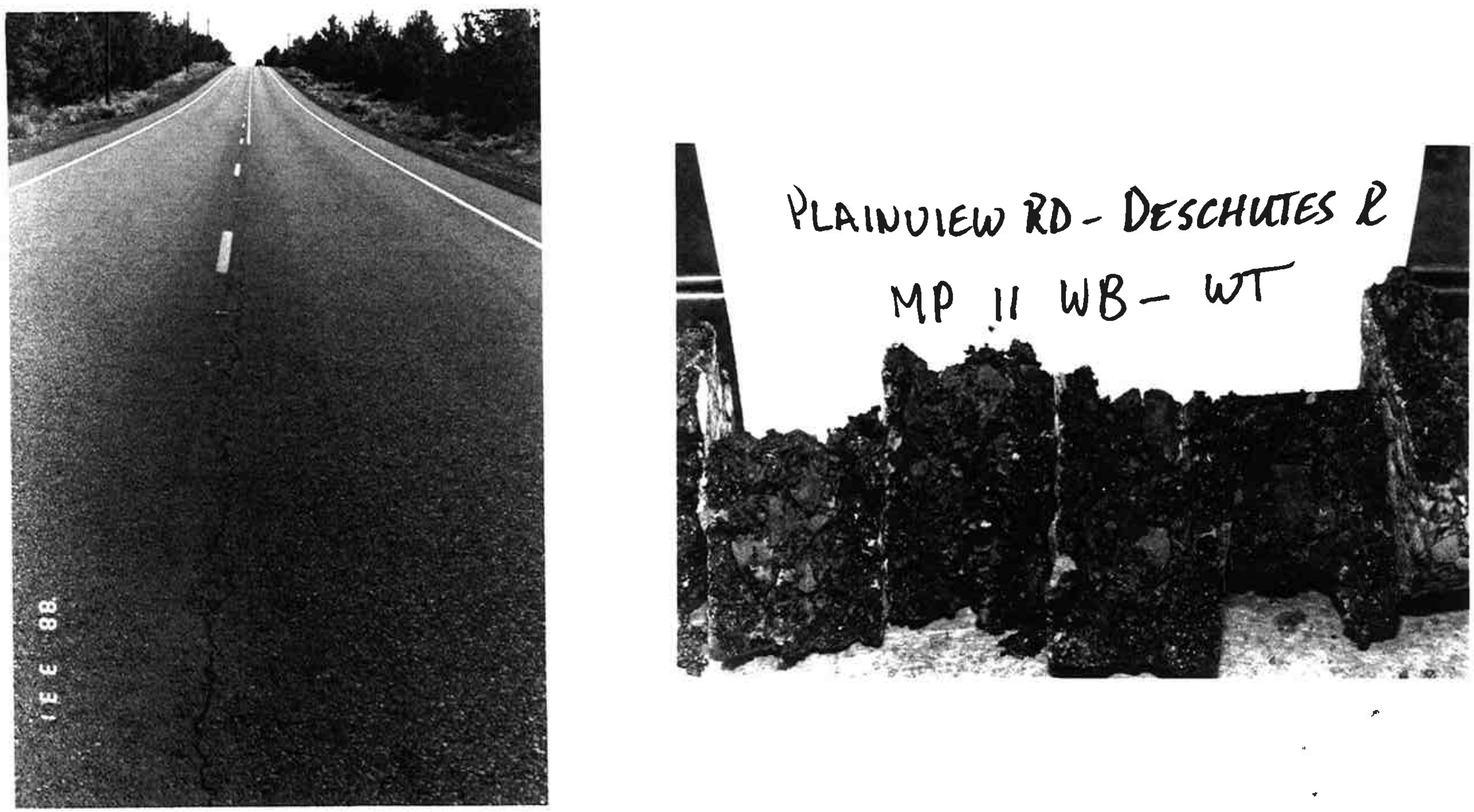
4.5 Mix Design Test Results

As mentioned earlier, 70% of the projects which did not have an anti-stripping additive were identified as stripping. All of these projects had an IRS of 81% or higher and 3 of these have an IRS of at least 100% as shown in Figure 4.14. As shown in the figure, there is no distinction between the projects classified as stripping and non-stripping, irrespective of the IRS value selected. The projects are shown by rank and a change in the percent coating criterion would not promote a relationship between IRS and projects classified as stripping. For example, if the criterion to determine if a project is stripping was changed to 60% or less asphalt coating, then 3 out of the 10 projects without an additive would be classified as stripping. This would be projects 1, 2, and 3. This change would result in two of the three projects with an IRS of 100% classified as stripping. A change in the coating criterion, the IRS criterion, or both does not reveal an IRS value that could be used to predict moisture susceptible mixes. Therefore, from this data, the IRS test does not provide a reliable indication of moisture susceptible mixes irrespective of the ratio chosen as a decision criterion.

A resilient modulus ratio test was conducted on five of the projects without an additive and four of these were classified as stripping. Of the four, three of the ratios were less than 70% as shown in Figure 4.15, which would indicate a moisture susceptible mix using the current ODOT criterion. The fourth project had one core with a high coating (95%) and the other core a low coating (60%). From looking at the figure it can be seen that the M_r test correctly identified four of the five projects for moisture susceptibility using a 70% ratio criterion. For this limited data, the resilient modulus ratio provides an improved indication of moisture susceptible mixtures over the IRS test.

For the lime-treated aggregate projects, the test results are similar. Displayed in Figure 4.16, the IRS value for the projects identified as

14



a) Overview

b) Close-up of Broken Cores

Figure 4.13. Plainview Road–Deschutes River.

9

15

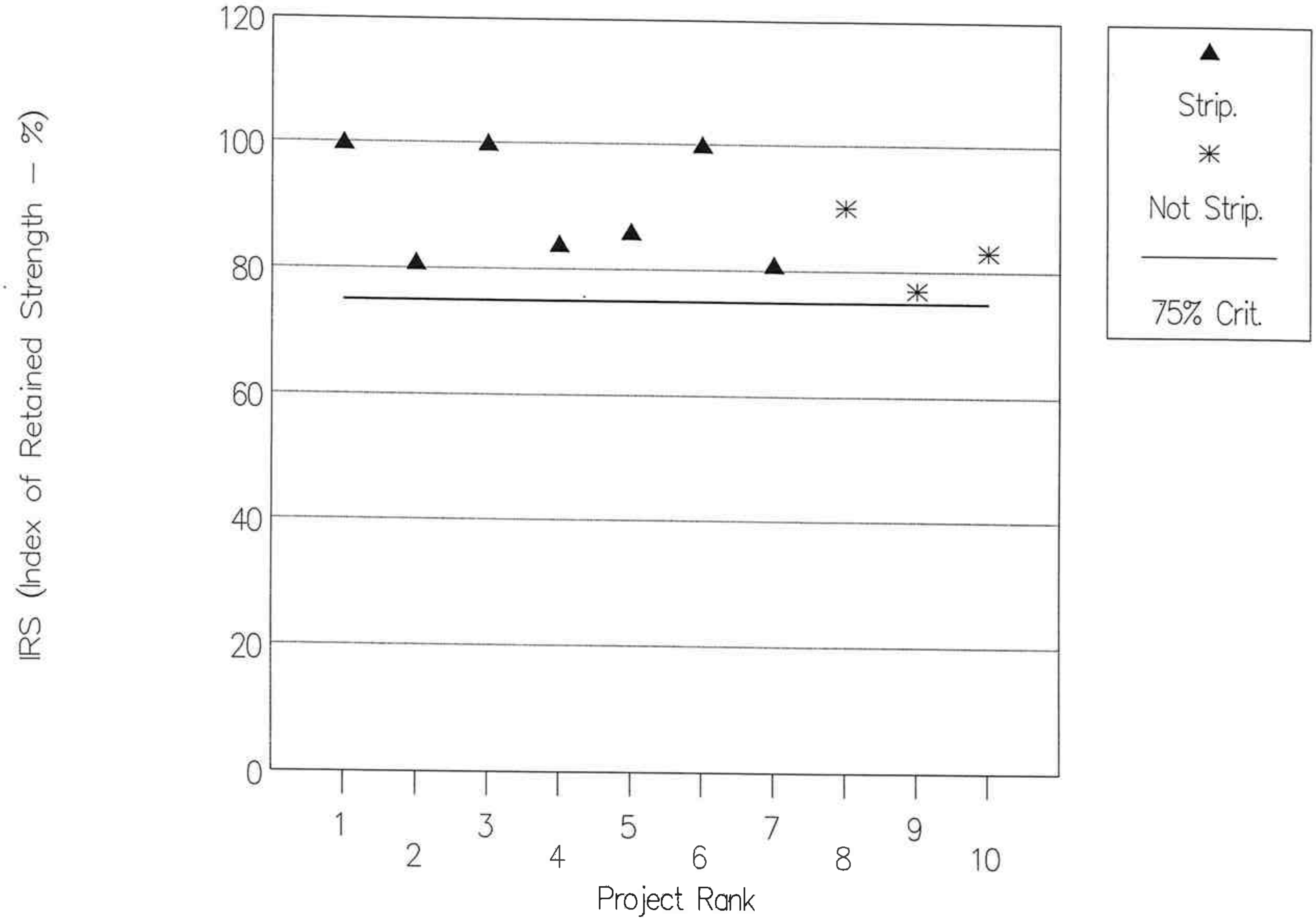


Figure 4.14. IRS Test Values for Projects Without Additives.

16

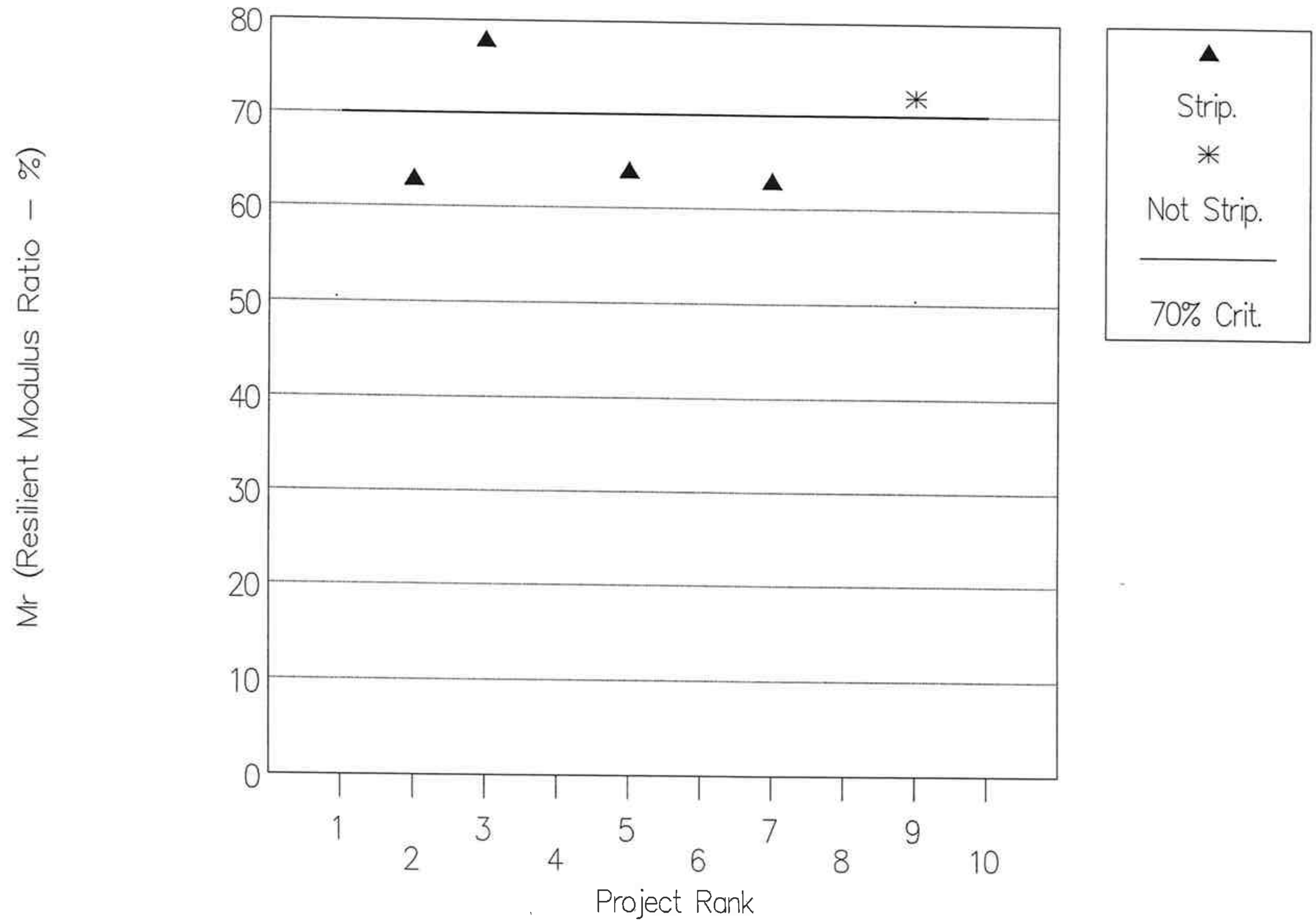


Figure 4.15. M_r Test Values for Projects Without Additives.

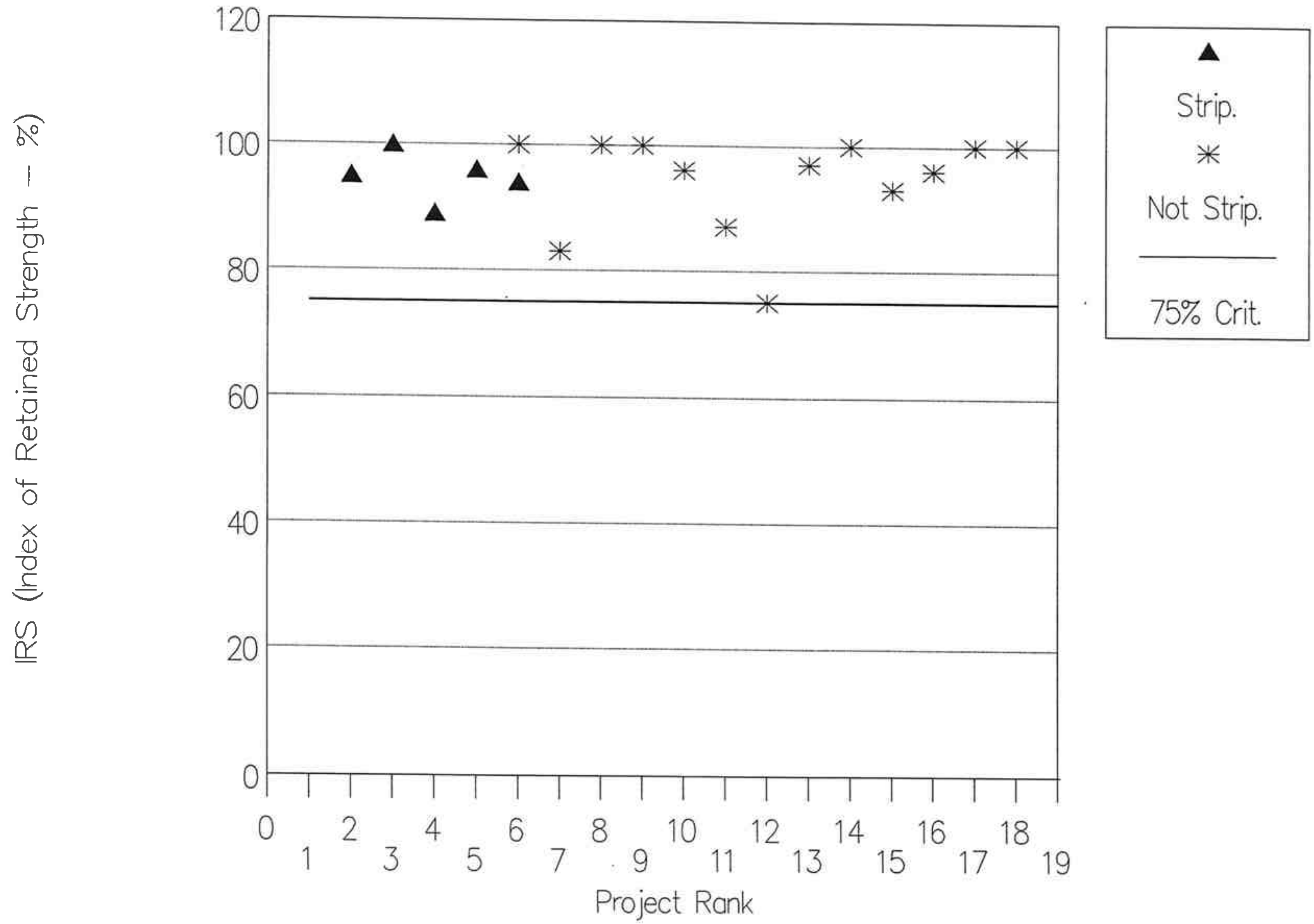


Figure 4.16. IRS Test Values for Projects With Lime-Treated Aggregate.

stripping varies from 89% to 100+%, well above the 75% minimum. Again, there is no distinction between the IRS mix design test values for projects classified as stripping versus nonstripping. A change in the percent coating criterion would not alter this assessment. The resilient modulus values, illustrated in Figure 4.17, indicate the moisture susceptibility of a pavement is difficult to predict for values between 0.70 and 0.75. Of the eight projects with M_r values in this range, three were identified as stripping. Only one project, with a resilient modulus ratio of 0.83, of the seven projects identified as stripping, had an M_r above 0.75. Some of the uncertainty in identifying moisture susceptible mixes for resilient modulus values between 0.70 and 0.75 can be attributed to inherent variation during the testing procedure and in the equipment.

An assessment of the test values for the projects with chemical additives is questionable with only three projects. The IRS values represent tests of mixes with the chemical additive in the asphalt. The M_r values are for samples without a chemical additive; therefore, an evaluation of stripping and the resilient modulus ratio cannot be made. For the 3 projects with chemical additives, the 2 projects identified as stripping have the highest and lowest IRS value. Even through the data is for only 3 projects, this limited data supports the findings for the projects without additives and the lime-treated projects concerning the validity of the IRS test.

4.6 Discussion

It is apparent that there is not a relationship between the pavement condition survey rating and the core analysis results with regard to identifying a project as stripping. This may, in part, be explained by the fact that only one set of cores were evaluated for each project site and, therefore, may or may not be representative samples. The core analysis does provide a direct evaluation of stripping within the pavement at the core location while the condition survey was used as an indirect measure of stripping. The lack of a correlation between the two analysis methods is not unexpected.

A review of the test procedures shows the resilient modulus test currently employed by ODOT provides a greater correlation with field performance than the IRS test. Based on the criterion for this report to determine if a project should be classified as stripping or nonstripping, the use of the IRS

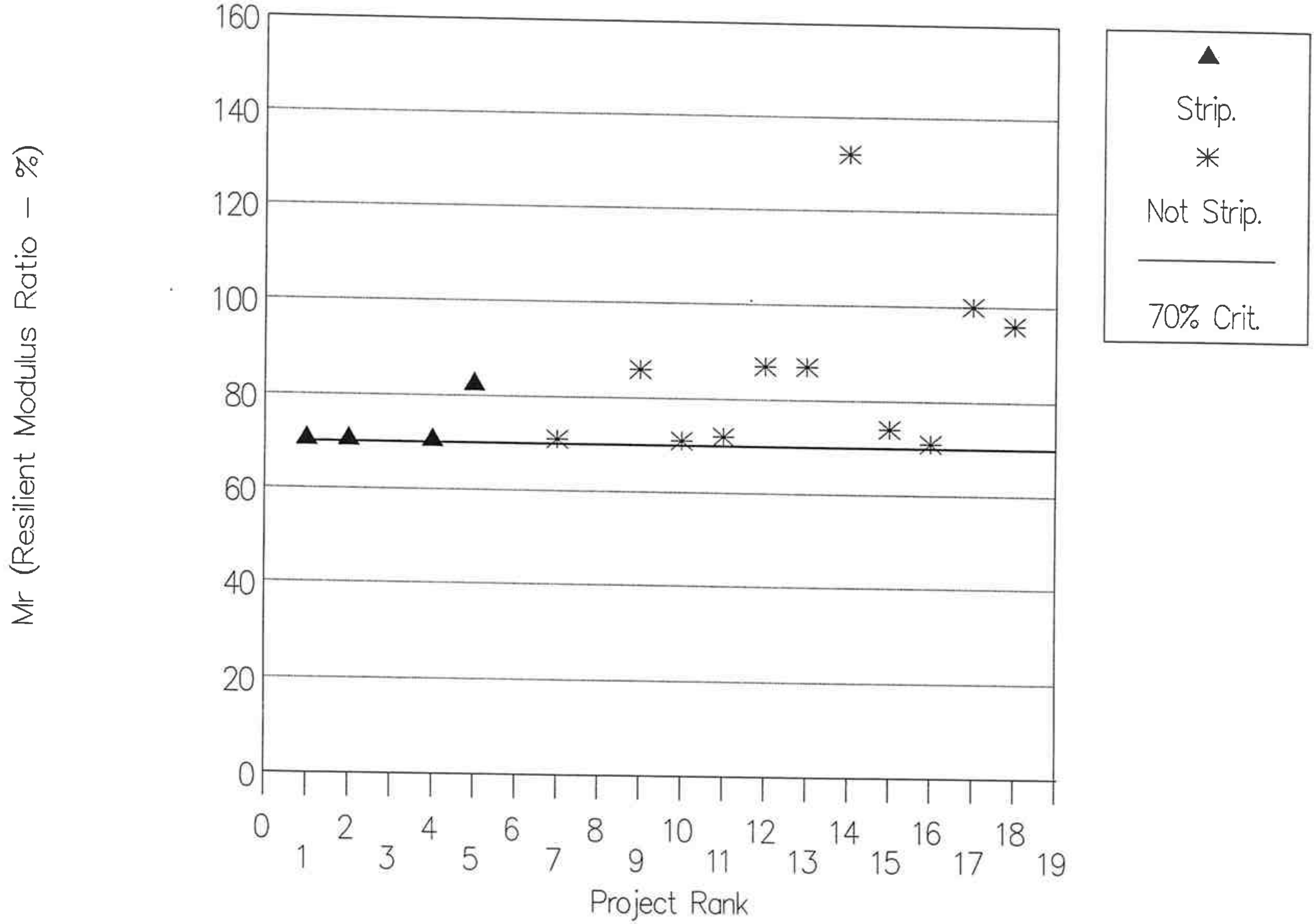


Figure 4.17. M_r Test Values for Projects With Lime-Treated Aggregate.

test does not provide a reliable indication of moisture susceptible asphalt concrete mixtures. These findings are based on a small database, therefore, efforts should be made to expand the database to develop an understanding of the relationship of the IRS and M_r test with field performance.

A statistical regression analysis was conducted and no strong correlations were evident. Using the data in Tables 4.2, 4.3, and 4.4, the analysis looked at age, air voids, project type, and test values for a possible correlation with percent coating determined from the core analysis.

The data in the tables was first evaluated as a group using age, air voids, IRS, and M_r as predictor variables for the percent coating of asphalt on the aggregate. Next, a regression analysis was done on the data categorized by the type (i.e., without, lime, chemical) treatment. In looking at the data from the projects by treatment, it is evident why the regression analysis did not yield a strong correlation for any of the predictor variables.

A comparison of the data in Table 4.2 for the projects without additive treatment illustrates large contrasts. For the two oldest projects, Baldock Slough to South Baker Interchange and Bend South City Limits to Murphy Road, both at 9 years, there is a significant difference in their asphalt coating rating. The Baldock Slough project had a retained coating of 40% for both cores while the Bend South City Limits project had both cores rated at 95%. The air voids data shows similar dichotomies, for example, one core for Baldock Slough had an air void content of 6.2% while one core for the Sherwood to Rex Hill project has an air void content of 6.0%, but the core rating is 90%. This analysis can be carried to the IRS test results. The IRS for the Baldock Slough project was 100% (core rating 40%) while the Burlington to Wilbridge project has the lowest IRS rating of 77% and a rating of 95% for both cores. When evaluating the IRS data there is evidence of contradiction even within projects. For instance, the IRS for the Moro to Grass Valley project was 100+% and the rating for the two cores showed one at 100% and the other at 70%.

The data in Table 4.3 for the lime-treated projects shows the same dissimilarities. For two of the oldest projects, McKay Reservoir to Mile Post 11.17 and Salt Creek Tunnel to Klamath County Line, the asphalt coating varies from 40% in both cores for the McKay Reservoir project to 100% in both cores for the Salt Creek Tunnel project. The air void content for two projects with cores rated at 95% varies from 1.1% for a Hermiston Highway to Washington State Line core to 11.1% for a North Powder to Haines core. A similar example for the IRS test as mentioned above can be found for this data set. The stockpiling times, which are unique to the lime-treated projects, have the same dichotomies as the other predictor variables. The McKay Reservoir project had the lime-treated aggregate stockpiled one week while the Salt Creek Tunnel project, with the higher coating (100% to 40%) was stockpiled approximately the same length of time. Furthermore, the Suttle Lake to Sisters project used lime-treated aggregate stockpiled approximately 1.5 weeks and had a rating of 40% for both cores while the North Santiam River to lava Lake Meadows Road project used treated aggregate stockpiled 40 weeks and both cores were rated at 90%.

Some of the differences mentioned above may be explained by other factors. However, there are enough of these differences throughout the data set between the different predictor variables that the differences could not be explained sufficiently by the other variables. These differences are also evident within the data for the projects with chemical-treated asphalt.

It is evident from reviewing the data that stripping is not sufficiently explained from the variables used in this report. From the data gathered, a strong correlation between air voids, age, and stripping was not evident. In addition, stockpiling times of lime-treated aggregate did not show a correlation between the length of time a lime-treated aggregate is stockpiled and the effectiveness of the treated aggregate in reducing the moisture susceptibility of the asphalt concrete mixture. Further information, such as accurately determining the core location (i.e., in a wheel track or on the fog line), the average daily traffic count for the site in question, and climatic conditions, may enhance a regression analysis to identify stripping in asphalt pavements.

4.7 Significant Findings

The significant findings from the field study are:

1) Lime treatment of aggregates appears to be an effective deterrent to stripping.
2) Lime treatment is more effective than no treatment or amine treatment.

3) Current methods of adding lime to the aggregate is effective in reducing moisture susceptibility.

4) Methods of lime addition can affect cost; however, their effect on pavement performance is not well defined.

5) Current practice of allowing stockpiled lime-treated aggregate to cure for as long as one year does not appear to inhibit the effectiveness of the lime treatment. However, stockpiling beyond a period of one year has resulted in severe cementing of the aggregate, a sign of carbonation.

6) Existing mix design criteria used to determine the need for antistripping additives appear to be effective on the basis of the condition survey. However, they may not be effective if the core analysis is a good indication of occurrence of stripping.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

A review of the literature and an analysis of the projects has led to the following conclusions. From the literature review:

1) Lime is more effective as an antistripping additive when thoroughly mixed with the aggregate in the presence of the proper amount of moisture.
2) Chemicals can be effective in reducing the moisture susceptibility of an asphalt mixture; however, the effectiveness is sensitive to the specific combination of asphalt, aggregate, and antistripping agent.
3) NCHRP 246 Indirect Tension Test and/or Modulus Test with Lottman Conditioning (resilient modulus ratio) provides a reasonable relationship with field data in predicting asphalt mixtures susceptible to moisture damage.

Three different categories of additive treatment were assessed in 32 projects. The results from the evaluation of these projects are:

4) The projects (up to nine years in age) which did not receive any additive treatment exhibit stripping-related distress in 70% of the projects.
5) The majority of the lime-treated aggregate projects are not showing signs of stripping related distress, and after four years, are in good condition.
6) The results for the three projects with amine-treated asphalt indicates that to determine the effectiveness of these additives requires further evaluation.
7) Procedures to document which design mixture is placed in the field are not clearly identified and/or consistently followed.
8) Procedures to document how long lime-treated aggregate is "stockpiled" are not clearly identified.
9) The IRS test does not appear to predict the moisture susceptibility of asphalt mixtures with acceptable accuracy.

10) A resilient modulus ratio of 0.70 or below appears to provide a better indication of moisture susceptible asphalt mixtures than the IRS (AASHTO T-165) test method.

5.2 Recommendations for Implementation

Based on the data in this report, the following actions are recommended:

1) Subject to the results of a concurrent study, it is recommended that the IRS testing and specification should be replaced by a specification for the resilient modulus test with a ratio of 0.70 as a minimum value.
2) The use of lime-treated aggregate should be maintained as a means of reducing the moisture susceptibility of an asphalt mix.
3) The current lime additive criteria should be expanded to include the possible use of lime when the M_r test falls below the minimum criterion. Operational considerations within the ODOT contract and construction process, however, may make this impractical unless the requirement for stockpiling is eliminated. Roads targeted with this criteria are primary roads (i.e. major highways such as 99W) not covered by the current ODOT criteria for the use of lime-treated aggregate.
4) Liquid chemical treatment of the asphalt should continue to be used with the following guidelines:
    a) Tests and specifications should be developed to assure adequate heat stability of chemical additives.
    b) Specifications should be written to limit the time that chemical additive and asphalt mix can be exposed to elevated temperatures.
    c) Chemical additives should be limited to secondary roads when the M_r is below the 0.70 value until further data on the effectiveness of the chemicals used can be established.
5) A coring plan should be developed and carried out to monitor stripping in projects throughout the state. The information should be used along with increased documentation of proce-

dures, from design through construction, to develop a database on stripping and the effectiveness of additives in reducing moisture damage.

5.3 Recommendations for Further Study

1) Conduct another survey of the same projects in two to three years to determine the level of stripping and the condition of the pavement structure.
2) Develop a test procedure to determine if lime-treated aggregate has been stockpiled too long allowing carbonation to occur.
3) Determine if carbonation of lime renders it ineffective for antistripping benefits.
4) Develop a program to analyze the cost effectiveness of using lime, chemicals, or polymers when a mixture is identified as being susceptible to moisture damage.
5) Develop tests for determining the amount of additive used in the field and improve documentation of amount of lime added and the method used.

6.0 REFERENCES

Al-Ohaly, A.A. and Terrel, R.L., "Effect of Microwave Heating on Adhesion and Moisture Damage of Asphalt Mixtures," presented at the Jan. 11-14, 1988, Transportation Research Board Meeting, Washington, D.C.

Brown, A.B., Sparks, J.W., and Marsh, G.E., "Objective Appraisal of Stripping of Asphalt from Aggregate," *STP No. 240*, ASTM, 1959, pp. 59-74.

Busching, H.W., Amirkhanian, S.N., Burati, J.L., Alewine, J.M., and Fletcher, M.O., "Effects of Selected Asphalts and Antistrip Additives on Tensile Strength of Laboratory-Compacted Marshall Specimens - A Moisture Susceptibility Study," *Proceedings*, Association of Asphalt Paving Technologists, vol. 55, 1986, pp. 120-148.

Coplantz, J.S., "Laboratory Test Methods to Predict Water Induced Damage to Asphalt Concrete", "*Masters Thesis*", submitted to the Graduate College of University of Nevada-Reno, 1987.

Coplantz, J.S., Epps, J.A., and Quilici, L., "Antistrip Additives: Background for a Field Performance," *TRR 1115*, Transportation Research Board, Washington, D.C., 1987, pp. 1-11.

Held, A., "Considerations of Aggregate Surface and Asphalt Cement Surface Chemistry Relative to Stripping of AC Pavements," Appendix to "Test Method to Determine the Degree of Asphalt Stripping from Aggregate," Kelly, P.B. et al., USDOT DTRS-57-85-C-00170, Mar. 1986, pp. B1-B8.

Hicks, R.G., et al, "Identification and Quantification of the Extent of Asphalt Stripping in Flexible Pavements in Oregon - Phase II," *FHWA-OR-85-3*, June 1985.

Hicks, R., Wilson, J.E., and Boyle, G.E., "Identification and Quantification of the Extent of Asphalt Stripping in Asphalt Pavements - Oregon Phase I," *FHWA-OR-83-3*, 1983.

Hinderman, W.L., "The Swing to Full-Depth ..., The Case for Laying Asphalt on the Raw Subgrade," *Information Series No. 146*, Asphalt Institute, College Park, MD, 1968.

Hubbard, P., "Adhesion in Bituminous Road Materials: A Survey of Present Knowledge," *Journal of the Institute of Petroleum*, vol. 44, no. 420, Dec. 1958, pp. 423-432.

Kennedy, T.W., "Use of Hydrated Lime in Asphalt Paving Mixtures," *Bulletin #325*, National Lime Association, March 1984.

Krebs, R.D. and Walker, R.D., *Highway Materials*, McGraw Hill Book Co., 1971.

Lottman, R.P., "Predicting Moisture-Induced Damage to Asphaltic Concrete - Field Evaluation," *NCHRP Report 246*, Transportation Research Board, Washington, D.C., May 1982.

Mack, C., "Physio-Chemical Aspects of Asphalt Pavements: Energy Relations at Interface Between Asphalt and Mineral Aggregate and Their Measurement," *Industrial and Engineering Chemistry*, Dec. 1935, pp. 1500-1505.

Majidzadeh, K. and F.N. Brovold, "State of the Art: Effect of Water on Bitumen-Aggregate Mixtures," HRB Special Report 98, Highway Research Board, 1968.

Maupin, G.W., "The Use of Antistripping Additives in Virginia," *Proceedings*, AADT, Vol. 51, 1982, pp. 342-362.

Mertson, E.W., and Wright, J.R., "Cationic Asphalt Emulsions: How They Differ from Conventional Emulsions in Theory and Practice," *Proceedings*, Highway Research Board, 1959, pp. 386-397.

Petersen, J.C., Plancher, H., and Harnsberger, P.M., "Lime Treatment of Asphalt - Final Report," National Lime Association, 1987.

Plancher, H., Dorrence, S.M., and Peterson, F.C., "Identification of Chemical Types in Asphalts Strongly Absorbed at the Asphalt-Aggregate Interface and Their Relative Displacement by Water," *Proceedings*, Association of Asphalt Paving Technologists, vol. 46, 1977, pp. 151-175.

Quinn, W.A., et al, "Mix Design Procedures and Guidelines for 1. Asphalt Concrete 2. Cement Treated Base 3. Portland Cement Concrete," Oregon State Highway Division, April 1987.

Rice, J.M., "Relationship of Aggregate Characteristics to the Effect of Water on Bituminous Paving Mixtures," *ASTM STP 240*, American Society for Testing and Materials, 1958, pp. 17-33.

Schmidt, K.J. and Graf, P.D., "The Effect of Water on the Resilient Modulus of Asphalt Treated Mixes," *Proceedings*, AADT, Vol. 41, 1972.

Stroup-Gardiner, M. and Epps, J.A., "Four Variables That Affect Lime in Asphalt-Aggregate Mixtures," *TRR 1115*, Transportation Research Board, Washington, D.C., 1987, pp. 12-22.

Takallou, H., "Stripping of Asphalt Pavements: State of the Art," Transportation Engineering Report 84-6, Oregon State University, 1984.

Takallou, H.T., Hicks, R.G., and Wilson, J.E., "Evaluation of Stripping Problems in Oregon," *ASTM STP 899*, American Society for Testing and Materials, 1985, pp. 22-48.

Thelen, E., "Surface Energy and Adhesion Properties in Asphalt-Aggregate Systems," Highway Research Board Bulletin 192, 1958.

Tunnicliff, D.G. and Root, R.E., "Use of Antistripping Additives in Asphaltic concrete Mixtures," *NCHRP Report 274*, Transportation Research Board, Washington, D.C., Dec. 1984.

Tunnicliff, D.G. and Root, R.E., "Introduction of Lime Into Asphalt Concrete," *FHWA/RD-86/071*, 1986.

Yoon, H.H. and Tarrer, A.R., "Effect of Aggregate Properties on Stripping," Presented at the Jan. 11-14, 1988, Transportation Research Board Meeting held at Washington, D.C. (Preprint 870077)

Yoon, H.H. Tarrer, A.R., Roberts, F.L., and Kiggundu, B.M., "Thermal Degradation Properties of Antistripping Additives and Enhanced Performance by Curing," Presented at the Jan., 1989, Transportation Research Board Meeting held at Washington, D.C.

Zeisman, W.A., "Influence of Constitution on Adhesion," *Industrial and Engineering Chemistry*, vol. 55, no. 10, Oct. 1963, pp. 18-38.

APPENDIX

PAVEMENT CONDITION SURVEY

AND

CORE ANALYSIS RESULTS

LIST OF TABLES

Overview

This appendix contains the complete results from the pavement condition survey and the core analysis. Tables 1.0, 2.0, and 3.0 represent the consensus of the rates for each project. The entries correspond to the form shown in Figure 4.2 in the body of the report. Table 4.0 contains the core analysis for each project. The core number and location were assigned by the region obtaining the core sample and therefore have no significance other than for identification by each region. Presented along with the asphalt coating rating is an assessment of the thickness of the asphalt on the aggregate. The core analysis procedure is outlined in Figure 4.3 in the report.

Table 1.0. Condition Survey Results for Projects without Additive Treatment

Year Constructed	Project Name	Mile Post No.	Raveling (% area)	Rut Depth (in.)	Allig. Cracking (% area)	Long. Cracking (% len.)	Transverse Cracking (space ft.)	Maint. Patch (% area)	Overall Eval.	Comments
1979	Baldock Slough-South Baker Interchange	304 EB 305.8 WB	Mod. Mod.	1/4 1/8	15 25	* 10*	10 10	55* 100	Poor Poor	Thermal cracks mostly in shoulders; MP 304 - both lanes have been chip sealed; longitudinal cracking, IWP 50%, OWP 5%; MP 305.8 WB - inside lane has chip seal, outside pitted.
1979	Bend S City Limits-Murphy Rd	Jake's Tk Stop Larry's Carpet	Slight Slight	1/2 1/2	10 10	10 10	100 100	None 10	Avg. Avg.	Larry's Carpet - thermal cracking; Jake's Tk Stop - longitudinal cracks in OWP in both lanes.
1984	Port of Moro Interchange	Exist Ramp WB	Slight	3/8	None	None	150	Avg.	Good*	Exit Ramp - closed surface texture
		On Ramp WB	Slight	1/4	< 5	None	None	Avg.	Good	and excessive asphalt.
1984	Rickreall-Suver Junction	62 60	Slight Slight	1/8 1/8	None None	None None	None None	None None	Good* Good	Some studded tire wear.
1984	Moro-Grass Valley	24.9 21	Slight Slight	1/4 1/4	< 5 None	None None	None None	None None	Avg.* Avg.*	Project stripping; 100% seal; MP 21.0 - potholes started; closed surface texture and excessive asphalt.
1985	Monroe-Crow Creek	103 101.8	None None	1/8 1/8	None None	None None	None None	None None	Good Good	
1985	Rondo-Blossom Lane	8 -	Slight -	1/8 -	None -	None -	None -	None -	Good -	
1986	Sherwood-Rex Hill	Elks Sign Xmas Shop Sign	Mod. Mod.	5/16 1/8	None None	None None	None None	None None	Avg. Avg.*	Xmas Shop Sign - raveling is more severe in outer lane - south bound.
1986	Burlington-Willbridge	11 WB 9 EB	Slight* Slight	1/16 1/4*	None None	None None	None None	None None	Good Good	MP 11 WB - inner and outer wheel paths of truck lane; MP 9 EB - rutting in outer lane.
1986	East Stayton-Mehama	18 21	Slight Slight	1/4 3/16	None None	None None	None None	None None	Good* Good*	MP 18 - flushing in wheel paths beginning; MP 21 - no flushing.

*See comments

Table 2.0. Condition Survey Results for Projects with Lime-Treated Aggregate

Year Constructed	Project Name	Mile Post No.	Raveling (% area)	Rut Depth (in.)	Allig. Cracking (% area)	Long. Cracking (% len.)	Transverse Cracking (space ft.)	Maint. Patch (% area)	Overall Eval.	Comments
1984	McKay Reservoir-MP 11.17	10 9	Slight Slight	1/8 1/16	None None	None None	* *	None None	Avg. Avg.	Thermal cracking just beginning; some pitting; closed surface texture and excessive asphalt @ MP 10
1984	Marion Co. Line-Bugaboo Rd	61.3 67.7	Mod. Slight	1/4 1/4	40 50	20 35	None None	45 None	Poor Poor	Rutting deeper in failed areas; extensive patching; MP 61.3 more severe than MP 67.7.
1984	Salt Creek Tunnel-Klamath Co. Line	61.2 59	None None	1/8 1/16	None None	None None	None None	None None	Avg.* Avg.*	Closed surface texture and excessive asphalt.
1985	Hermiston Hwy-Washington State Line	194 196	None None	1/16 1/8	None None	None None	None None	None None	Good* Good	Some flushing in wheel paths
1985	Lava Butte-Fremont Hwy	152 163	None None	1/4 1/8	None None	None None	100* None*	None None	Good Good	Thermal cracking @ MP 152 - 100 ft; @ MP 163 - one crack.
1985	Tower Rd Int-Stanfield Int.	167 160	Slight Slight	1/4 3/8	None None	None None	100 100	None None	Good Good	Thermal cracks part way across travel line @ 100 ft spacing; MP 160 - thermal cracks in shoulder.
1985	La Grande S City Limits-Hot Lake/Apt. Rd	6.6 9	Slight Slight	1/8 1/8	None None	None None	None None	None None	Good Good	
1985	Suttle Lake-Sisters	90.2 96	None None	1/16 1/16	None None	None None	None None	None None	Good Good	
1985	Santiam River-Lava Lake Meadow Rd	76 76.8	Slight Slight	1/4 1/8	None None	None* None	None None	None None	Good Good	@ MP 76 - slight longitudinal cracking @ meet line.
1985	Haines-Pochontas Rd	42 46.5	Slight Slight	1/16 1/8	3 3	5 5	None* 75	2 2	Good Good	Studded tire wear in wheel paths.
1986	Hermiston-Stanfield	8.8 6.8	Slight Slight	- 3/16	None None	None None	None None	None None	Good Good	
1986	Minam-Spring Creek	37 42	Slight Slight	1/16 1/32	None None	None None	* *	None None	Good Good	@ MP 37 - two isolated thermal cracks.
1986	Meacham-Hilgard	249 243	Slight* Slight*	1/8 1/8	None None	None None	None 40	None None	Good Good	Some pitting.
1986	North Powder-Haines	36 38	None None	None None	None None	None None	50* 150	None None	Good Good	Closed surface texture and excessive asphalt.
1986	Sandy River-Corbett Interchange	21 19	Slight Slight	1/8 1/8	None None	None None	None None	None None	Good Good	
1986	Eagle Creek-Salt Creek Tunnel	55.1 54	Slight Slight	1/8 1/16	None None	None None	None None	None None	Good Good	

Table 2.0. Condition Survey Results for Projects with Lime-Treated Aggregate (cont.)

Year Constructed	Project Name	Mile Post No.	Raveling (% area)	Rut Depth (in.)	Allig. Cracking (% area)	Long. Cracking (% len.)	Transverse Cracking (space ft.)	Maint. Patch (% area)	Overall Eval.	Comments
1986	McNary Hwy-Umatilla	185.3	None	1/8	None	None	None	None	Good	
		184.7	None	1/8	None	None	None	None	Good	
1987	Linn Co Line-Suttle Lake	81.9	None	1/16	None	None	None	None	Good	
		84.2	None	1/16	None	None	None	None	Good	
1987	Irrigon Jct-First St	175	Slight	1/8	None	None	None	None	Good	
		172	Slight	1/8	None	None	None	None	Good	

*See comments

Table 3.0. Condition Survey Results for Projects with Amine-Treated Asphalt

Year Constructed	Project Name	Mile Post No.	Raveling (% area)	Rut Depth (in.)	Allig. Cracking (% area)	Long. Cracking (% len.)	Transverse Cracking (space ft.)	Maint. Patch (% area)	Overall Eval.	Comments
1980	Plainview Rd-Deschutes River	9 11	Slight Slight	1/4 3/8	5 5	15 15	100 100	None None	Good* Good*	Studded tire wear; premature load and thermal distress; longitudinal cracks along meet and fog lines.
1984	Sandy River-Mitchell Point	57 51	Mod. Mod.	1/2 1/16	None None	None None	None None	None None	Avg.* Good	Frequent overlays on project (1000 ft); open texture with considerable rock loss.
1984	South Baker-Durkee	315 324	Slight Slight	1/2 1/2	None None	None None	None 50	None None	Avg.* Avg.*	MP 324 - no thermal cracks; outside wheel path flushed; MP 315 EB - closed surface texture and excessive asphalt in OWP; - IWP overall evaluation is good.

*See comments

Table 4.0. Core Analysis Data.

Method of Treatment	Project Name	Core Number and Location	Specific Gravity Bulk	Specific Gravity Rice	% Air Voids	Coating (Visual Rating %)		Comments
None	Baldock Slough-S	18 - WT	2.25	2.399	6.2	40*	D	*Microwaved for 2 min
	Baker Interchange	18 - FL	2.19	2.404	8.9	40	D	to separate top lift
None	Bend S City Limits-	Larry's Carpet WT	2.42	-	-	95	D+	
	Murphy Rd	Larry's Carpet FL	2.41	2.460	2.0	95	D+	
None	Port of Morrow	6 - WT	2.42	-	-	95	S+	
	Interchange	6 - FL	2.40	2.552	6.0	95	S	
None	Rickreall-Suver Jct	1 - 03431 WT	2.29	2.438	6.0	60	S	
		2 - 03431 FL	2.33	2.436	4.4	95	S+	
None	Moro-Grass Valley	5 - WT	2.55	2.584	1.3	100	T	This project has been
		5 - FL	2.54	2.594	2.1	70	S+	chip sealed
None	Monroe-Crow Creek	1 - 03434	2.25	2.453	8.3	50	D	
		2 - 03434	2.25	2.437	7.7	60	D	
None	Rondo-Blossom Lane	1 - 03400	2.27	2.439	6.9	75	S-	
		2 - 03433	2.25	2.440	7.8	85	D	
None	Sherwood-Rex Hill	1 - WT	2.35	2.501	6.0	90	S	Lots of uncoated
		1 - FL	2.33	2.516	7.4	70	D	fines and broken aggregate
None	Burlington-Wilbridge	4 - WT	2.34	2.412	3.0	95	S+	
		4 - FL	2.31	2.393	3.5	95	S+	
None	E Stayton-Mehama	1 - 03435 WT	2.31	2.419	4.5	90	S+	MP 18.00
		1 - 03435 FL	2.26	2.432	7.1	70	S	MP 18.00
Lime	McKay Reservoir-	7 - WT	2.35	2.556	8.1	40	D	Sample arrived broken
	MP 11.17	7 - FL	2.37	2.534	6.5	40	D	
Lime	Marion Co Line-	1 - 08432	2.33	2.509	7.1	85	S-	
	Bugaboo Rd	2 - 08432	2.28	2.497	8.7	95	S	Cinders in top 1 in.
Lime	Salt Creek Tunnel-	MP 58.9 WT	2.27	-	-	100	T	
	Klamath Co Line	MP 58.9 FL	2.28	2.335	4.5	100	T	
Lime	Hermiston Hwy-Washington	3 - WT	2.55	2.578	1.1	95	S+	
	State Line	3 - FL	2.50	-	-	90	S	
Lime	Lava Butte Rd-	WT	2.39	-	-	100	S+	
	Fremont Jct.	FL	2.38	2.465	3.4	100	S+	
Lime	Tower Int.-	1 - WT	2.42	-	-	95	S	
	Stanfield Int.	1 - FL	2.43	2.567	5.3	95	S	
Lime	La Grande S City Limits-	10 - Hwy 203 WT	2.25	2.410	7.0	80	S	
	Hot Lakes/Apt. Rd	10 - Hwy 203 FL	2.30	2.412	4.6	75	S	
Lime	Suttle Lake-Sisters	MP 96 WT	2.31	2.474	6.6	40	S	Sanding M+C embedded
		MP 96 FL	2.32	2.469	6.0	40	S	in top 1/2 in.
Lime	Santiam River-Lava Lake	1 - 03531 WT	2.34	-	-	90	S	Sanding M+C embedded
	Meadow Rd	2 - 03536 FL	2.36	2.480	4.8	90	S	in top 1/2 in.
Lime	Haines-Pocohontas Rd	12 - MP 46 WT	2.28	-	-	100*	S+	*Microwaved for 2 min
		12 - MP 46 FL	2.28	2.363	3.5	90	S	to separate top lift
Lime	Hermiston-Stanfield	5 - WT, MP 8.8	2.39	-	-	80	S	
		5 - FL, MP 6.8	2.36	2.550	7.5	90	D	
Lime	Minam-Spring Creek	9 - Hwy 82 WT	2.50	-	-	95	S+	
		9 - Hwy 82 FL	2.50	2.594	3.6	95	S+	

Table 4.0. Core Analysis Data (continued).

Method of Treatment	Project Name	Core Number and Location	Specific Gravity Bulk	Specific Gravity Rice	% Air Voids	Coating (Visual Rating %)		Comments
Lime	Mecham-Hilgard	8 - WT	2.30	2.369	2.9	90	S+	
		8 - FL	2.29	2.362	3.0	85	S	
Lime	North Powder-Haines	11 - Hwy 66 WT	2.10	-	-	95*	S+	*Microwaved for 2 min
		11 - Hwy 66 FL	2.09	2.351	11.1	95	S+	to separate top lift
Lime	Sandy River-Corbett	3 - WT	2.38	-	-	90	S+	
		3 - FL	2.42	2.485	2.6	90	S	
Lime	Eagle Creek-Salt Creek Tunnel	MP 55.1 - WT	2.38	2.522	5.6	70	S	Sanding materials in
		MP 55.1 - FL	2.36	2.518	6.3	70	S	areas down to 3/4 in.
Lime	McNary Hwy-Umatilla Ave	4 - Hwy 730 WT	2.49	-	-	95	S	
		4 - Hwy 730 FL	2.46	2.516	2.2	95	S	
Lime	Linn Co Line-Suttle Lake	MP 81.9 - WT	2.30	-	-	95	S	Sanding materials in
		MP 81.9 - FL	2.27	2.458	7.6	95	S	cores down to 1/4 to 1/2 in.
Lime	Irrigon Jct-First St.	2 - WT	2.52	-	-	95	S	
		2 - FL	2.55	2.591	1.6	95	S	
Amine	Plainview Rd-Deschutes River	MP 11 - WT	2.31	2.504	7.7	40	D	
		MP 11 - FL	2.33	2.504	6.9	40	D	
Amine	Sandy River-Mitchell Point	2 - WT	2.43	2.479	2.0	90	S	
		2 - FL	2.38	2.477	3.9	60	D	
Amine	S Baker-Durkee	14 MP 315 EB WT	2.30	-	-	100*	T	*Microwaved for 2 min
		14 MP 324 WB FL	2.27	2.336	2.8	100	T	to separate top lift

Location: WT - Wheel Track, FL - Fog Line

Visual Rating: D - Dry, S - Sufficient, T - Thick