FINAL REPORT

EVALUATION OF POROUS PAVEMENTS USED IN OREGON

Volume I

SPR 5298

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16. Abstract

Porous pavements or open-graded asphalt mixtures have been in use in Oregon since the late 1960s. The use of this pavement type has increased over the years because the pores in the mat provide an efficient way for water to drain from the pavement surface. This greatly increases safety in the areas of skid resistance and splash and spray. An added benefit from these pavements is that tire noise is partly absorbed into the voids of the pavement.

The purpose of this study was to evaluate porous pavements, especially the F-mix, as they are used in Oregon. The input from inside (i.e., contractors, ODOT personnel, asphalt experts) and outside (i.e., literature published over the years from agencies in the U.S. and abroad) Oregon was used to study open-graded mixes. This information was then used for improving porous pavements in Oregon.

Laboratory and field studies were performed on Oregon's open-graded mixtures. These tests were designed to understand how the mixture types performed with Oregon's conditions. These tests included texture depth, permeability, accident analysis, skid testing, rutting, splash and spray, noise, core gradation, asphalt properties, and tack coat shear testing.

A number of findings resulted from this study. Porous pavements provide a 1-2 dB A-weighted roadside noise improvement when compared to B-mix pavements. This difference would not be perceptible to an individual with average hearing. However, noticeable improvements for the F-mix did occur in the 500-4000 Hz range. Splash and spray visibility is improved, and safety on the roadway is improved. Potential problems with porous pavements include post-construction skid resistance, construction difficulties, and clogging of the pavement mat. Suggestions have been made in this study in terms of solving these problems and increasing the benefits.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

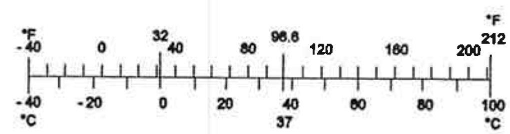
Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	millimeters squared	mm^2
ft^2	square feet	0.093	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometers squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9	Celsius temperature	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometers squared	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celsius temperature	1.8 + 32	Fahrenheit	°F



* SI is the symbol for the International System of Measurement

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EVALUATION OF POROUS PAVEMENTS USED IN OREGON

1.0 INTRODUCTION

1.1 Background

Oregon began experimenting with an open-graded-type asphalt concrete in the 1930's. Early tests showed high skid resistance and decreased glare from headlights. Due to the noticed advantages, this pavement type developed into the plant-mix seal coat that saw substantial use by Western states in the late 1940's and 1950's (Copas et al., 1978). The late 1970's saw the change in open-graded mixtures that started Oregon toward the designation of their E- and F-mixes (Huddleston et al., 1993).

These mixtures were originally developed as a friction coarse, but they have also proven to reduce noise, splash and spray, and rutting. These benefits, along with some problems such as reduced durability and increased winter maintenance, have made it necessary to improve the quality of the mixes used in porous pavements. To facilitate this improvement, there is a need to quantify the improved safety as well as monitor the change in mixture properties (e.g. permeability, voids, etc.) over time. Finally, there is a need to evaluate the feasibility of placing porous pavements on both old and new portland cement concrete.

1.2 Project Objectives

The overall objective of this study is to develop improved guidelines for use of porous pavements in Oregon. Specific objectives are as follows:

1) Documentation of the advantages and disadvantages of porous pavements in the areas of safety, environmental, and performance;

2) Evaluation of mix properties over time (e.g. permeability/voids, surface friction, splash and spray, noise);

3) Recommendation of modifications to existing specifications as needed (e.g. moisture content, IRS, ECS, etc.); and

4) Development of guidelines for considering environment, pavement type and traffic, as well as for long-term maintenance of porous pavements.

1.3 Organization of Report

The study consisted of the following six tasks:

Task 1: Literature Review/Questionnaire Survey

The results of the literature review and the questionnaire survey are given in a separate report (Younger et al., 1994). Chapter 2 of this report summarizes those findings, plus new findings since January 1994.

Task 2: Field Evaluation of Porous Pavements Used in Oregon

The field evaluation portion of this project is discussed in Chapter 3 of this report. The field evaluation covered topics such as texture depth, permeability, splash and spray, accident surveys, and skid testing.

Additional testing included an evaluation of the noise properties of porous pavements as compared to dense-graded pavement types and portland cement concrete (PCC) pavements. These are presented in Chapter 5.

Task 3: Laboratory Evaluation

The laboratory evaluation for this project was performed on field cores taken from the field evaluation sites. The cores were tested for permeability, moisture sensitivity using the Environment Conditioning System (ECS), gradation, and asphalt properties. Chapter 4 discusses these results.

Task 4: Analysis of Data

Data analysis is discussed throughout the report. A summary of all findings is presented in Chapter 6.

Task 5: Field Study

An F-mix pavement was placed over a PCC pavement on Interstate 5 north of Grants Pass, Oregon, under the Oregon Department of Transportation (ODOT) project name of Azalea to Jumpoff Joe. This portion of the Azalea to Jumpoff Joe project was completed in September of 1994. The overlay was completed without major problems, but long term study of the project performance was not possible. The tack coat study portion of this report and Appendix F have sections pertaining to this task.

Task 6: Reports

The literature review/questionnaire survey contained in the interim report (Younger et al., 1994) and this report complete this task. This report is the culmination of all of the research completed in this project.

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2.0 LITERATURE REVIEW AND QUESTIONNAIRE SURVEY

A literature review and questionnaire survey were completed as a separate report (Younger et al., 1994). The literature review consisted of an evaluation of information in both the United States and abroad.

The questionnaire was a survey of the porous pavement users in Oregon. The three survey groups were the ODOT project managers, the ODOT district maintenance managers, and asphalt contractors in Oregon.

2.1 History of Use

2.1.1 Oregon

Oregon began serious use of F-mix pavements in the late 1970's (Huddleston et. al., 1993). The success of these projects has made this pavement type popular with ODOT. Currently, Oregon has placed approximately 1820 (2930 km) center-line miles of F-mix along its highway system. F-mixes are a popular pavement surface along Oregon's Interstate system. (Appendix A provides a complete listing of all Oregon F-mix jobs, and Appendix B presents specifications for F-mixes used in Oregon.)

Table 2.1 compares the broadband limit gradation for the Oregon F-mix with a number of open-graded pavement types used elsewhere. These are also compared to Oregon's dense-graded B-mix gradation and the Oregon open-graded emulsion mix (OGEM) gradation. The table shows the higher percent of larger aggregate used for open graded mixtures.

The Oregon F-mix void contents and placement depth seem to match most closely those techniques used in European countries (Smith, 1992). The 15 to 20% void content and 2 in (50 cm)

Table 2.1. Broadband limit gradations

Oregon Mixtures				
Sieve Size	ODOT B-mix (ODOT, 1991)	ODOT E-mix (ODOT, 1991)	ODOT F-mix (ODOT, 1991)	ODOT OGEM (ODOT, 1991)
1 in (25 mm)	99-100	-	99-100	-
3/4 in (19 mm)	92-100	99-100	85-96	95-100
1/2 in (13 mm)	75-91	90-98	55-71	70-90
1/4 in (6 mm)	50-70	25-40	15-30	15-43
No. 10 (2 mm)	21-41	2-12	5-15	-
No. 40 (1 mm)	6-24	-	-	0-7
No. 200 (0.075 mm)	2-6	1-5	1-6	0-2

Open-Graded Asphalt Mixtures Used Elsewhere					
Sieve Size	FHWA (Proposed)	United Kingdom (1993)	Australia (Booth et al., 1991)	France (1991)	Spain (Ruiz et al., 1990)
1 in (25 mm)	100	-	-	-	-
3/4 in (19 mm)	86-96	100	-	-	100
1/2 in (13 mm)	60-70	55-75	100	100	75-100
1/4 in (6 mm)	10-20	20-30	29	10	32-50
No. 10 (2 mm)	4-10	7-13	10	10	10-22
No. 40 (1 mm)	-	-	-	-	5-12
No. 200 (0.075 mm)	0-4	3.5-5.5	0-4	0-5	3-6

placement depth seem to provide the best characteristics for splash and spray and noise reduction, while still maintaining strength and usefulness.

ODOT's estimates for the cost of F- and B-mixes for a 2 in (50 mm) deep square yard (0.84 m^2) section are about \$3.21 for F-mix and \$2.83 for B-mix. This estimate assumes a modified binder with high AC content in the F-mix.

2.1.2 Other States

Porous pavements evolved from the early efforts to improve pavement friction through the application of uniformly graded aggregate of about one-half inch (12 mm) nominal size, over a layer of asphalt concrete (Smith, 1992). This treatment first became known as the plant mix seal coat. These mixes posed a problem due to the aggregate coming loose from the pavement surface. Agencies then began developing a pavement to reduce the problems by increasing the asphalt content and changing the gradation to include some smaller sized aggregate. Through trial and error methods, the present open-graded friction course (OGFC) was developed, and there are still many different mixture design methods.

A 1991 study in Arizona (Hossain et al, 1991) was performed to analyze the use of a full-depth open-graded pavement. This pavement was not the normal partial-depth design, where the water drains laterally through the voids, but a pavement designed for the water to drain vertically down through the pavement structure. The pavement was designed in such a manner as to allow the water to drain for a 10-year 24-hour storm.

2.1.3 Overseas (Europe, S. Africa)

European countries including Belgium, France, Italy, Netherlands, Spain, Switzerland, and United Kingdom frequently use porous pavements as a surface course (Smith, 1990). Usage of this mixture type has even ranged as far away as South Africa. These countries normally use a mix of more than 20 percent air voids and 40 mm to 50 mm (1.5 to 2 in). These mixes are generally of a void content and thickness greater than used in most US states, excepting Oregon. Reports of use in these countries are favorable, and use continues. A main reason for this is the noise-reducing properties of porous pavements.

2.2 Advantages

A summary of the porous pavement mixture benefits, which have resulted in widespread use in Oregon, is presented in Table 2.2. Open-graded mixes seem to have a number of advantages that make it a viable mixture option. One of the main advantages of open-graded mixtures is improved safety. This improvement comes in a number of areas: decreased highway glare, improved skid resistance, reduced noise, reduced splash and spray, and reduced hydroplaning potential (Smith, 1992).

Skid resistance on open-graded mixes is most beneficial under rainy conditions. Porous mixes seem to maintain their skid properties during wet conditions and at higher speeds better than other mix types. An interesting example of improved skid resistance on Oregon highways is shown in Figure 2.1 (Huddleston et al., 1993). The only disconcerting thing about the skid resistance of porous pavements is that problems have been reported for newly constructed pavements (Booth, 1991). A good example of this phenomena is presented in Figure 2.2 (ODOT, 1993). This study, by ODOT, seems to show that the skid numbers for F-mixes, are lower than B-mixes of the same age. This phenomena was also studied in the Netherlands, due to a fatal accident (Deuss, 1994). The results of this latter study presented a 25 to 30% decrease in skid resistance on new porous mixes as compared to new dense mixes.

A study performed in the United Kingdom (UK) by the Transportation Road Research Laboratory (TRRL) quantified the splash and spray characteristics of porous pavements (Nelson, 1990). TRRL developed an electronic device which measures light backscatter from a laser

Table 2.2. Advantages of porous pavements

Advantage	Benefits	Sources of Information
Skid Resistance	In rainy pavement situations, high speed skid resistance is retained more than dense mixes.	Huddleston et al., 1993; Booth, 1992; Isenring et al., 1993
Splash and Spray	Reduced visibility reduction from tire spray.	Nelson et al., 1990
Noise Reduction	Reduced pavement noise.	Copas et al., 1978; Horak et al., 1994; Polcak, 1990; Nelson et. al., 1990
Hydroplaning	Chance of hydroplaning is reduced because water does not stay on surface.	Copas et al., 1978; Isenring et al., 1990
Rutting	High aggregate interlock reduces rutting potential.	Smith, 1992
Glare Reduction	Night time pavement glare is reduced for improved safety.	Huddleston et al., 1993; Colwill et al., 1993

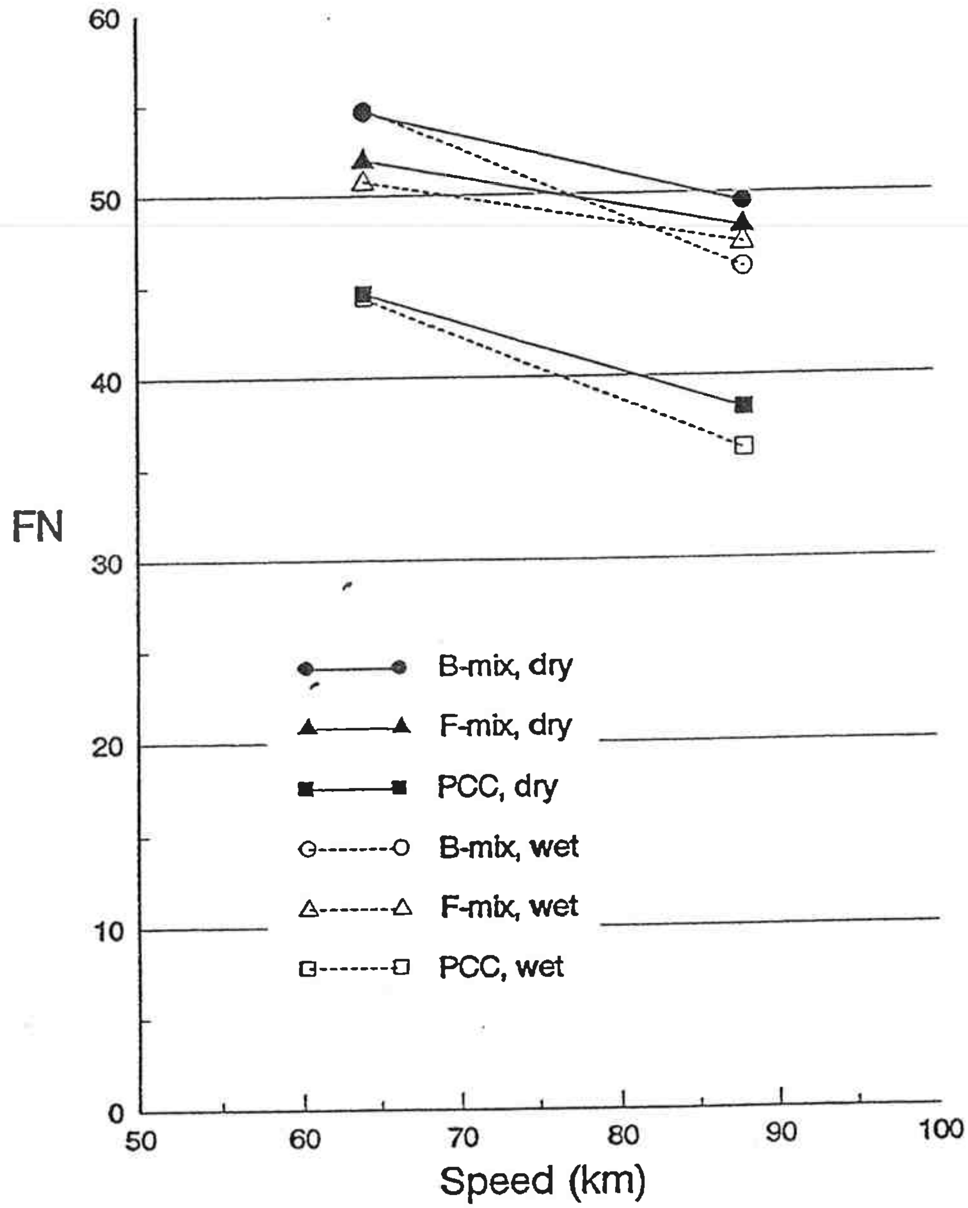

Figure 2.1. Effect of speed on frictional values (after Huddleston, 1993)

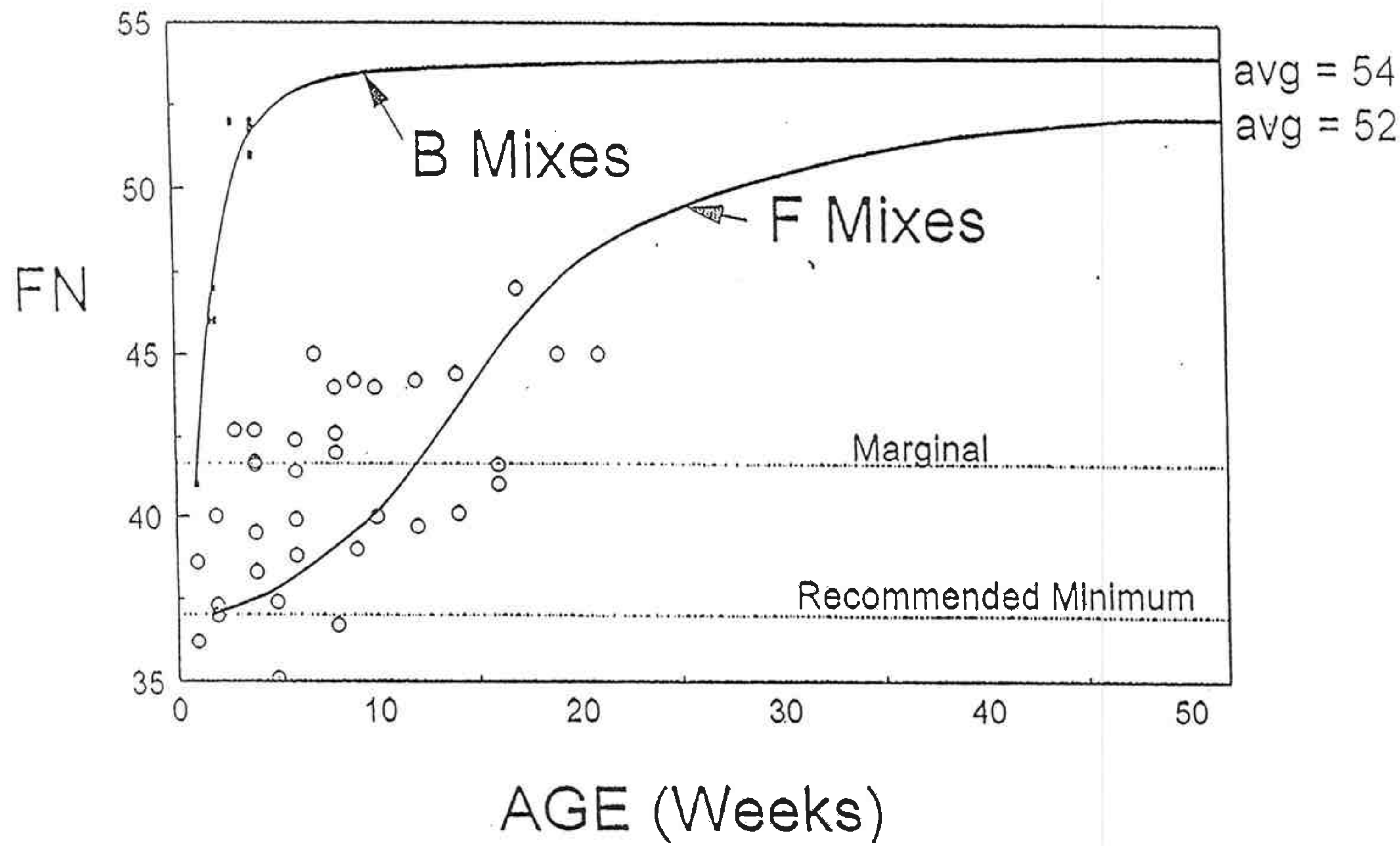

Figure 2.2 Frictional testing of newly constructed mixes (after ODOT, 1993)

source in which spray was measured as a voltage received from the detector. Open-graded friction course (OGFC) was shown to provide a significant reduction in water spray.

The other safety benefits, reduced hydroplaning and potential glare reduction, have also been factors in the increased usage of OGFC. A glare reduction example was illustrated in the NCHRP Synthesis 180 (Smith, 1992). A photo demonstrated how glare reduction changes at a juncture of porous to dense-graded mixtures. As for hydroplaning benefits, porous mixtures work better because the excess water is allowed to drain through the pores, and not run across the pavement surface (Copas et al., 1978).

Another important advantage of porous pavements is from an environmental view. Noise levels of porous pavements have been reported to be somewhat lower than that of dense-graded pavements or PCC. Rolling tire noise on pavements is generated when the individual elements of the tire tread come into contact with the road surface. When the tire leaves the pavement surface, a "pumping" effect causes the noise (Jorgan, 1994). The porous structure of open-graded mixtures allows some of this air to be pumped into the voids, instead of off of the pavement surface, and thus decreases noise. The actual A-weighted sound levels seem to be lowered by around 0 to 6 dB(A) (Smith, 1991). The public have voiced an opinion that porous pavements seem to provide a quieter ride both inside and outside the vehicle, but sound experts say that the human ear can only detect a 3 dB(A) change in sound levels (Huddleston et al., 1992). A study by the Maryland State Highway Administration also looked into this difference by checking the 1/3 octave levels of the sound spectrum (Polcak, 1990). Figure 2.3 displays how the upper range frequencies improve with the use of porous pavements. As the upper range of frequencies are considered "harsher" to the human ear, this graph shows a significant improvement.

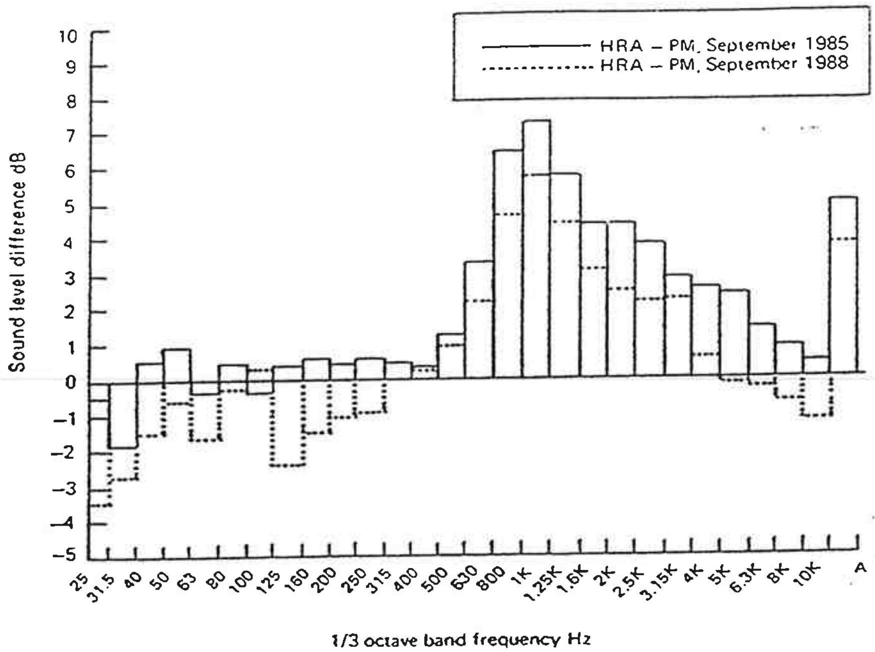

Figure 2.3. Comparison of the 1/3 octave band for porous asphalt and concrete (after Polcak, 1990)

Table 2.3. Limitations/disadvantages of porous pavements

Limitations	Sources of Problems	Sources of Information
Construction	Hand work, draindown, feathering	Younger et al., 1993
Winter Performance	Loss of de-icer in pores, snowplow damage, sand clogging pores	Camomilla et al., 1990; Smith, 1992; Huddleston et al., 1993
Oxidation	Oxidation of binder results in raveling	Smith, 1992
Patching	Small quantities of F-mix rarely made, so patches of dense mix restrict flow (unless an open-graded cold mix is used)	Smith, 1992

One other benefit of porous pavements are their rutting resistance. The high aggregate interlock of porous mixes allow this pavement type to resist deformation for longer periods of time (Huddleston et al., 1992).

2.3 Limitations

Based on the literature review and the questionnaire survey, several limitations (or disadvantages) of porous pavements were also identified. They are summarized in Table 2.3 and each are discussed in detail below.

Limitations for F-mixes were identified during the survey of contractors and ODOT project managers (Younger et al., 1994). Problems have been experienced with porous mix construction, particularly in the area of placement. The lack of fine particles in the mix make it difficult to feather a mix to meet the adjacent pavement grade. Any handwork provides challenges, as porous mixtures do not rake very well. Hauling of porous mixtures can sometimes be a problem, as the high asphalt contents will often drain down to the bottom of the mixture, causing fat spots in the pavement mat.

Another reported problem with porous mixtures is that of winter performance. The rough macrotexture of porous pavements can result in aggregate pickout by snowplows (Huddleston et al., 1993). The increased voids have also been shown to cause problems with deicing chemicals, as they flow through the pores of the pavement faster, and sometimes require as much as three times the amount of chemicals to be effective (Camomilla et al., 1990). This is not a general consensus, as the survey conducted by Smith (1992) found that 12 agencies indicated no difference, eight indicated that it was less effective, and two even said that de-icing chemicals on porous pavements were more effective. The use of winter sand for increasing friction on porous mixtures has been shown to clog the pavement and decrease the effectiveness of the void structure.

There are other reported problems with using porous mixtures. One is that porous mixtures have increased problems with oxidation (Smith, 1992). The open nature of this mix type causes the asphalt cement to oxidize more rapidly than normal, resulting in raveling. This problem is not as great today, due to the thicker asphalt film obtained in porous mixes with modified binders.

Another limitation is in patching of open-graded mixtures. Since porous mixes drain laterally, it is not viable to patch large portions of problem open-graded areas with a dense graded mix. Open-graded mixes are not normally mixed at the plant in small, patching sized batches, since the mixing and compacting temperatures are different from a dense mix. If batching plants would agree to make small batches of open-graded hot mix patching would be a lesser problem. An option would be to use an open-graded emulsion mixture for patching.

2.4 Summary

The questionnaire survey results suggest several problems that can exist during the construction of F-mix pavements (Younger et al., 1994). However, there are a number of techniques presently being used to work around these problems. Through the sharing of these techniques, the qualities of the F-mix pavements throughout Oregon are improving every year.

The significant advantages of porous pavements still seem to outweigh the disadvantages, and until it is proved otherwise, ODOT continues to place F-mix pavements throughout Oregon.

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3.0 FIELD EVALUATION

The field evaluation portion of this study was designed to provide insight into how porous pavements perform under road conditions. Tests were conducted to measure texture depth, permeability, noise levels, and the splash and spray characteristics of the pavement surface. Also, accident records were analyzed to determine whether or not porous pavements have any effect on increasing the safety at a new location.

3.1 Projects Evaluated

A selection of projects around Oregon were chosen as a part of our study of porous pavements. These are shown in Table 3.1 and in Figure 3.1.

These sites were chosen to provide a mixture of environmental regions with varying traffic and pavement age. Time effects could be quantified over a shorter period of time with a range of pavement ages. Of course, the pavement sites differ in weather/traffic/layout which affects the analysis. But with the number of sites evaluated, a general idea of how F-mixes perform under varying circumstances could be developed.

3.2 Evaluation Methods

Various test methods were employed in the field evaluation effort. Each is described below.

3.2.1 Texture Depth

Texture depth was measured using the sand patch method (Texas DOT, 1972). The sand patch method consists of spreading a known quantity of sand in a circle on the pavement surface. The texture depth is a function of the sand circle radius placed on the pavement (see Figure 3.2).

Table 3.1. **Porous pavement projects**

Map No.	Project Name (F-mix unless stated otherwise)	Mile Points	ADT (1992)	Construction Date
1	Marquam Bridge to N. Tigard Interchange (I-5)	291.8 - 300.4	90983	1991
2	Hayesville to BattleCreek (I-5)	249.5 - 259.1	53750	1990
3	Azalea to Jump Off Joe (I-5)	67.0 - 90.2	15506	1994
4	Jump Off Joe to N. Grants Pass (I-5)	67.1 - 58.2	22500	1992
5	Murphy Road to Lava Butte (U.S. 97)	141.5 - 150.8	21750	1989
6	E. Pendleton to Emigrant Hill (I-84)	213.0 - 217.7	8550	1993
7	Oregon 138 near Diamond Lake (OGEM)	83.0 - 86.3	970	1976

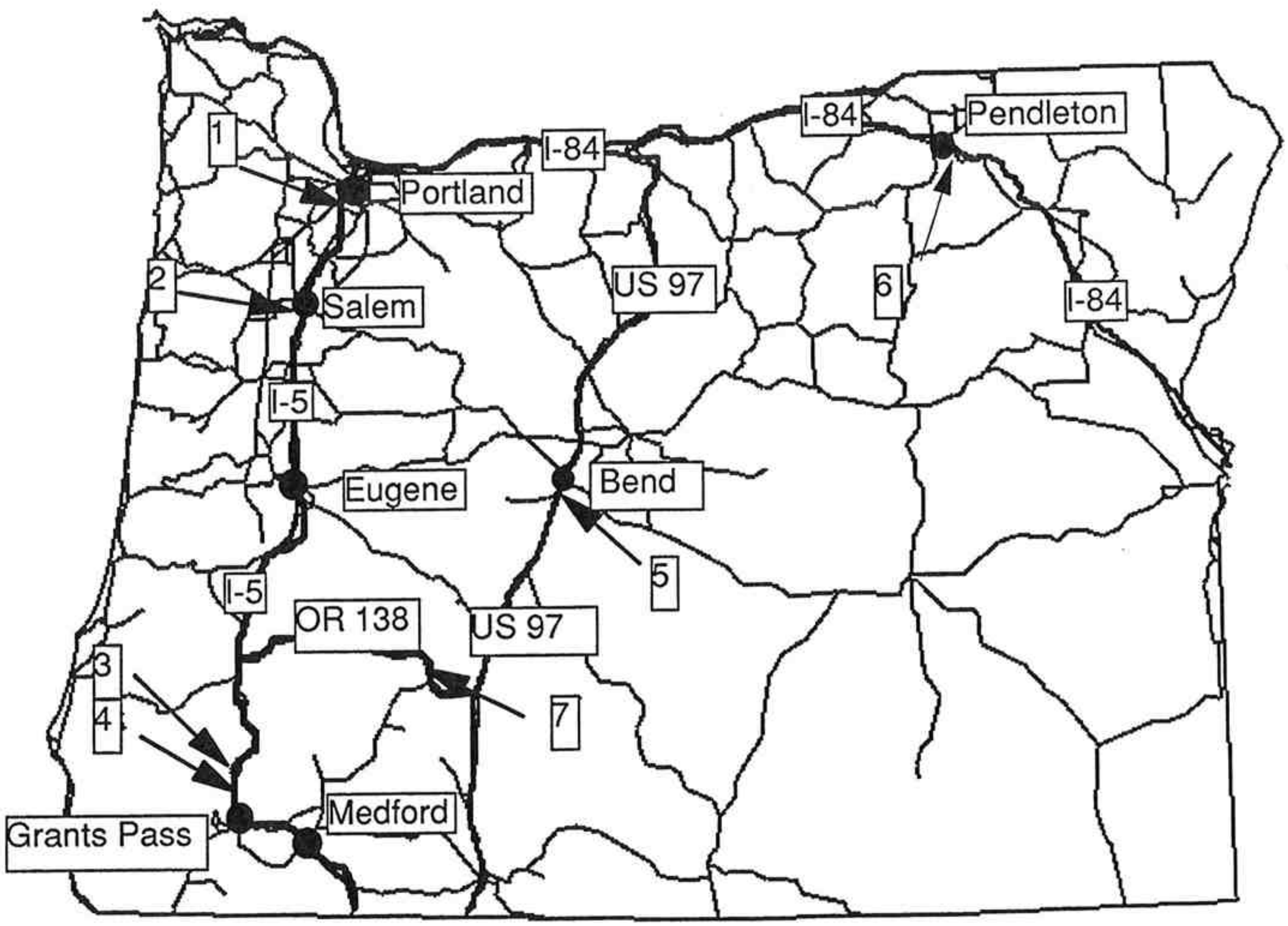


Figure 3.1. Oregon project site locations

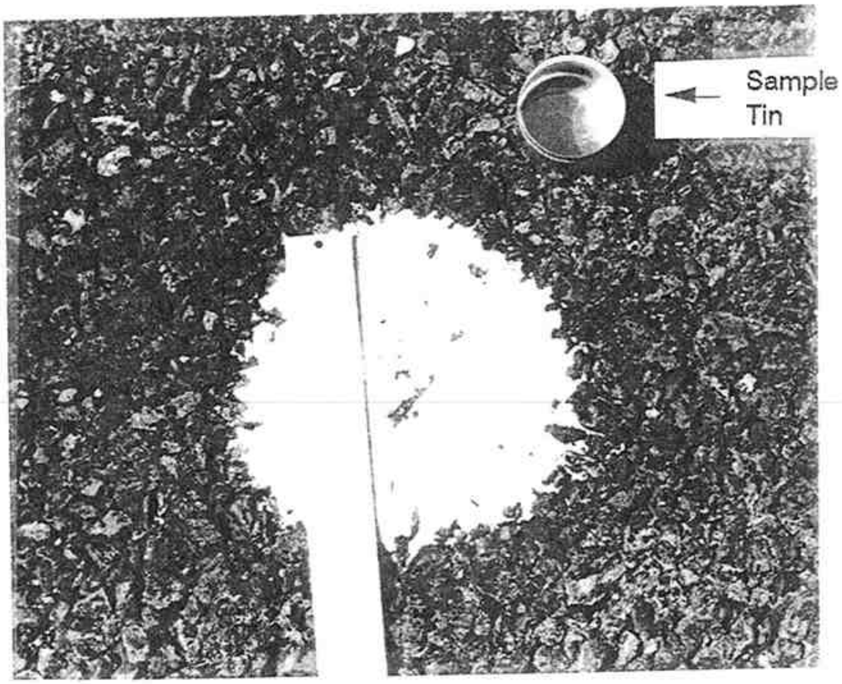


Figure 3.2. **Texture depth measurement device**

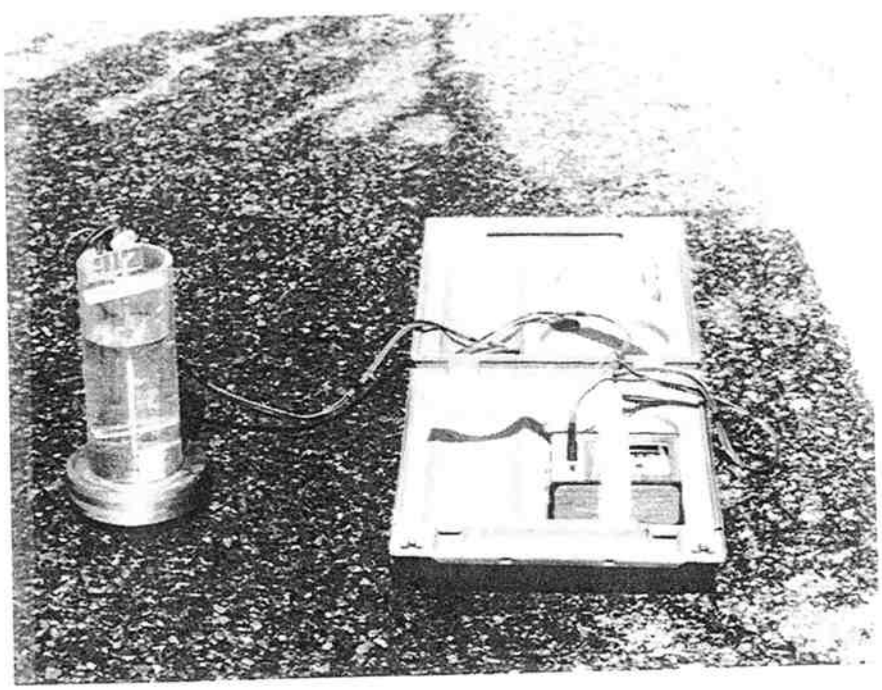


Figure 3.3. **Permeability measuring device**

Use of this method proved to be a problem at some of the newer sites because the sand dropped into the pores of the pavement. When this happens, the only statistically viable data from the measurement is that the pavement pores are highly open.

Another limitation of this test is that the pavement surface must be dry in order to perform the test. Because of this the I-84 (E. Pendleton - Emigrant Hill) site was not tested in 1993. This site could easily be tested at a future time, when the pavement surface is dry.

3.2.2 Water Permeability

All test sections were evaluated for water permeability using the device shown in Figures 3.3 and 3.4. The test uses a hard plastic standpipe 13 in (330 cm) long. Initially, the water is 11.3 in (287 cm) from the pavement surface. The time requirement for the water to drain out of the pipe from a height of 9.5 to 7.5 in (241 to 190 cm) is recorded. A rubber ring connects the permeameter to the pavement surface. The inside diameter of this ring is 2.3 in (58.2 cm) (where water drains into the pavement). Two 2.5 in (63.2 cm) metal rings are placed on the outside of the pipe to hold the rubber ring onto the pavement surface.

It was noted that moving the permeameter only short distances could change the readings from the test. This was due to variations in the pavement surface allowing more or less water to flow. This was very evident at the U.S. 97 project as parts of the mat had large stone particles picked out as a result of winter snowplow damage and/or raveling problems. Placement of the permeameter was very important, because if the device were placed near or on a pavement defect, the values would change drastically. In addition, if the permeameter was placed on a section that had bleeding problems, the permeability values would decrease rapidly because the surface was relatively impermeable. Care had to be taken to place the permeameter away from these areas. The permeameter was placed at three different places for each measurement location, but because of higher variability, even more sites would have been useful.

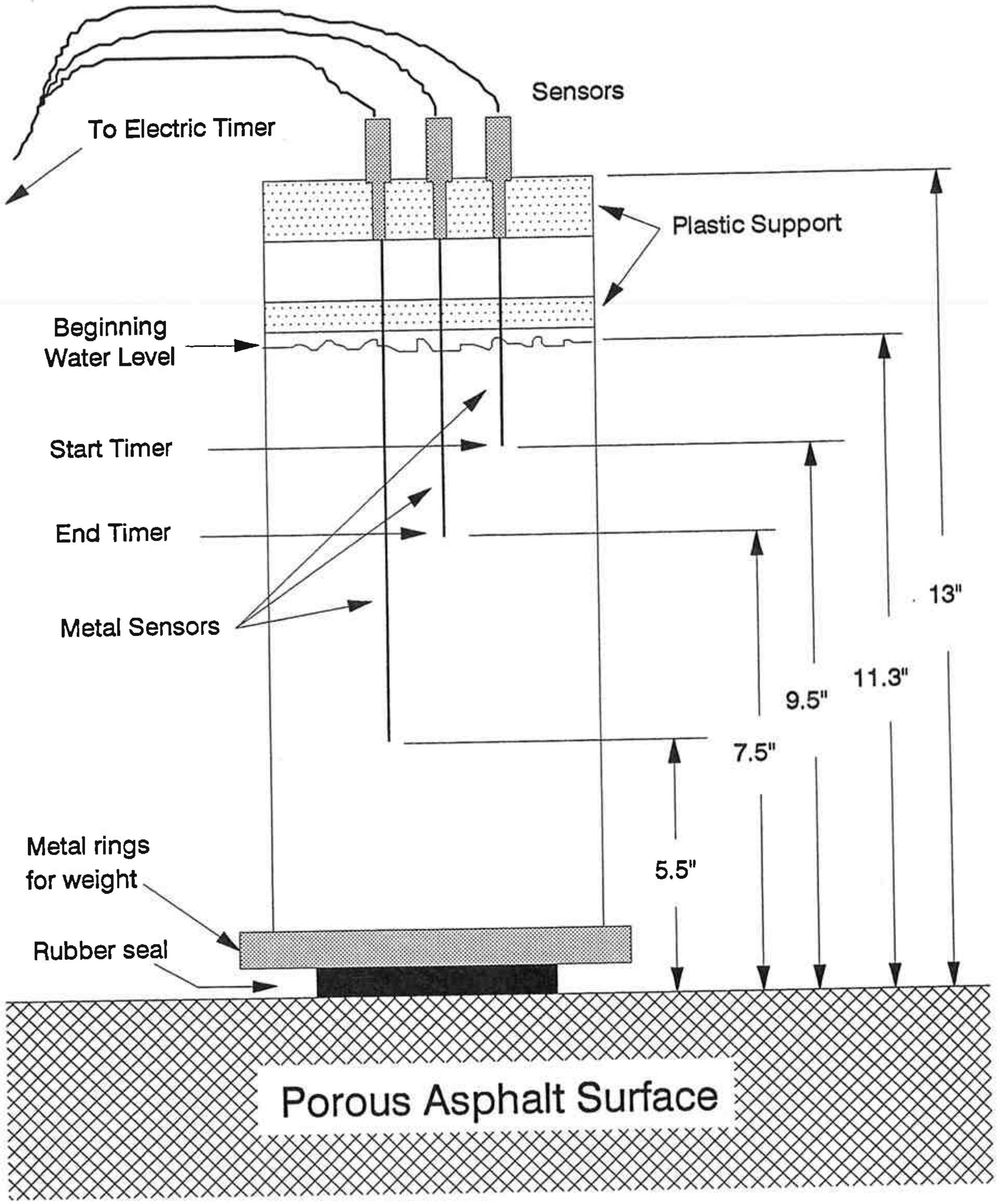


Figure 3.4. Drawing of permeability device (1 in = 25.4 mm)

3.2.3 Rutting

Rutting measurements were taken during the field surveys in August 1993 and September of 1994. Two measurements were taken and averaged at each site for both inside and outside wheel tracks. Rutting was measured by placing a straight 6 ft (1.8 m) long rut depth guage across the wheel tracks, and then measuring the deflection of the gauge. Figure 3.5 shows how the rut depth gauge is used to measure the rutting in the wheel path. The rut depth data were compared from 1993 and 1994 in an attempt to evaluate the change in road deformation for the test site.

3.2.4 Friction Testing

Friction testing was performed on all the sites, except I-5 (Marquam Bridge - North Tigard) site, to quantify how Oregon's pavements perform under wet and dry conditions. Testing was performed using a K.J. Law Model 1290 Computer Controlled Pavement Friction Tester. Figure 3.6 shows a picture of this device. Tests were performed at speeds from 30 to 60 mph (48 to 96 km/h) in both wet conditions and dry conditions.

Dry conditions for this testing were based on ASTM E 274 testing procedure. Water is applied at a calibrated rate at 40 mph (65 km/h) to deliver 4.0 gal $\pm$ 10 %/min in (600 ML/min mm) of wetted width. The water layer has to be at least 1 in (25 mm) wider than the test tire tread and applied centrally between the edges. Wet conditions were actually same as dry conditions, only that there was additional water from heavy rainfall on the roadway. Heavy rainfall was measured in a subjective manner by ODOT's pavement friction testing crew.

The speed gradient under wet conditions was the main focus of this testing plan. Reports such as Huddleston et al., 1990, have shown that porous pavements retain wet condition friction numbers better at higher speeds than other pavement types. The friction gradient is actually just the slope of the line for testing points acquired at various speeds. Speeds chosen for this test were 30,

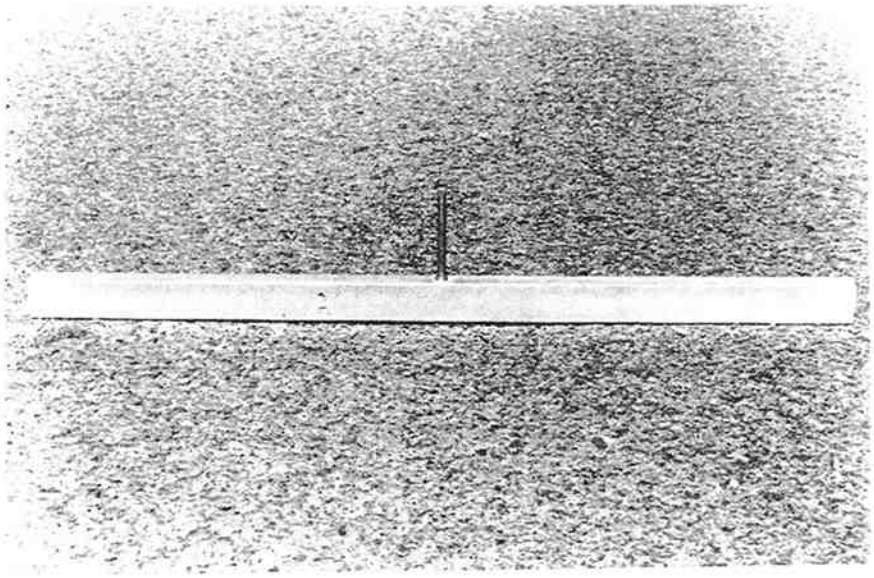


Figure 3.5. Rut depth measuring device

Figure 3.6. K.J. Law pavement friction tester

40, 50, and 60 mph (48, 64, 86, and 96 km/h). Some areas, however, were in a 55 mph (89 km/h) speed zone and were not tested at 60 mph (96 km/h).

3.2.5 Accident Data

ODOT compiles accident information gathered yearly (ODOT, 1994). These data were used in an attempt to quantify the safety benefits of F-mix pavements. Data were evaluated from 1986 to 1993. The data used for the analysis are for both intersectional and nonintersectional accidents. An intersectional accident on an Interstate are defined as accidents which occur as a result of a conflict at an on/off ramp. Tables C.11 through C.15 in Appendix C present the raw data used in this analysis. These tables show that there were relatively few accidents involving intersectional conflicts as compared to nonintersectional. Since there were not enough accidents in the intersectional category to analyze, it was decided to analyze only the nonintersectional data.

Additional data presented in Appendix C (Table C.10) display the traffic ADT volumes for the test sections from 1986 to 1992. At the time of this report (September 1994) ODOT did not have the 1993 traffic volume information available.

3.2.6 Splash and Spray

Splash and spray tests were performed on selected pavement surfaces throughout Oregon. The surfaces examined were PCC, B-mix, and F-mix pavements (Table 3.2). Tests were conducted so that measurements were taken once for every 10 ft (3 m) of pavement length. Since measurements were conducted behind a car traveling 55 mph (87 km/h), a measurement was recorded every 0.124 seconds. The spray device, designed and constructed at Oregon State University, was mounted behind the rear wheel of the test vehicle directly in the spray path.

Table 3.2. Location of projects evaluated

Highway	Date Evaluated	Speed (mph) (km/h)	Surface Type	Date of Construction
U.S. 34 – Tangent	November 1994	55 (88)	F-mix	1992
U.S. 34 – Tangent	November 1994	55 (88)	PCC	1992
U.S. 99W – S. Corvallis	November 1994	55 (88)	B-mix	1993
I-5 - S. M.P. 195-200	November 1994	55 (88)	PCC	1989
I-5 - S. M.P. 205-210	November 1994	55 (88)	B-mix	1976

The schematic of the device is shown in Figure 3.7. Water from the roadway is "kicked back" by the tire and goes through the 1 in × 6 in (25.4 mm × 150 mm) opening. The water that flows through the opening blocks the light from the LED array, and changes the amount of voltage registered from the circuit. This voltage is shown as the change in water intensity flowing from the tire. The lower the voltage readout, the more spray coming off the roadway. The data are recorded by the program written for this purpose. The data are recorded at the set in 0.124 seconds/measurement, although the measurement rate can easily be changed in the program. The program records measurements over a distance specified by the user. The voltage data are saved during the program run, and easily taken off the hard disk for analysis on a spreadsheet. Voltages for the splash and spray are taken between 2.3 V and 0 V. The voltage reads 0 volts if the LED light source is completely blocked, and 2.3 volts if there is no obstruction. Usually the spray intensity measurements will fall in the 2.0 to 1.0 voltage range, with a higher voltage defined as less spray.

Tests were conducted to try to discern a difference in spray qualities for the F-mix pavement, B-mix pavement, and PCC pavement. As tests were all conducted at a 55 mph (87 km/h) standard speed 3 ft (0.9 m) behind the wheel of a 1979 Mazda pickup and 1.2 ft (0.37 m) above the pavement surface, the spray intensity charge (measured as voltage) should be directly comparable across pavement types. The rainfall intensity for a 15-minute interval is taken at each site.

3.3 Test Results

3.3.1 Texture Depth/Water Permeability

Data for texture depth and water permeability were collected in August 1993 and repeated in September 1994. Table 3.3 summarizes the data.

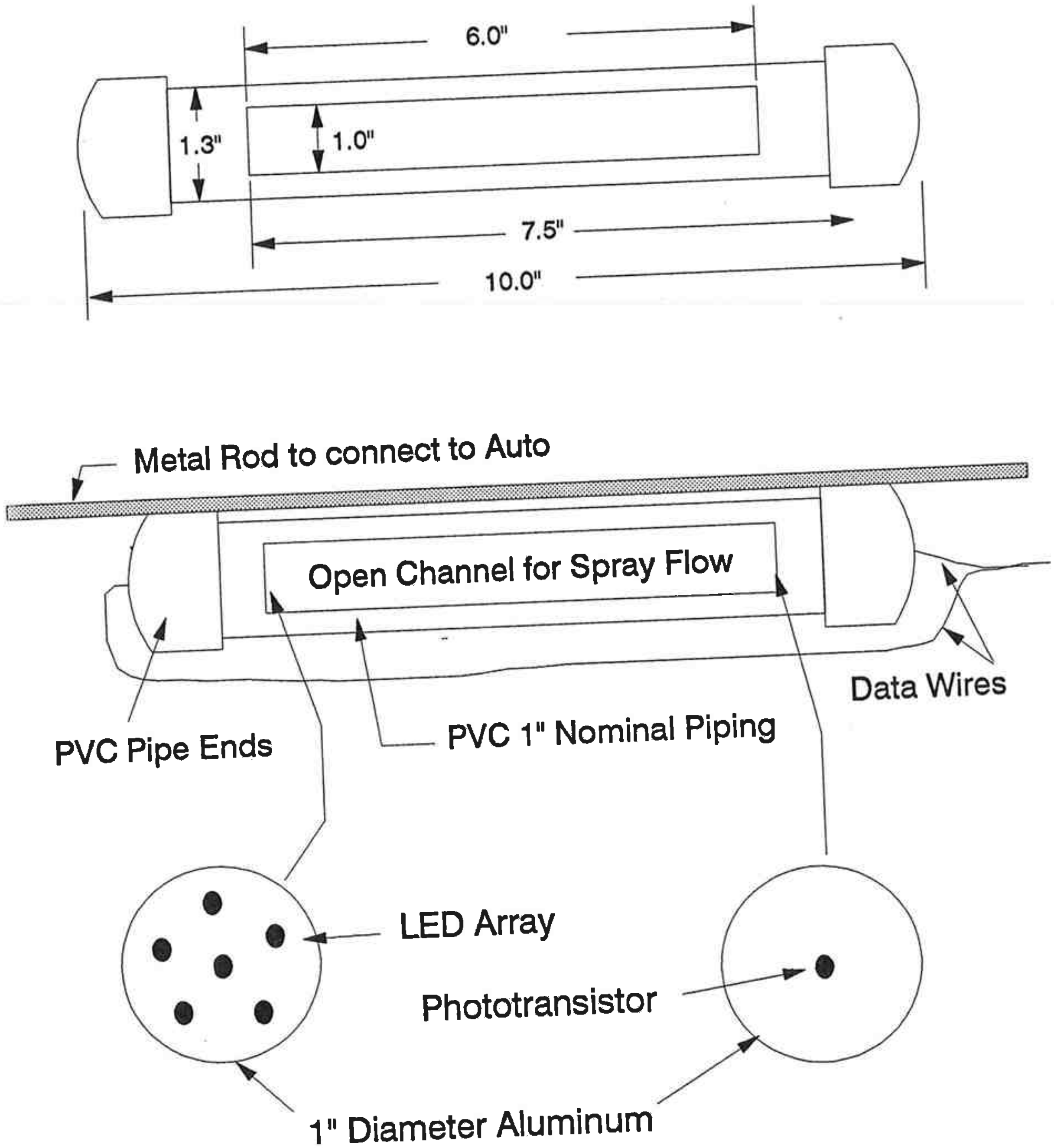


Figure 3.7. Diagram of splash and spray device (1 in = 25.4 mm)

Table 3.3. Permeability and texture depth test results

Site	Mix Type	Location	Texture Depth (in)[f] 8/93	Texture Depth (in)[f] 9/94	Permeability (s) 8/93	Permeability (s) 9/94
a) Open Mixes						
I-5 Terwilliger	F	Shoulder IWP BWP OWP	0.055 0.063 0.072 0.079	[b]	1.40 1.24 1.16 0.87	[b]
I-5 Salem	F	Shoulder IWP BWP OWP	0.068 0.088 0.085 0.081	[b]	1.52 1.00 0.99 0.76	[b]
I-5 Grants Pass	F	Shoulder IWP BWP OWP	0.121 0.106 0.085 0.073	[c]	0.83 0.66 0.98 1.26	0.91 0.93 0.93 1.20
I-84 Pendleton	F	Shoulder IWP BWP OWP	[a]	0.092 0.100 0.099 0.098	1.18 0.84 1.32 0.92	1.43 0.84 0.97 0.89
U.S. 97 Bend	F	Shoulder IWP BWP OWP	0.055 0.062 0.054 0.068	0.082 0.90 0.076 0.088	2.09 1.01 1.44 1.41	1.65 1.35 1.74 1.53
Oregon 138 Diamond Lake	OGEM	Shoulder IWP BWP OWP	0.058 0.062 0.051 0.054	[d]	2.66 2.90 2.01 1.48	[d]
b) Other Mixes[e]						
I-5 Salem	B	Shoulder	0.025		6.67	
Tyler St.-Corvallis	New PCC	Shoulder		N/A		∞
Harrison St.-Corvallis	Slurry Seal	Shoulder		0.033		5.43
Circle Blvd.-Corvallis	New C-mix	Shoulder		0.010		18.32
14th St.-Corvallis	Old C-mix	Shoulder		0.018		14.21
15th St.-Corvallis	Old PCC	Shoulder		0.029		6.87

Notes:
[a]Pavement wet during Pendleton survey, so no texture depth readings were taken.
[b]Measurements not taken due to traffic control restrictions.
[c]Grants Pass site not measured as sand ran into pores too fast.
[d]Site not reachable due to construction.
[e]Other mix types only taken one time for comparison purposes.
[f]1 in = 25.4 mm

In an attempt to better use the texture depth data, a correlation between permeability and texture depth was hypothesized. Figure 3.8 shows that there is a fair correlation between the two data sets. Field experience might show that readings for a certain texture depth may vary a small amount with pavement changes, but this data shows a reasonable correlation.

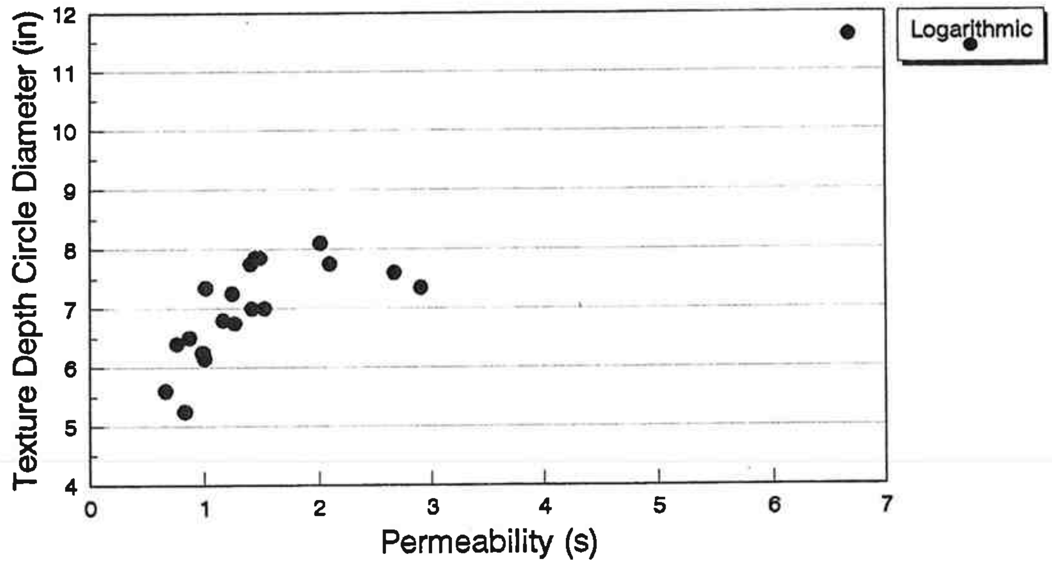
An area of interest was to see whether or not the permeability and/or texture depth changed over the period of the study, but the one year's change in permeability that could be recorded during the study time does not provide any definite conclusions. This is especially so because a limited number of sites were retested the following year due to construction at one site and problems with traffic control safety concerns on the busy Interstate 5 thoroughfare. From the data shown in Table 3.3, it would appear that the permeability values for Grants Pass-Jumpoff Joe increased during the year's time. Sand patch measurements were not recorded at this site, as the sand drained into the pores of the pavement during these tests and caused the texture depth measurements to be flawed.

3.3.2 Rutting

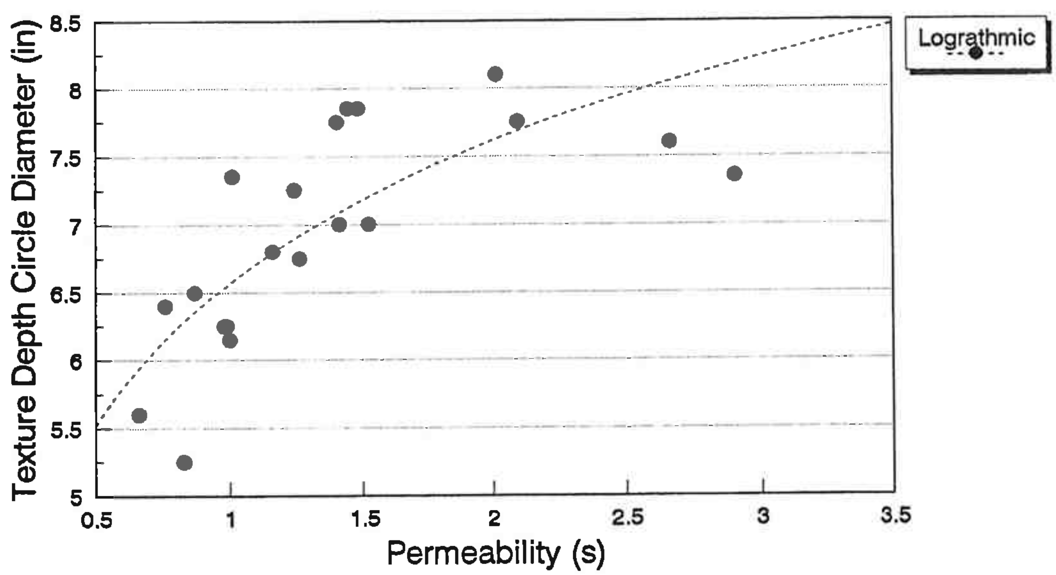
Rut depths were recorded from the sites in August 1993 and September 1994 at the same time as the permeability and sand patch tests. Table 3.4 displays the data for each test section over the two year period, and Figure 3.9 provides a graphical view of the rut changes. It is easily noticed that the depth of the ruts change little over the one-year period. This is a trademark of F-mix pavements, as rutting is not normally a problem.

3.3.3 Friction Data

Friction data were also collected twice during this study. Collection of the first set of data for this portion of the study was completed by February 1994 (Table 3.5). Figure 3.10 shows the first data collected during this experiment. The wet and dry data were collected under the same



a) All projects



b) Only F-mixes

Figure 3.8. Correlation between permeability and texture depth (1 in = 25.4 mm)

Table 3.4. Rut depth levels by year

Project	Date Constructed	Location	8/93 (in) (cm)	9/94 (in) (cm)
Marquam to N. Tigard (I-5)	1991	OWT IWT	none 1/4-1/8 (0.6-0.3)	*
Hayesville - BattleCreek (I-5)	1990	OWT IWT	1/8 (0.3) 1/4 (0.6)	*
Jumpoff Joe - N. Grants Pass (I-5)	1993	OWT IWT	none 0-1/8 (0.0-0.3)	1/8-1/4 (0.3-0.6) 1/8-1/4 (0.3-0.6)
Murphy Road - Lava Butte (U.S. 97)	1989	OWT IWT	1/8-1/4 (0.3-0.6) 1/4 (0.6)	1/4 (0.6) 1/4 (0.6)
E. Pendleton - Emigrant Hill (I-84)	1993	OWT IWT	none none	1/8-1/4 (0.3-0.6) 0-1/8 (0.0-0.3)
Diamond Lake (OR 138)	1976	OWT IWT	1/4-1/2 (0.6-1.3) 1/8-3/8 (0.3-0.4)	**

NOTES:
*Measurements not taken due to traffic control restrictions.
**Site not reachable due to construction.

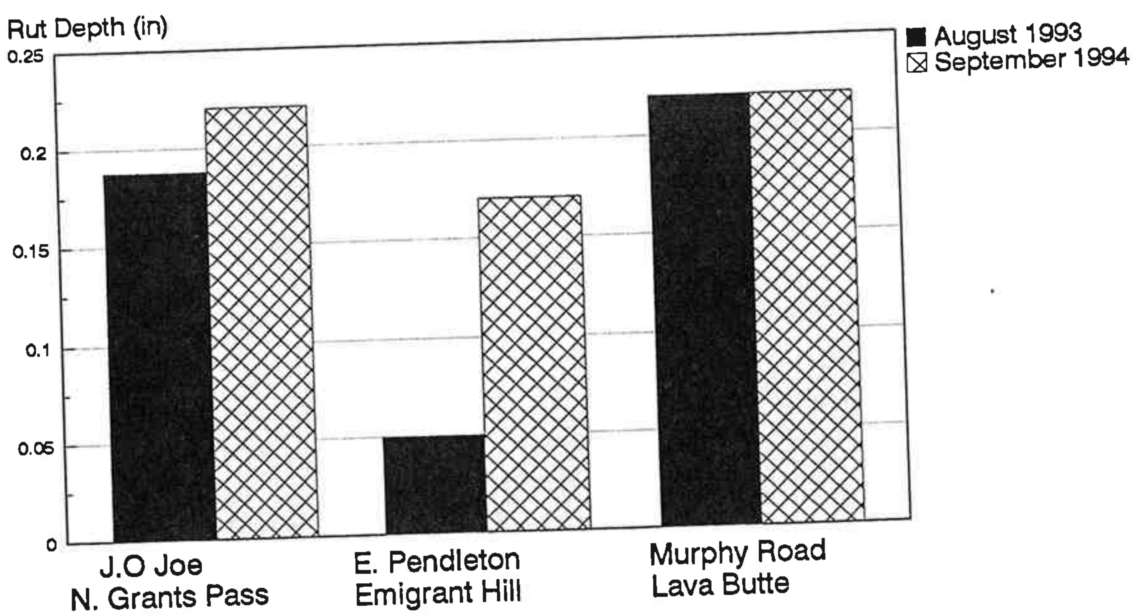


Figure 3.9. Rut depth changes by year (1 in = 25.4 mm)

Table 3.5. Frictional test results

Project	Condition	Date	Nominal Speed (mph) (km/h)	FN	Actual Speed (mph) (km/h)
Murphy Road-Lava Butte	Wet		30 (48)		
			40 (64)	47.2	40 (64)
			50 (80)	44.7	50 (80)
			55 (88)	43.6	55 (88)
Hayesville-BattleCreek	Wet	11/17/93	30 (48)	45.8	32 (50)
			40 (64)	43.1	41 (66)
			50 (80)	40.6	50 (80)
			55 (88)	39.3	55 (88)
		11/18/93	30 (48)	45.8	30 (48)
			40 (64)	43.1	40 (64)
			50 (80)	40.9	50 (80)
			55 (88)	39.8	55 (88)
		12/8/93	30 (48)	59.7	30 (48)
			40 (64)	54.5	40 (64)
			50 (80)	52.3	49 (79)
			55 (88)	50.1	55 (88)
		2/23/94	30 (48)	60.1	31 (50)
			40 (64)	55.8	41 (66)
			50 (80)	50.4	50 (80)
			55 (88)	51.1	55 (88)
	Dry	1/18/94	30 (48)	53.3	31 (50)
			40 (64)	49.9	41 (66)
			50 (80)	47.8	50 (80)
			55 (88)	46.5	55 (88)
		6/14/94	30 (48)	49.5	32 (52)
			40 (64)	46.6	40 (64)
			50 (80)	44.2	50 (80)
			55 (88)	43.9	55 (88)
N. Grants Pass	Wet	12/7/93	30 (48)	52.6	31 (50)
			40 (64)	48.9	41 (66)
			50 (80)	45.6	50 (80)
			55 (88)	43.6	56 (88)
			60 (97)	41.9	59 (100)

Table 3.5. **Frictional test results (Continued)**

Project	Condition	Date	Nominal Speed (mph) (km/h)	FN	Actual Speed (mph) (km/h)
Jumpoff Joe	Wet	11/15/93	30 (48) 40 (64) 50 (80) 60 (97)	45.1 44.0 42.5 41.2	31 (50) 40 (64) 50 (80) 60 (97)
I-5 Marquam Interchange	Wet	11/8/93	30 (48) 40 (64) 50 (80) 60 (97)	38.2 36.4 34.5 34.1	31 (50) 40 (64) 50 (80) 59 (95)
		12/1/93	30 (48) 40 (64) 50 (80) 60 (97)	56.8 52.4 48.6 46.4	30 (48) 40 (64) 50 (80) 58 (88)
		2/24/94	30 (48) 40 (64) 50 (80) 55 (88)	57.4 43.8 49.2 48.4	31 (50) 40 (64) 50 (80) 55 (88)
	Dry	1/18/94	30 (48) 40 (64) 50 (80) 55 (88)	48.3 45.3 43.5 43.2	31 (50) 40 (64) 50 (80) 52 (84)
		7/19/94	30 (48) 40 (64) 50 (80) 55 (88)	41.0 38.0 37.1 35.5	32 (52) 40 (64) 49 (79) 54 (87)
I-5 PCC	Wet	2/23/94	30 (48) 40 (64) 50 (80) 55 (88)	55.7 47.8 42.5 39.8	30 (48) 40 (64) 50 (80) 55 (88)
	Dry	1/18/94	30 (48) 40 (64) 50 (80) 55 (88)	49.6 43.8 39.3 37.5	30 (48) 41 (66) 51 (82) 56 (90)
		6/14/94	30 (48) 40 (64) 50 (80) 55 (88)	45.5 40.1 35.3 34.1	31 (50) 41 (66) 51 (82) 56 (90)

Table 3.5. Frictional test results (continued)

Project	Condition	Date	Nominal Speed (mph) (km/h)	FN	Actual Speed (mph) (km/h)
I-5 B-Mix	Wet	2/23/94	30 (48) 40 (64) 50 (80) 55 (88) 60 (97)	61.7 55.1 48.6 45.8	31 (50) 40 (64) 50 (80) 55 (88)
	Dry	1/18/94	30 (48) 40 (64) 50 (80) 55 (88)	56.5 51.2 45.7 44.9	31 (50) 40 (64) 50 (80) 55 (88)
		6/14/94	30 (48) 40 (64) 50 (80) 55 (88)	54.4 49.8 45.5 44.8	31 (50) 41 (66) 50 (80) 55 (88)

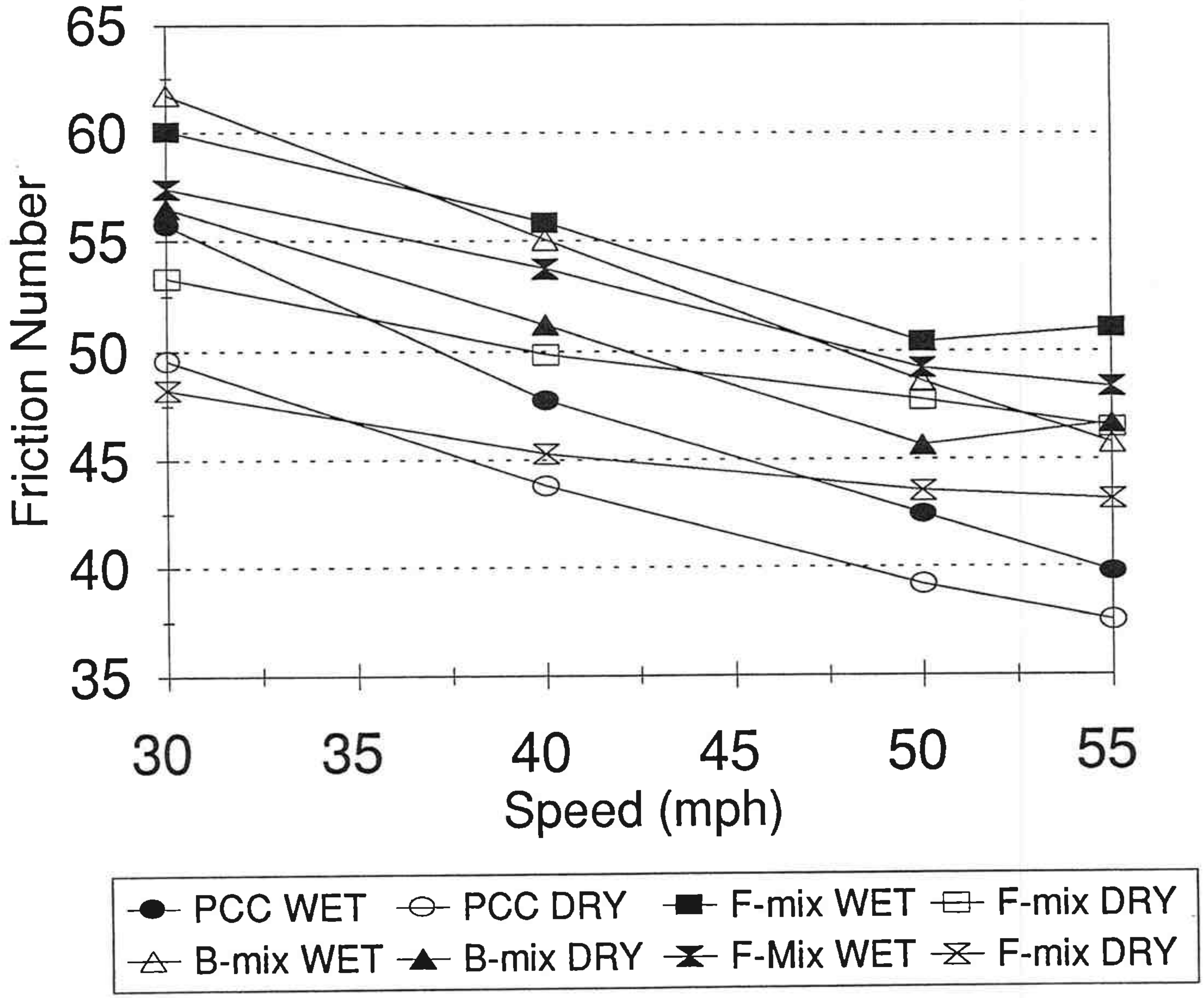


Figure 3.10. Frictional testing results (1994)

procedures as discussed earlier, where a "dry" test is actually a test performed using only the ASTM standard amount of water from the skid tester, and the "wet" test with both tester water and high intensity rainfall. As this data shows, for all data points the skid numbers for the "wet" condition were found to be higher than those recorded for the "dry" condition.

Additional tests were taken during the summer of 1994 to try to discover what anomalies in the data could be weeded out. Table 3.5 displays this data as well. From the data collected by the June and July testing for the I-5 test sites for B-mix, PCC, F-mix at BattleCreek to N. Jefferson, and F-mix at Marquam Interchange, yield "dry" pavement friction numbers that are even lower than those from earlier tests. No "wet" friction tests were possible during the summer months due to lack of rain. Even without the "wet" measurements, the low skid numbers collected in June and July show that these data would not help explain why friction numbers were lower for "dry" than those for "wet" conditions.

3.3.4 Accident Data

Nonintersectional accident data are shown in tabular form for each project (see Tables 3.6 through 3.10 - dotted line signifies change from B to F-mix). The total accident numbers for the year were divided by the average daily traffic (ADT) for each road section and year. The yearly accidents/ADT values were then plotted for each year (see Figures 3.11 through 3.15).

The two sites that provide the most information are the I-5 (Marquam to N. Tigard) and I- (Hayesville to BattleCreek) projects. Porous pavements on those projects were placed in 1990 for the Marquam to N. Tigard project, and 1989 for the Hayesville to BattleCreek project. A change in the accidents/ADT would indicate whether or not the placement of the pavement affected the safety of the roadway. Figure 3.11 seems to show a significant reduction in the total number of

Table 3.6. Marquam Bridge to N. Tigard Interchange accident data

Year	Avg. ADT	Fatal	Fatal/ ADT	Nonfatal	Nonfatal/ ADT	Property	Property/ ADT	Total	Total/ ADT
1986	79708	1	0.125	14	1.756	14	1.756	29	3.638
1987	85175	1	0.117	11	1.291	14	1.644	26	3.053
1988	91742	0	0.000	9	0.981	15	1.635	24	2.616
1989	91575	0	0.000	18	1.966	19	2.075	37	4.040
1990	85671	0	0.000	15	1.751	20	2.335	35	4.085
1991	93975	0	0.000	18	1.915	17	1.809	35	3.724
1992	90983	0	0.000	12	1.319	11	1.209	23	2.528
1993	**	0		4		9		13	

Table 3.7. Hayesville to BattleCreek accident data

Year	Avg. ADT	Fatal	Fatal/ ADT	Nonfatal	Nonfatal/ ADT	Property	Property/ ADT	Total	Total/ ADT
1986	39233	2	0.510	23	5.862	19	4.843	15	3.823
1987	41700	0	0.000	27	6.235	27	6.475	14	3.357
1988	50133	1	0.199	17	3.391	30	3.989	17	3.391
1989	50188	0	0.000	32	6.376	24	4.782	29	5.778
1990	50888	1	0.197	30	5.895	38	7.467	23	4.520
1991	51950	3	0.577	23	4.427	22	4.235	19	3.657
1992	53750	1	0.186	29	5.395	32	5.953	16	2.977
1993	**	2		25		30		9	

Table 3.8. Jumpoff Joe to N. Grants Pass accident data

Year	Avg. ADT	Fatal	Fatal/ ADT	Nonfatal	Nonfatal/ ADT	Property	Property/ ADT	Total	Total/ ADT
1986	13350	0	0.000	9	6.742	6	4.494	15	11.236
1987	13750	1	0.727	6	4.364	7	5.091	14	10.182
1988	14494	1	0.690	7	4.830	9	6.210	17	11.729
1989	15194	1	0.658	10	6.582	18	11.847	29	19.087
1990	15550	1	0.643	7	4.502	15	9.646	23	14.791
1991	15506	0	0.000	7	4.514	12	7.739	19	12.253
1992	16000	0	0.000	9	5.625	7	4.375	16	10.000
1993	**	0		5		4		9	

Table 3.9. E. Pendleton to Emigrant Hill accident data

Year	Avg. ADT	Fatal	Fatal/ ADT	Nonfatal	Nonfatal/ ADT	Property	Property/ ADT	Total	Total/ ADT
1986	5675	0	0.000	1	1.762	5	8.811	6	10.573
1987	6060	1	1.653	1	1.653	3	4.959	5	8.264
1988	25576	0	0.000	1	0.391	0	0.000	1	0.391
1989	6400	0	0.000	0	0.000	1	1.563	1	1.563
1990	6975	0	0.000	2	2.867	1	1.434	3	4.301
1991	7025	2	2.847	1	1.423	2	2.847	5	7.117
1992	8550	0	0.000	2	2.339	3	3.509	5	5.848
1993	**	0		0		1		1	

Table 3.10. Murphy Road to Lava Butte accident data

Year	Avg. ADT	Fatal	Fatal/ ADT	Nonfatal	Nonfatal/ ADT	Property	Property/ ADT	Total	Total/ ADT
1986	12825	**	0.000	**	0.000	**	0.000	**	0.000
1987	13325	**	0.000	**	0.000	**	0.000	**	0.000
1988	15275	0	0.000	11	7.201	11	7.201	22	14.403
1989	15900	3	1.887	10	6.289	14	8.805	27	16.981
1990	20375	2	0.982	8	3.926	9	4.417	19	9.325
1991	21850	0	0.000	5	2.288	14	6.407	19	8.696
1992	21750	1	0.460	8	3.678	13	5.977	22	10.115
1993	**	1		7		11		19	

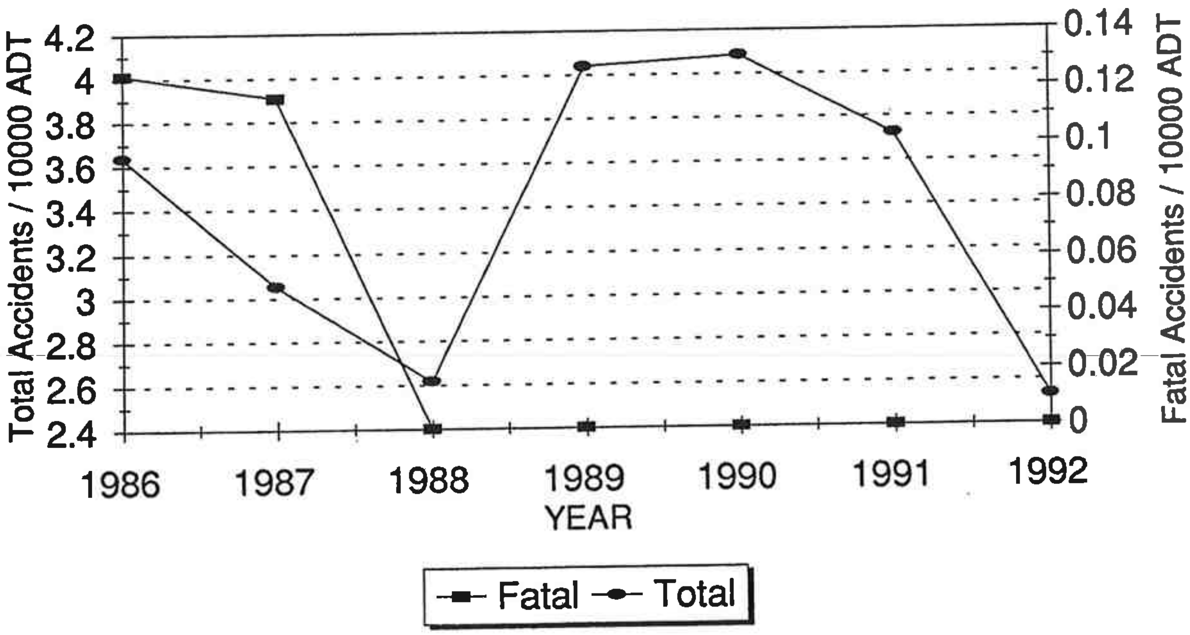


Figure 3.11. Marquam to N. Tigard accident information

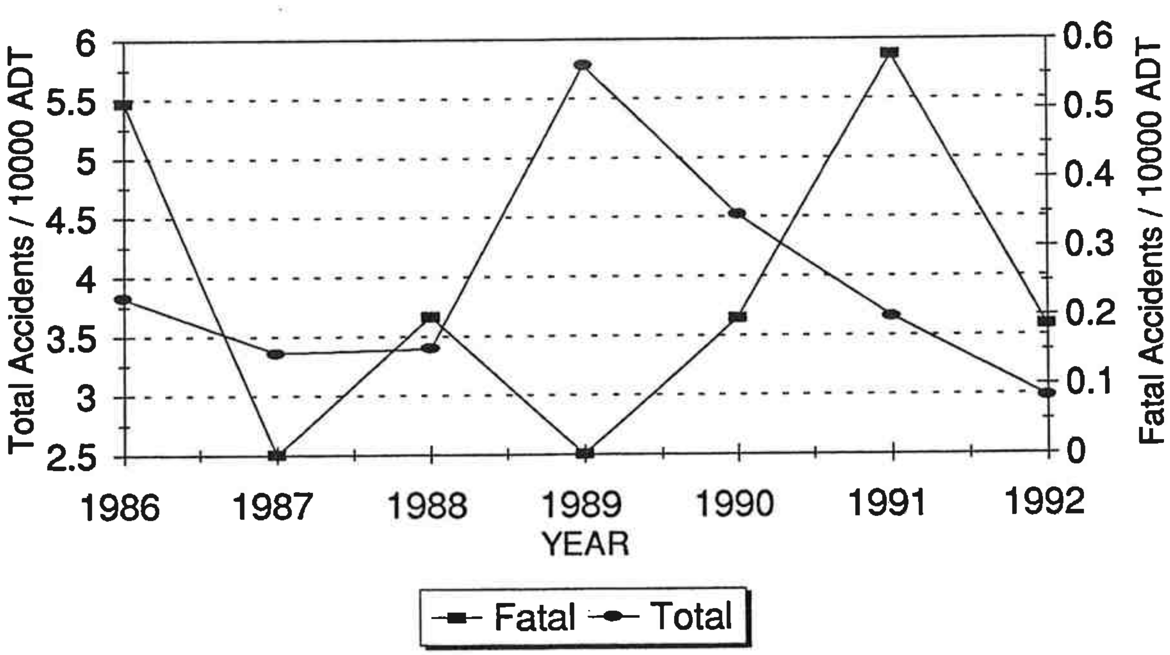


Figure 3.12. Hayesville to BattleCreek accident information

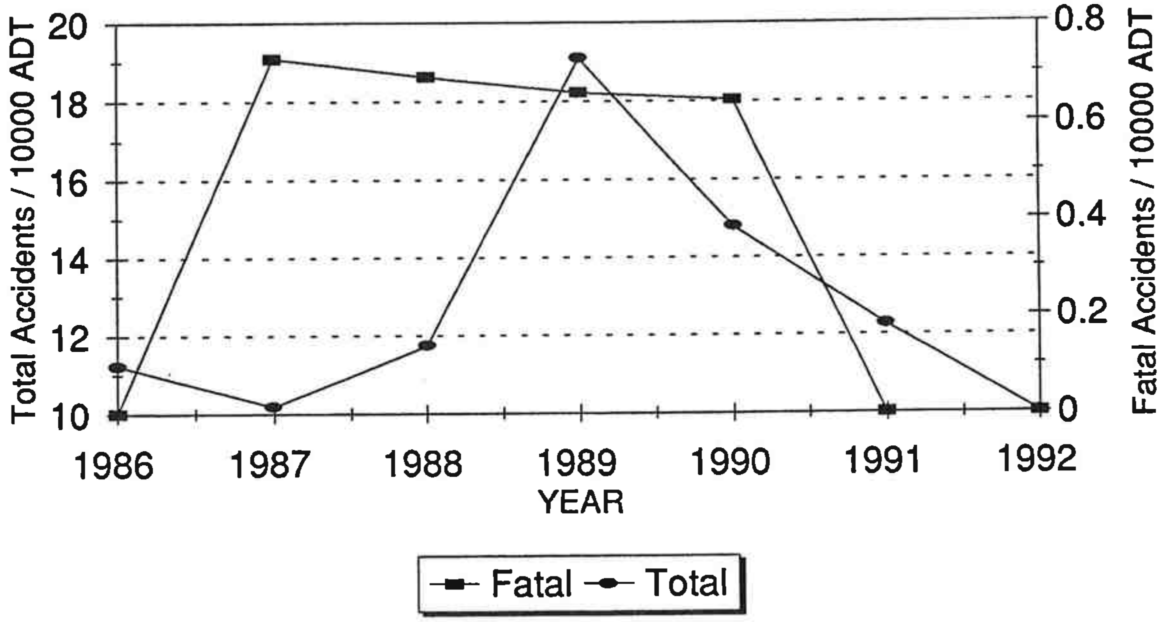


Figure 3.13. Jumpoff Joe to N. Grants Pass accident information

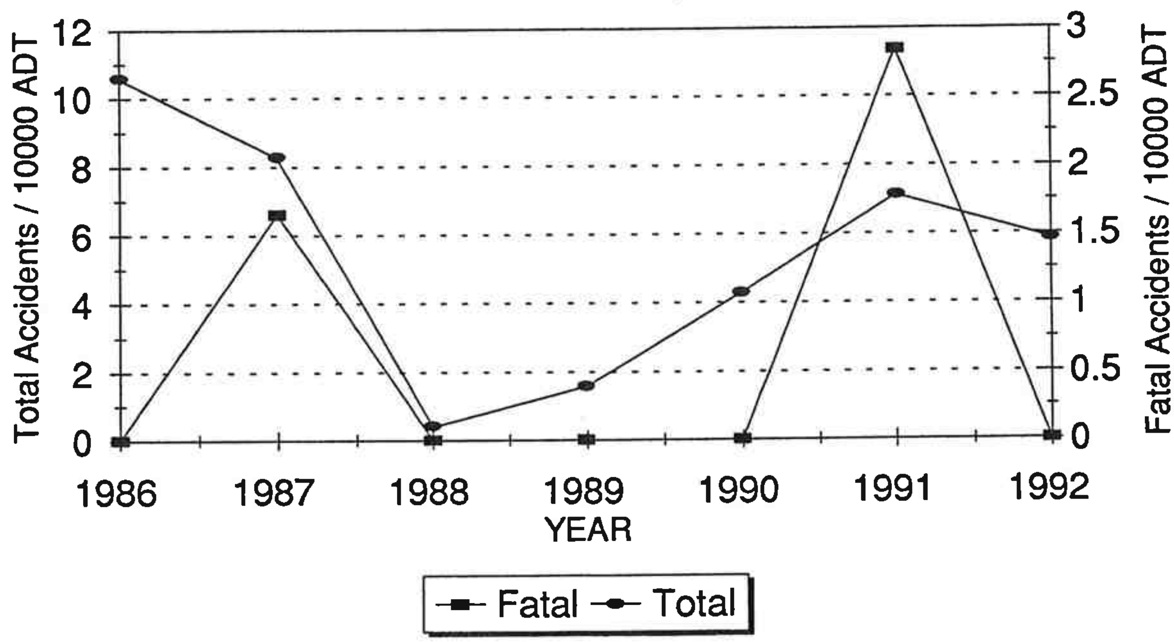


Figure 3.14. E. Pendleton to Emigrant Hill accident information

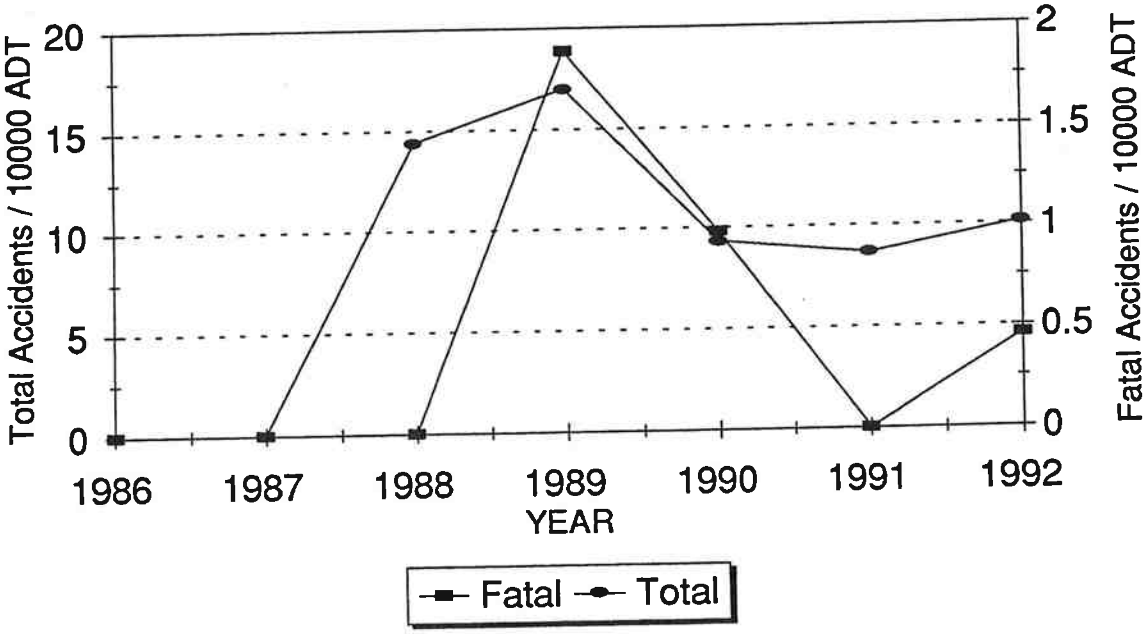


Figure 3.15. Murphy Road to Lava Butte accident information

accidents as a result of the change in pavement type in 1990. Figure 3.12, however, would seem to suggest that there was a significant decrease in fatal accidents, yet a rise in total accidents after the 1989 porous pavement. The rather low number of fatal accidents shown in Tables 3.6 through 3.10 would seem to suggest that there is not enough data in this area to make a significant conclusion. Further study ideas should include a comparison of F-mixes to both B-mixes and PCC pavements to see if accident rates are changing on all pavements.

3.3.5 Splash and Spray Results

Splash and spray data are shown in Figure 3.16. This figure displays a comparison between an F-mix pavement, a B-mix pavement, and a PCC pavement. These pavements were chosen for the analysis because each was determined from visual inspection to be in good condition, and were fairly new. The F-mix pavement and PCC pavement were taken on Interstate 5 between mileposts 195 and 210. The rainfall intensity for these two sites for a 15 minute peak was 0.06 inches. The B-mix site intensity was 0.04 inches. The average spray voltage for each pavement type was 1.55 V for F-mix, 1.47 V for B-mix, and 1.47 V for PCC. As the spray meter only measures spray between the 2.0 to 1.0 V range, this 0.1 V decrease suggests a 10% improvement in spray. The exact effect that rainfall intensity has is not known, yet a higher intensity produces more water on the road and more spray. The actual spray intensity for the B-mix pavement would be less than 1.47 if this data were taken at the high rainfall intensity recorded for F-mix and PCC.

An attempt was made to try to measure rainfall intensity directly across pavement types along U.S. 34 near Tangent. Figure 3.17 displays these results. The rainfall intensity for this measurement was 0.025 inches for the 15 minute intensity, yet the rainfall was continuous throughout the day. The dotted line in Figure 3.17 marks the juncture of F-mix to PCC pavement, with F-mix in the 0-910 ft range. The figure shows the pavement spray for the PCC was less than the F-mix

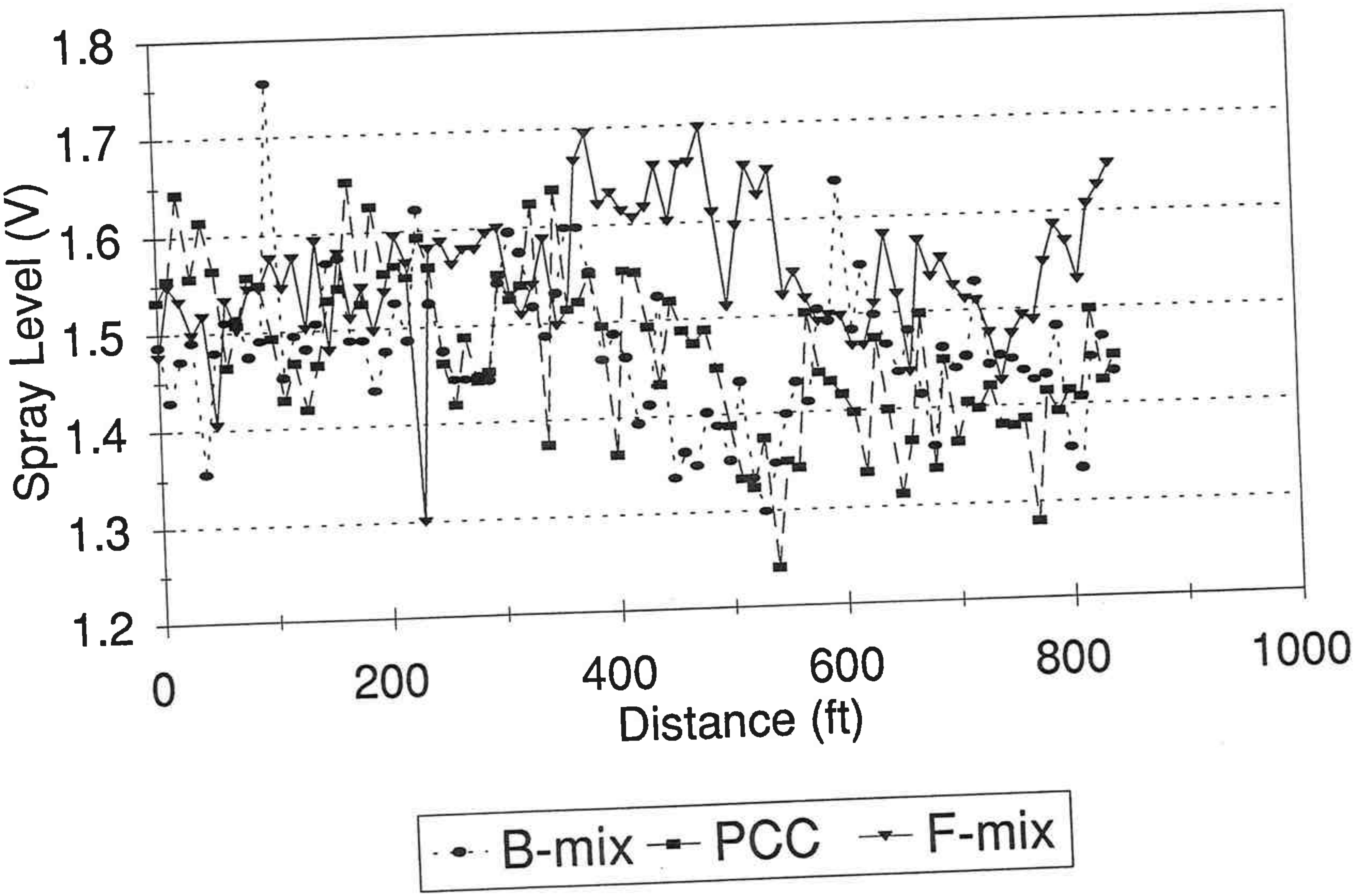


Figure 3.16. Spray levels for F-mix, B-mix, and PCC pavements

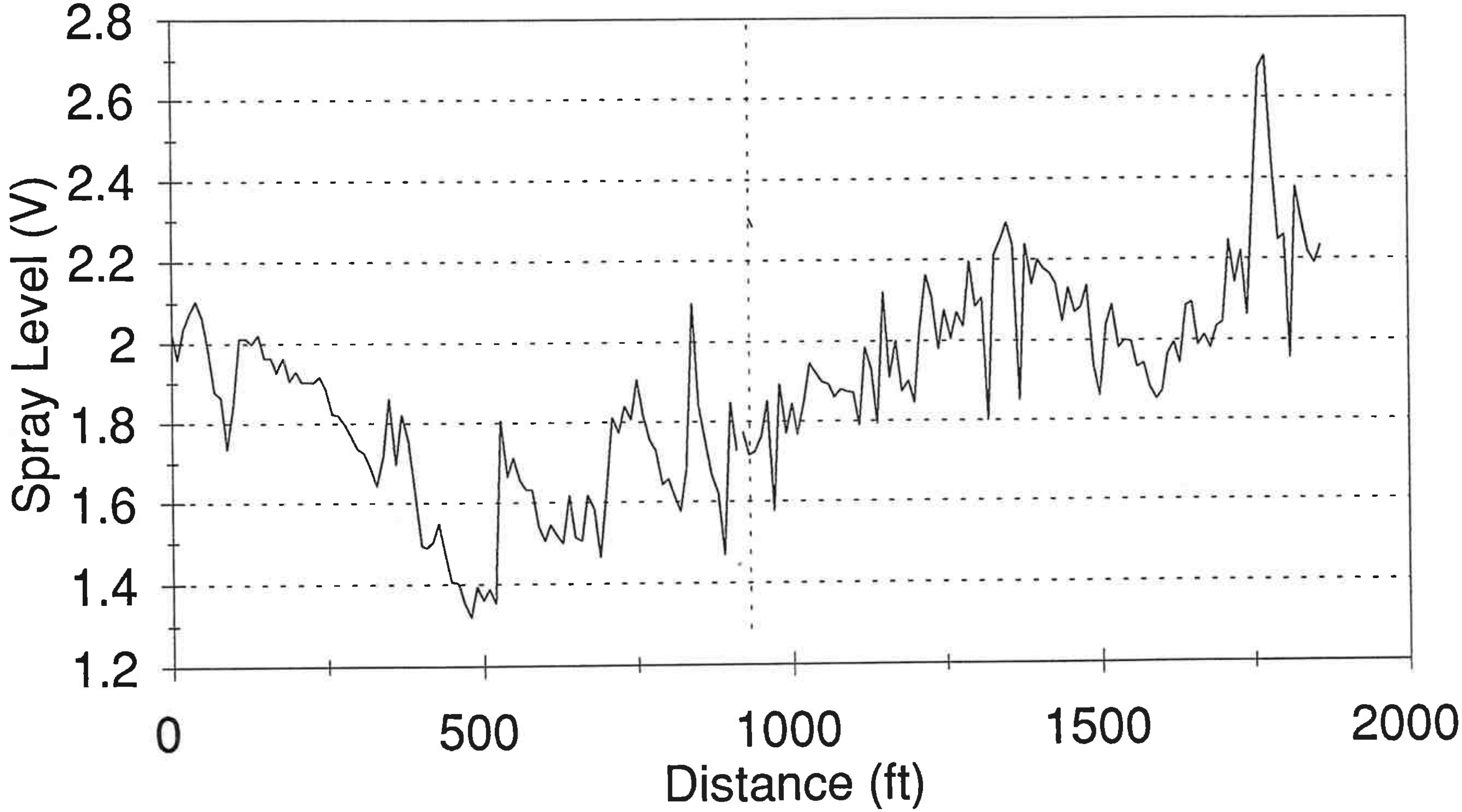


Figure 3.17. F-mix to PCC spray comparison across joint

(higher voltage). The reason is the F-mix pavement near the juncture was flooded and not draining. Pools of water in the lane caused the high amount of splash and spray. The F-mix pavement is an overlay completed in 1992, yet the cross slope was relatively flat. This apparently hindered the lateral flow of water and lessened the F-mix's spray reducing properties.

3.4 Summary of Results

The data collected during the field study portion of this project were very useful in determining the field performance of porous pavements. The rutting and permeability measurements provide information about the change in properties of porous pavement for a one-year time period. The skid and accident data provide mixed insight into the safety properties of porous pavements. However, to have these data make more sense, it is necessary to record the pavement properties over a longer period of time.

Good data could be collected over an extended period of time, but first some of the test deficiencies would have to be addressed. One deficiency occurs when measuring the pavement permeability. For example, the measurements of "permeability" on some dense asphalt and PCC pavements is suspect. Currently, the permeameter is connected to the pavement surface by a hard rubber disk. This allows the water to not only flow through the voids of the pavement, but through the uneven texture of the surface, thus making the permeameter more of a texture meter than an actual permeability measurement device. The connecting mechanism to the pavement should, in actuality, be a soft rubber or some material that can mold into the pavement texture. Another idea is to increase the frequency of measurements for each project site so the overall permeability average will be more representable.

Another area which should be studied is the anomalies in the data from the skid measurements. There is no obvious explanation as to why the data came out with the "dry" measurements showing a lower skid number than those for the "wet" conditions. A hypothesis for this problem is that the water from the sprayer on the friction tester actually loosens any dirt on the road surface, but does not provide enough water or time to wash it away completely. This would then mix with the road oils and cause a slicker pavement surface.

Splash and spray testing was performed on a number of sites around Corvallis. These tests show a minimum of 10% spray reduction during a high intensity storm. Testing over F-mixes with poorly graded cross-slopes presented problems with drainage and increased spray. Additional testing with the developed device is needed to quantify spray over a range of vehicles operating over a range of speeds.

4.0 LABORATORY STUDY

This chapter presents the results of a laboratory study used to evaluate some porous pavement parameters in a controlled environment. This chapter summarizes the procedures used, the data, acquired, and discusses the significance of the data.

4.1 Core Sampling Plan

The sampling plan used for the projects described in Chapter 3 is summarized in Figure 4.1. Since the Environmental Condition System (ECS) test requires asphalt concrete cores to be of 4 ± 0.16 in (102 ± 4 mm) height, only two projects could be evaluated for these properties, the Pendleton and the N. Grants pass sites. These two sites contained two 2 in (51 mm) thick layers of F-mix, while all other sites were made of only one layer of F-mix. All other sites followed the non-ECS core sampling plan shown in Figure 4.1. All site samples were tested for mix permeability, aggregate gradation, asphalt content and properties, and voids.

4.2 Test Procedures - ODOT

All tests for aggregate gradation, asphalt properties, and asphalt content were performed by ODOT, using their standard test procedures.

4.2.1 Asphalt Recovery

The extraction of asphalt from asphaltic mixtures is covered under Oregon State Highway Division (OSHD, 1989) Test Method 314-86 (TM 34-86). This method is a modified version of AASHTO T164 and T170 designations (AASHTO, 1990). The extraction technique uses a benzene solution conforming to the ASTM D835 Standard Specification of Nitration Grade Benzene for the reagent in the extraction process.

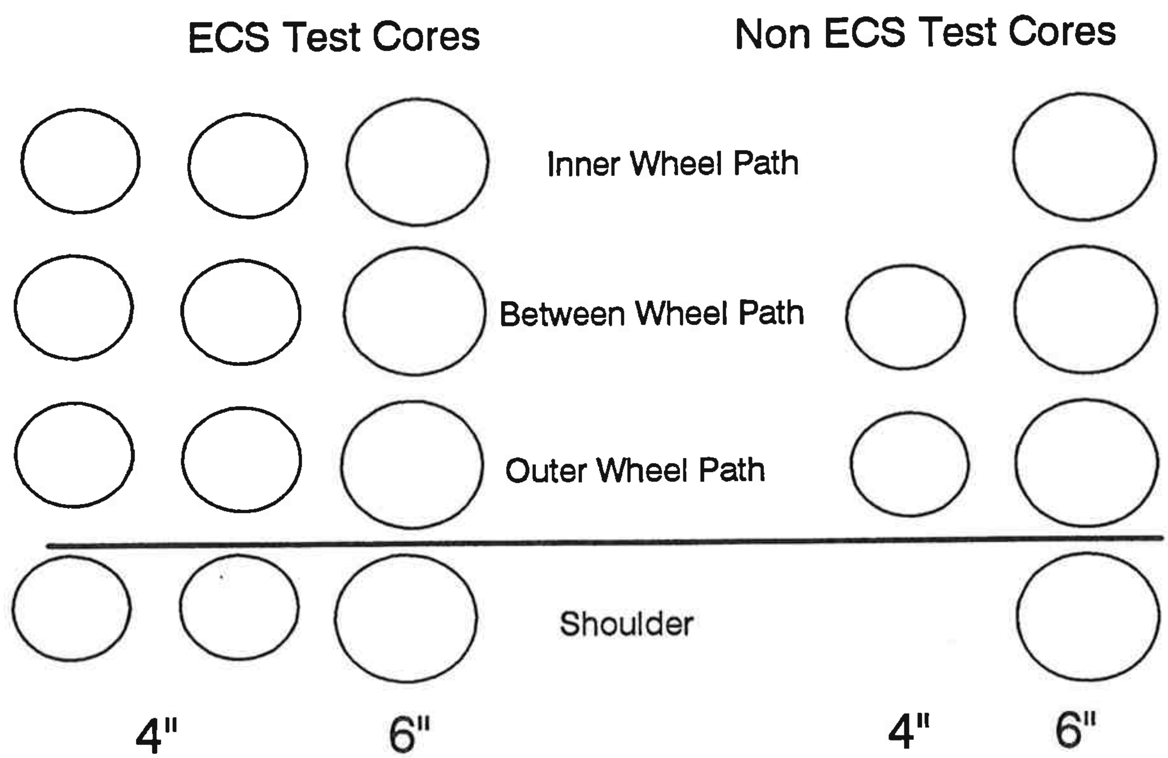


Figure 4.1. Core sampling plan

4.2.2 Asphalt Properties

Three asphalt property tests were performed on the recovered asphalt. These are penetration (TM 401), kinematic viscosity (TM 402), and absolute viscosity (TM 417). The penetration test (Oregon TM 401) is the same method as defined in ASTM D 5-73. All tests were performed at the 25 °C (77°F) test temperature. The absolute viscosity of asphalt (Oregon TM 417) is the same method as defined in ASTM D 2171-78 while the kinematic viscosity of asphalt (TM 402) is the same test method as defined in ASTM D 2170-76 (ASTM, 1993).

4.3 Test Procedures – OSU

OSU performed laboratory tests on the cores for moisture sensitivity and permeability (using the ECS), and for tack coat shear. The ECS test was developed by OSU as part of a research project under the Strategic Highway Research Project (Allen, 1993).

4.3.1 ECS

The Environmental Conditioning System (ECS) was designed to simulate actual water conditioning within the specimen. The ECS test protocol follow the outline shown in Table 4.1. The ECS is made up of three subsystems as shown in Figure 4.2: the fluid conditioning apparatus, environmental conditioning cabinet, and loading system.

The fluid conditioning system was designed to measure air and water permeability and provide water conditioning. This system was designed as a constant head permeameter with pressure gradient measured by three separate gauges. One is connected before the system, the second after the system, and the third is a differential pressure gauge across the system. The fluid conditioning system also includes a thermocouple with four channels that can be used to monitor the water flow temperature before and after flow through the specimen, and the temperature of a dummy specimen in the chamber, and the temperature of the water reservoir. The three water flow meters are

Table 4.1. Summary of the ECS test procedure (after Allen, 1993)

Step	Description
1	Prepare test specimens according to SHRP specimen preparation protocol.
2	Determine the geometric and volumetric properties of the specimen.
3	Encapsulate specimen in silicon sealant and latex rubber membrane.
4	Place the specimen in the ECS load frame, and determine air permeability.
5	Determine unconditioned (dry) triaxial resilient modulus.
6	Vacuum condition specimen (subject to vacuum of 20 in (508 mm) Hg for 10 minutes).
7	Wet specimen by pulling distilled water through specimen for 30 minutes using a 20 in (508 mm) Hg vacuum.
8	Determine unconditioned water permeability.
9	Heat the specimen to 140°F (60°C) for six (6) hours under repeated loading. This is a hot cycle.
10	Cool the specimens to 77°F (25°C) for at least four (4) hours. Measure triaxial resilient modulus and water permeability.
11	Repeat steps 9 and 10 for two (2) more hot cycles.
12	Cool the specimen to 0°F (-18°C) for six (6) hours, without repeated loading. This is a freeze cycle.
13	Heat the specimen to 77°F (25°C) for at least four (4) hours and measure the triaxial resilient modulus and the water permeability.
14	Split the specimen and perform a visual evaluation of stripping and binder migration.
15	Plot the ECS resilient modulus ratio.

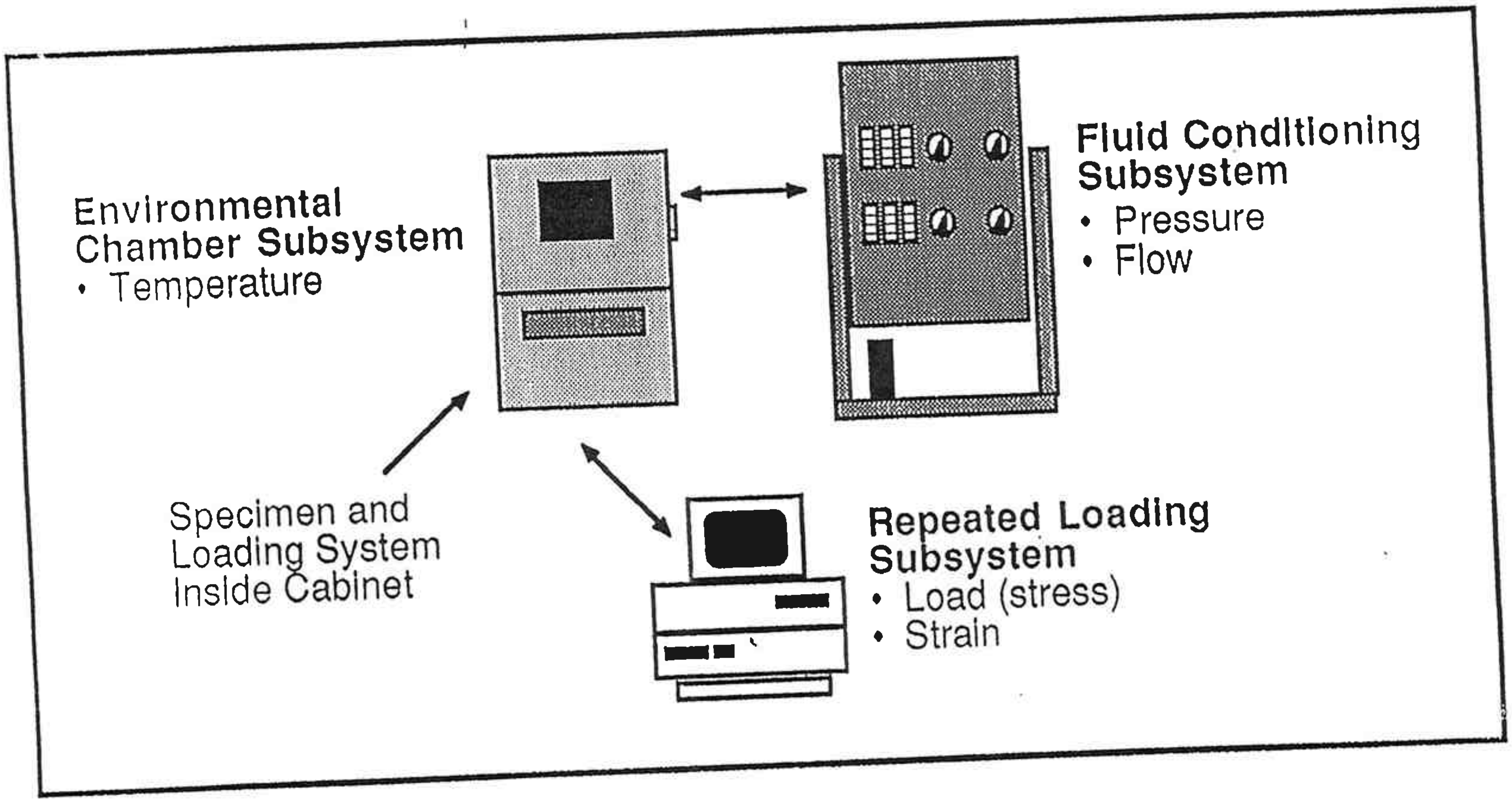


Figure 4.2. Overview of the Environmental Conditioning System (After Allen, 1993)

connected to the water conditioning system to provide a sufficient flow range from 1 to 3000 cm^3/min, and three air flow meters that can read a range of 100 to 70000 cm^3/min for measuring specimen air permeability. The coefficient of permeability, k can be calculated from these measurements using the procedure outlined in Appendix E.

Figure 4.3 shows a schematic of the loading system used with the ECS. This system is connected to a personal computer that controls the test through a controller card. The servovalve drives the system by controlling the pressure of the compressed air. Loads are delivered to the system through a load ram and load cell system that rests on top of the specimen. The deflections are monitored by linearly variable differential transducers (LVDTs), mounted on the specimen, and allowing calculation of the resilient modulus of the specimen. The tests are conducted using a haversine pulse load of duration 0.1 s and frequency of 1 Hz.

The testing protocol for the ECS requires that an environmental conditioning cabinet be used that is capable of heating to 100°C and cooling to -20° C within a tolerance of ± 1 °C. Temperature changes and time limits are specified in the protocol as well.

4.3.2 Shear Testing

4.3.2.1 Specimen Preparation. The laboratory shear test was performed to measure the shear strength of a tack coat placed between a portland cement concrete (PCC) pavement and a F-mix layer. The test specimens were constructed using the rolling wheel compactor developed at OSU through the Strategic Highway Research Program (SHRP) (Terrel et al., 1993). Figure 4.4 illustrates the rolling wheel compaction procedure used. The procedure is briefly summarized below.

A concrete section measuring 28 × 28 in (710 × 710 mm) and 3 in (76 mm) in height was poured in a mold. This concrete slab was extracted after curing for 10 days and placed in the 5 in (130 mm) rolling wheel device. A tack coat of CSS-1 emulsion was then placed on the concrete layer following ODOT specification section 00730 (ODOT, 1991). Application rates used were 0.05,

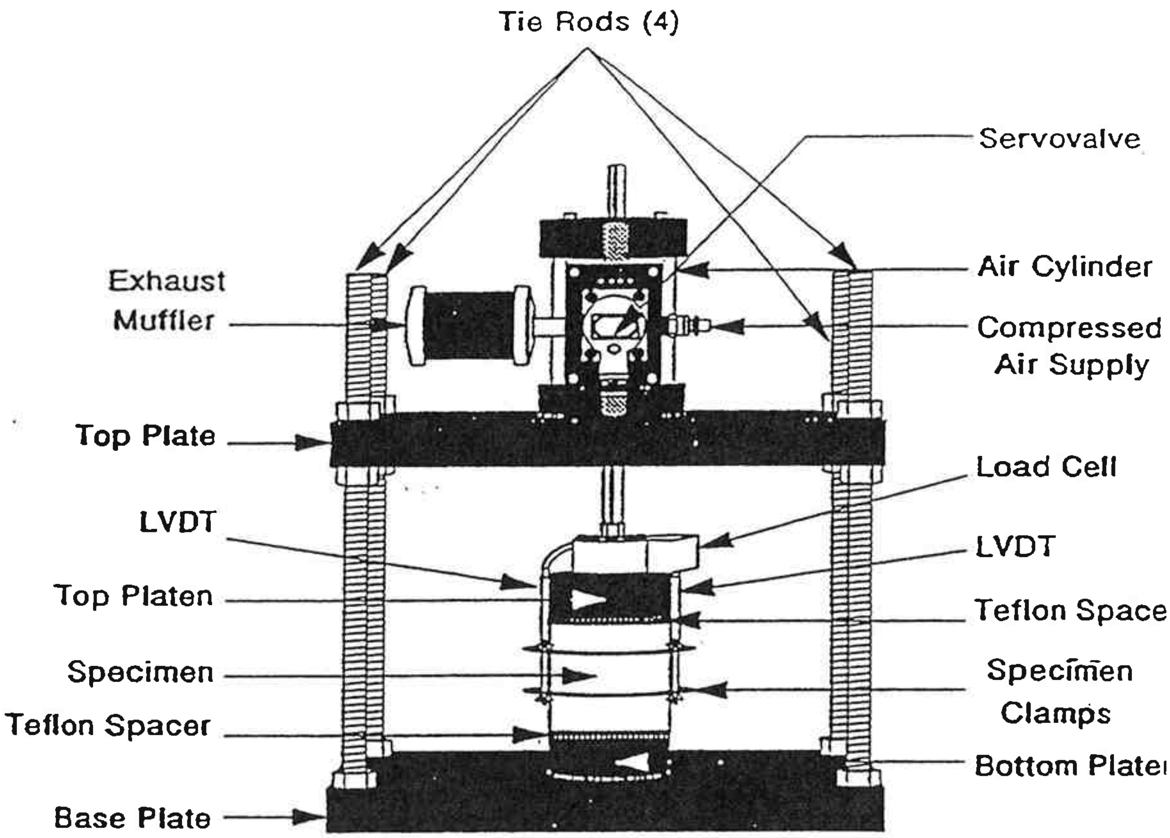


Figure 4.3. Schematic of ECS load frame (after Allen, 1993)

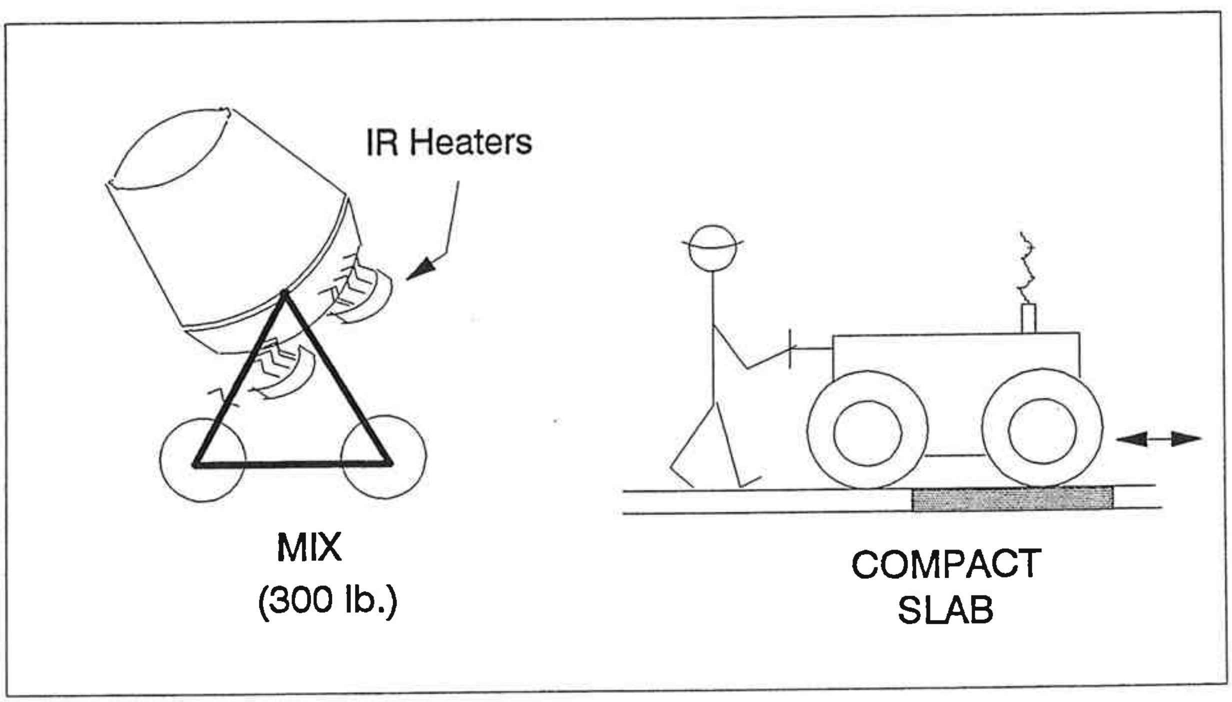


Figure 4.4. Rolling wheel compaction setup

0.10, 0.15, and 0.20 gal/yd^2 (0.23, 0.45, 0.68, and 0.91 l/m^2). The tack coat was then allowed cure until the water "broke" from the asphalt but still retained its tackiness, as defined in the ODOT specification. The F-mix asphalt layer placed on the tack coat surface in accordance to the rolling wheel compaction method.

Specimens were extracted from the 5 in (13 cm) slab of F-mix over PCC with a 4 in (10 cm) core barrel. From each section of emulsion spray rates, 5 samples were extracted. These samples were then set aside for testing. Two samples from each group were subject to some long term aging for 48 days at 85°C in a force draft oven. This was performed to simulate the conditions the samples would go through while aging in the field. Two other samples were set aside for normal testing. The fifth sample was produced as a backup in case there were testing problems with the samples, and more tests had to be performed.

4.3.2.2 Testing Methods. The samples were tested in a tensile shear mode using the schematic of the device shown in Figures 4.5 and 4.6 and the photo shown in Figure 4.7. The specimens were subject to a shear force at a rate of 10 lb/s (4.5 N/s) along the tack coat bond until failure. The test results were recorded using an X-Y plotter on a graph sheet. An example of the graphical results are shown in Figure 4.8.

4.4 Test Results – Field Cores

4.4.1 Aggregate Gradation/Asphalt Content/Voids

Tables 4.2 through 4.6 summarize the results from this portion of the project: the core results along with the initial job mix formula (JMF) were produced in ODOT's labs for each project. Figures 4.9 through 4.12 provide a graphical representation of how the gradation between the job mix formula (JMF) and the field cores differ. There seems to be little or no change in the gradations. The small gradation change is not sufficient to suggest any excess pore filling during

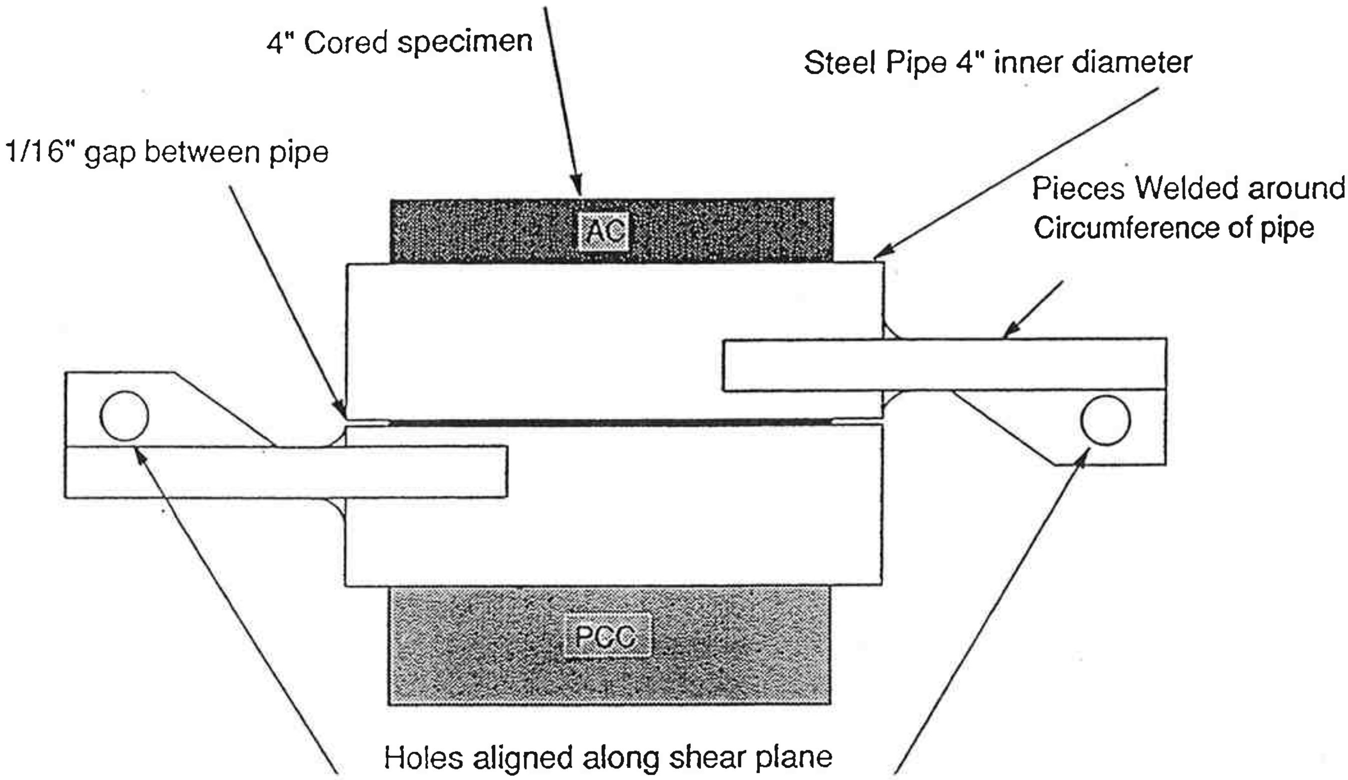


Figure 4.5. Shear testing device – front view (1 in = 25.4 mm)

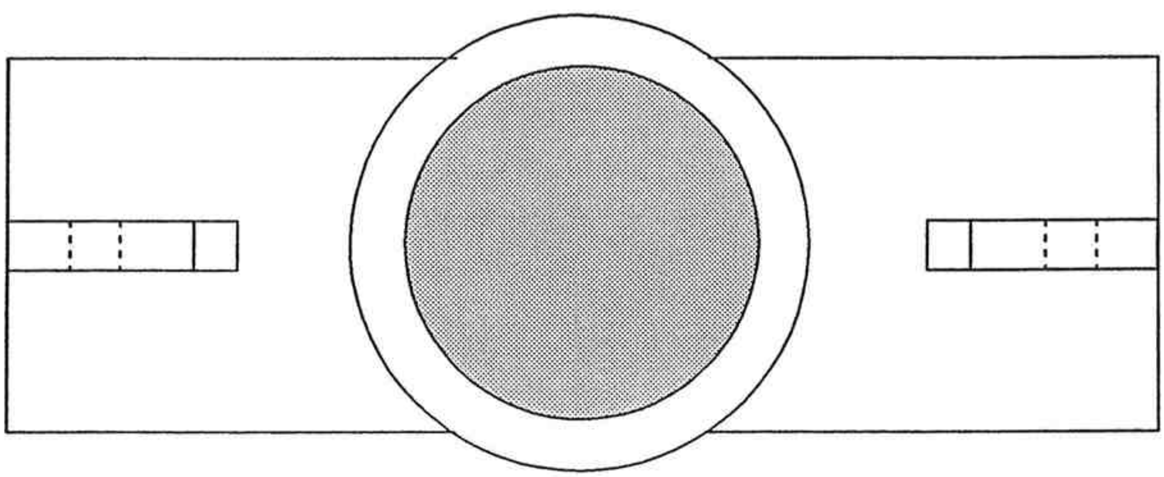


Figure 4.6. Shear testing device – top view

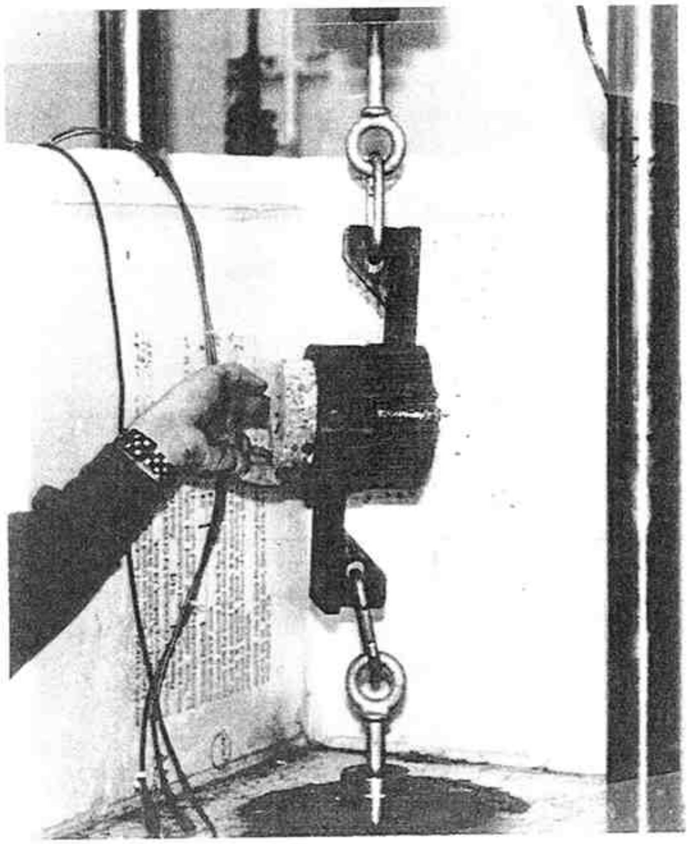


Figure 4.7. Photo of shear test setup

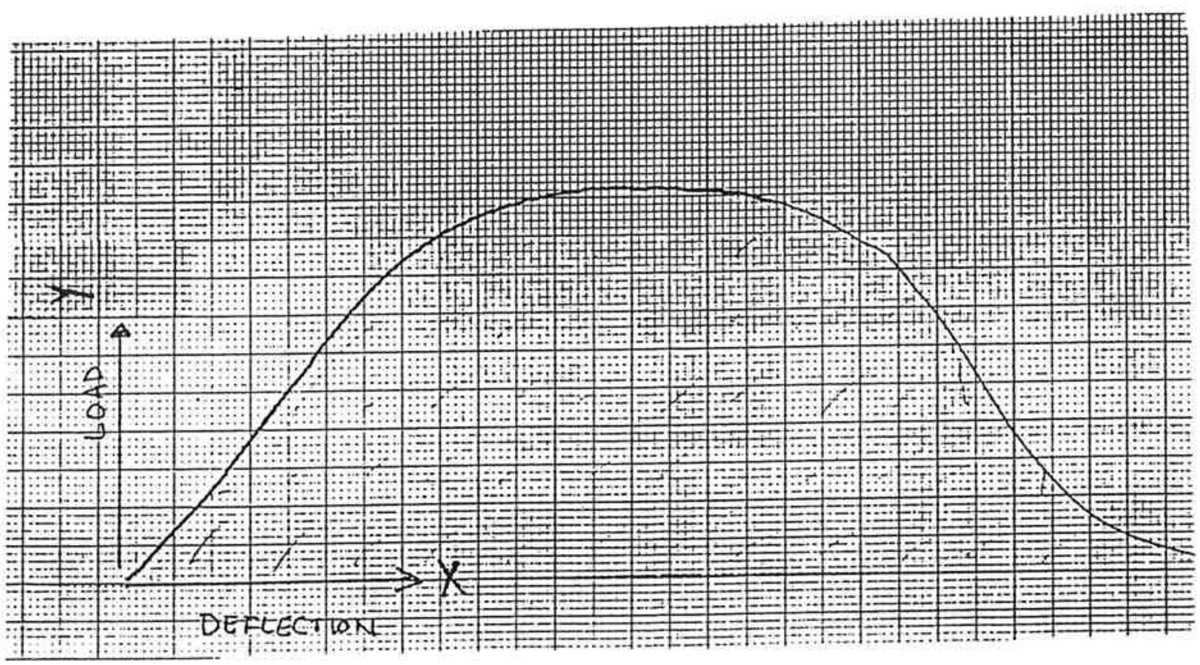


Figure 4.8. X-Y plot of test results

Table 4.2. Core data for Murphy Road to Lava Butte

Milepost	141.79	141.79	141.79	141.79	Job Mix Formula
Core Location	OWT	IWT	BWT	Shoulder	
Gradation					
1" (25.4 mm)	100	Combined with OWT for test	100	Combined with BWT for test	100
3/4" (19.0 mm)	98		100		98
1/2" (12.5 mm)	81		81		75
3/8" (9.5 mm)	59		59		56
1/4" (6.35 mm)	30		32		25
#4 (4.75 mm)	21		22		–
#10 (2.00 mm)	12		14		9
#40 (0.41 mm)	6		7		4
#200 (0.075 mm)	1.3		1.1		2.7
Bulk Specific Gravity	2.13	1.93	2.04	1.93	2.36
Rice Specific Gravity (JMF)	2.535	2.535	2.535	2.535	2.535
Air Voids (%)	16.0	23.9	19.5	23.9	6.9
Asphalt Cement Information					
Asphalt Content (%)	4.2	Combined with OWT for test	4.2	Combined with OWT for test	5.0
Penetration, 25°C (dmm)	50		30		Chevron
Kinematic Viscosity, 135°C (cS)	806		1179		MAC-45
Absolute Viscosity, 60°C (P)	6710		15450		Lime Treat

Table 4.3. Core data for Jumpoff Joe to N. Grants Pass

Milepost	61.38	61.38	61.38	61.38	Job Mix Formula
Core Location	BWP	OWP	IWT	Shoulder	
Gradation					
1" (25.4 mm)	100	100	100	100	100
3/4" (19.0 mm)	93	93	94	92	94
1/2" (12.5 mm)	65	68	66	67	66
3/8" (9.5 mm)	47	48	45	47	39
1/4" (6.35 mm)	27	28	26	27	24
#4 (4.75 mm)	20	22	21	22	–
#10 (2.00 mm)	13	15	14	14	12
#40 (0.41 mm)	8	9	8	8	7
#200 (0.075 mm)	4	4.3	4.3	4.1	3.9
Bulk Specific Gravity	2.2	2.21	2.24	2.15	2.46
Rice Specific Gravity (JMF)	2.635	2.635	2.635	2.635	2.635
Rice Specific Gravity (ECS Cores)	2.616	2.616	2.616	2.616	NA
Air Voids (%)	16.5	16.1	15.0	18.4	6.6
Air Voids (%)	15.9	15.5	14.4	17.8	6.6
Asphalt Cement Information					
Asphalt Content (%)	4.3	5.1	4.8	5	5
Penetration, 25°C (dmm)	32	37	41	39	Chevron PBA-5
Kinematic Viscosity, 135°C (cS)	740	736	696	715	0.5% PaveBond
Absolute Viscosity, 60°C (P)	7330	7410	7030	6850	Lime Treat

Table 4.4. Core data for E. Pendleton to Emigrant Hill

Milepost	215	215	215	215	Job Mix Formula
Core Location	OWT	Shoulder	IWT	BWT	
Gradation					
1" (25.4 mm)	100	Combined with OWT for test	100	Combined with IWT for test	100
3/4" (19.0 mm)	100		96		95
1/2" (12.5 mm)	70		64		65
3/8" (9.5 mm)	51		46		43
1/4" (6.35 mm)	32		28		26
#4 (4.75 mm)	24		22		-
#10 (2.00 mm)	13		12		12
#40 (0.41 mm)	5		5		6
#200 (0.075 mm)	1.7		2.5		3.2
Bulk Specific Gravity	2.07	1.98	2.13	2.04	2.153
Rice Specific Gravity (JMF)	2.493	2.493	2.493	2.493	2.493
Rice Specific Gravity (ECS Core)	2.500	2.500	2.500	2.500	
Air Voids (%) (JMF)	17.0	20.6	14.6	18.2	13.6
Air Voids (%) (ECS Core)	17.2	20.8	14.8	18.4	6.9
Asphalt Cement Information					
Asphalt Content (%)	4.2	Combined with OWT for test	3.2	Combined with IWT for test	6.0
Penetration, 25°C (dmm)	47		59		Columbia
Kinematic Viscosity, 135°C (cS)	1141		938		PBA-6
Absolute Viscosity, 60°C (P)	8070		5270		Lime Treat

Table 4.5. Core data for Hayesville to BattleCreek

Milepost	150.73	150.73	150.73	150.73	Job Mix Formula
Core Location	OWT	Shoulder	IWT	BWT	
Gradation					
1" (25.4 mm)	100	Combined with OWT for test	100	Combined with IWT for test	100
3/4" (19.0 mm)	97		96		93
1/2" (12.5 mm)	68		66		67
3/8" (9.5 mm)	45		44		43
1/4" (6.35 mm)	28		27		23
#4 (4.75 mm)	24		23		-
#10 (2.00 mm)	17		17		10
#40 (0.41 mm)	10		11		5
#200 (0.075 mm)	5.2		6		2.4
Bulk Specific Gravity	1.91	2.17	2.11	1.65	2.28
Rice Specific Gravity (JMF)	2.469	2.469	2.469	2.469	2.469
Air Voids (%)	22.6	12.1	14.5	33.2	7.6
Asphalt Cement Information					
Asphalt Content (%)	4.1	Combined with OWT for test	4.1	Combined with IWT for test	5.5
Penetration, 25°C (dmm)	34		48		Chevron AC-30
Kinematic Viscosity, 135°C (cS)	1260		994		Lime Treat
Absolute Viscosity, 60°C (P)	32000		21400		

Table 4.6. Core data for Crater Lake Highway

Milepost	83.1	83.1	83.1	83.1	Job Mix Formula
Core Location	IWT	BWT	Shoulder	OWT	
Gradation					
1" (25.4 mm)	100	100	100	100	
3/4" (19.0 mm)	100	95	95	95	Specifications not known
1/2" (12.5 mm)	83	81	75	79	
3/8" (9.5 mm)	63	62	60	62	
1/4" (6.35 mm)	38	38	37	38	
#4 (4.75 mm)	24	25	24	24	
#10 (2.00 mm)	14	14	13	14	
#40 (0.41 mm)	11	10	9	10	
#200 (0.075 mm)	5.8	5.2	5	5.4	
Bulk Specific Gravity	1.84	1.9	1.88	1.91	
Rice Specific Gravity (JMF)					
Air Voids (%)					
Asphalt Cement Information					
Asphalt Content (%)	2.5	2.8	2.7	2.6	
Penetration, 25°C (dmm)	23	Not enough for test	15	19	
Kinematic Viscosity, 135°C (cS)	1080		7403	1430	
Absolute Viscosity, 60°C (P)	18300		39800	29800	

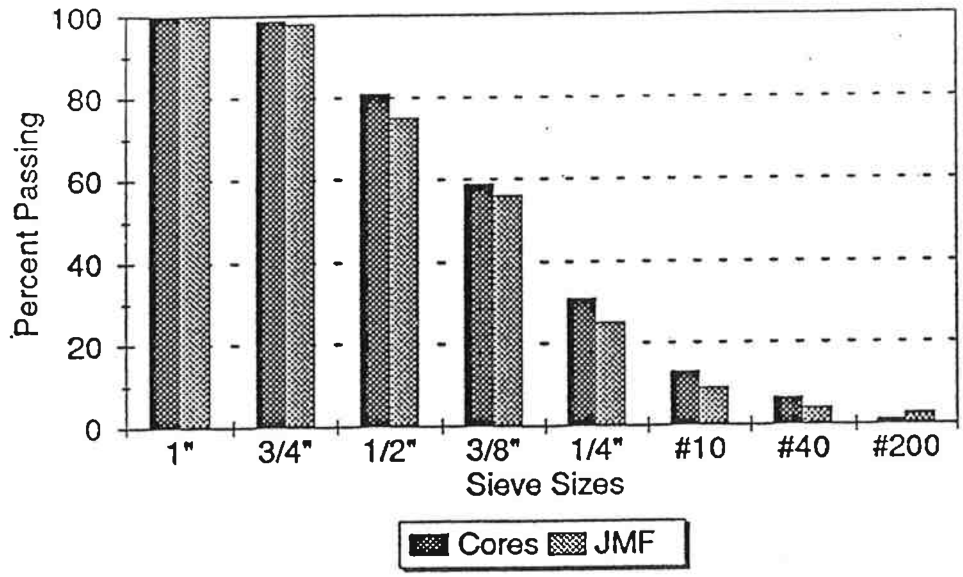


Figure 4.9. Murphy Road to Lava Butte gradation changes (1 in = 25.4 mm)

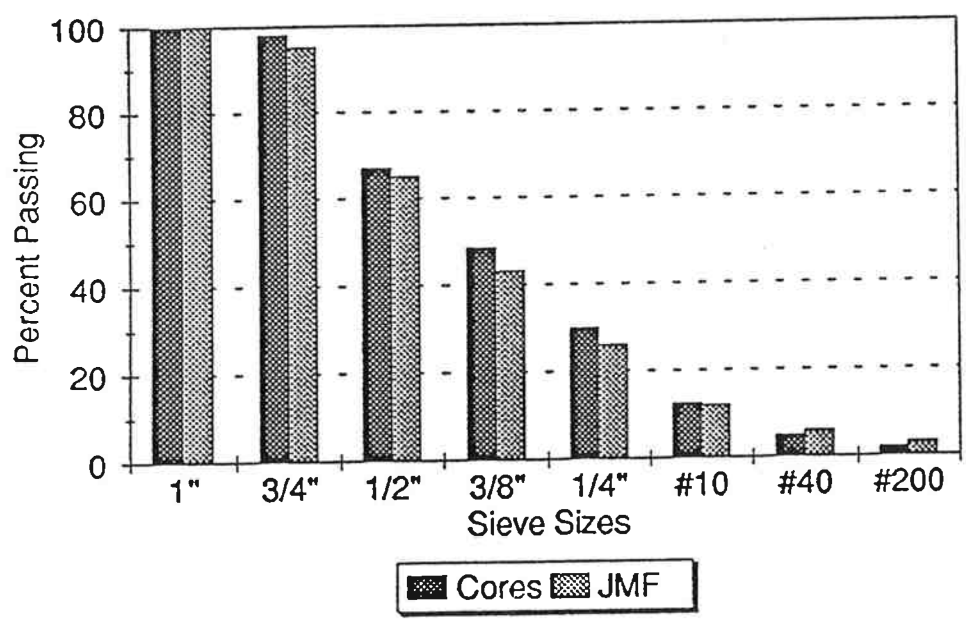


Figure 4.10. Jumpoff Joe to N. Grants Pass gradation changes (1 in = 25.4 mm)

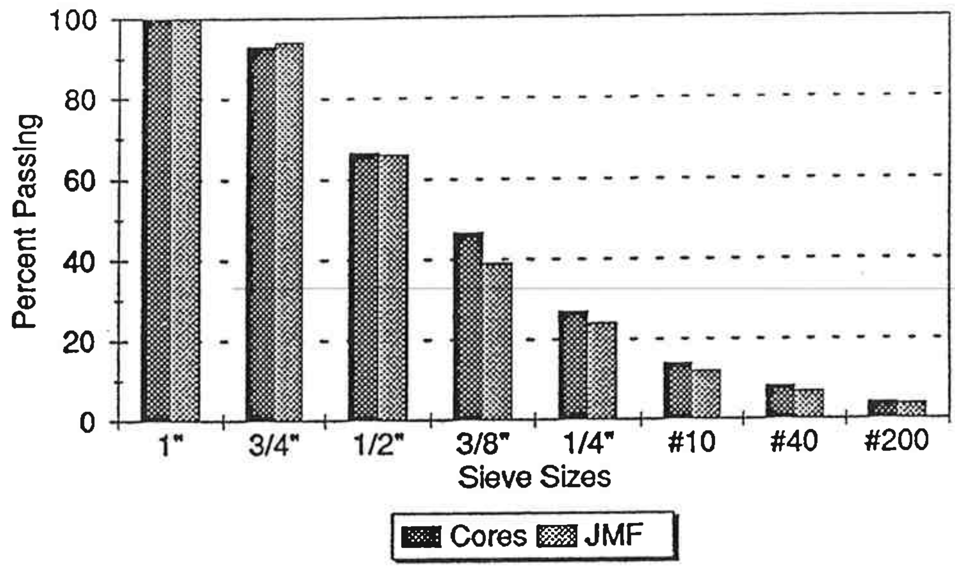


Figure 4.11. E. Pendleton to Marquam Hill gradation changes (1 in = 25.4 mm)

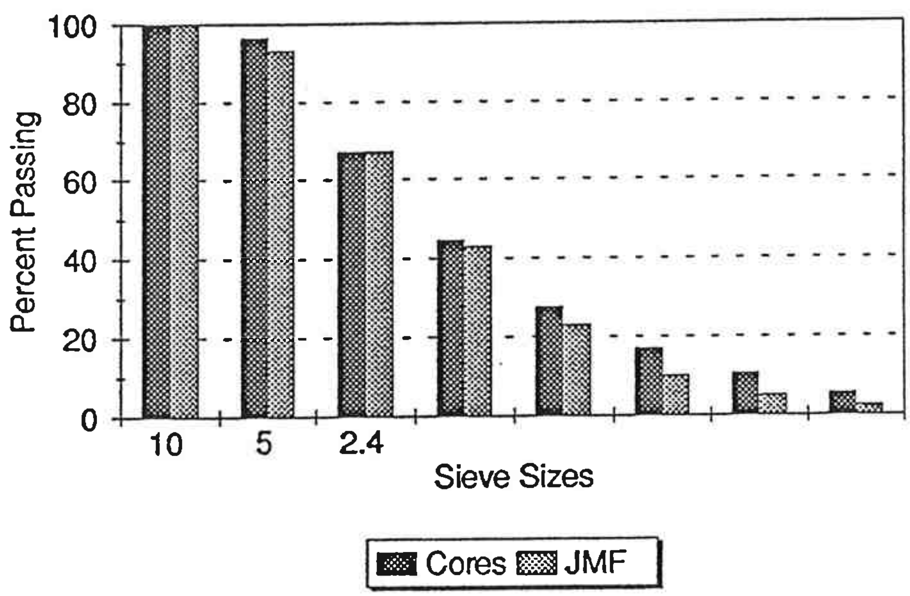


Figure 4.12. Hayesville to BattleCreek gradation changes (1 in = 25.4 mm)

the life of the project. It would seem to make sense, however, that the gradations for the No. 10 minus sizes would increase in percent passing for the field cores, due to particle infiltration into the pores of the pavement. The pumping effect of tires sucking out the small particles has kept the voids clean. If this were the case, one could hypothesize that the gradations for the BWT and shoulder cores would actually have a higher number of fines, as these pavement areas are not subject to the fines being pumped as readily as inside the wheel tracks. However, this phenomenon was not evident from the data.

4.4.2 ECS

The results of the ECS testing are shown in Tables 4.7 and 4.8. The five areas of data reported are ECS MR modulus, ECS MR ratio, visual stripping, coefficient of permeability for water, and coefficient of permeability for air.

The ECS MR modulus and ratio are the main focus of the pass/fail criteria for the ECS test procedure. For ECS testing of porous pavements the failure criteria of a sample is defined as a ratio of less than 0.75 (Terrel et al., 1993). The ratio is the ECS modulus for the cycle divided by the ECS modulus for the initial conditions. Figures 4.13 and 4.14 provide a graphical representation of the ECS modulus ratio for each cycle. If the 0.75 ratio failure criteria is used, it would appear that the samples P1PP, P2PP, P4PP, P5PP, P6PP, J3PP, and J5PP all failed the test. There would appear to be some possible water sensitivity problems for the Pendleton project, since 5 out of 7 of the core samples exhibited water sensitivity after being in the field a short while (project ended in spring of 1993, and samples taken in summer of 1993).

The I-5 (Jumpoff Joe - N. Grants Pass) project exhibited potential water sensitivity on only 2 out of 8 samples for a pavement that was a year older than the Pendleton project. The pavement at the I-84 (E. Pendleton- Emigrant Hill) project may have problems with stripping in the future.

Table 4.7. ECS results: I-5 (Jumpoff Joe to N. Grants Pass)

ECS Cycle	Sample ID	ECS Modulus (ksi) (MPa)	ECS MR	Stripping (%)	Coefficient of Permeability K (cm/s) (water)	Coefficient of Permeability K (cm/s) (air)
Initial	J1PP	229.2 (1580)	1.00	0 - 5	1.03E-03	3.8E-05
First		226.0 (1558)	0.99		7.66E-04	
Second		222.1 (1531)	0.97		1.08E-03	
Third		271.1 (1869)	1.18		9.58E-04	
Initial	J2PP	370.7 (2556)	1.00	0 - 5	5.80E-04	Impermeable
First		304.1 (2097	0.82		6.31E-04	
Second		294.3 (2029)	0.79		7.84E-04	
Third		291.3 (2008)	0.79		4.48E-04	
Initial	J3PP	720.7 (4969)	1.00	5 - 10	1.05E-03	Impermeable
First		414.2 (2856)	0.57		5.21E-04	
Second		412.0 (2840)	0.57		1.19E-03	
Third		413.4 (2850)	0.57		9.01E-04	
Initial	J4PP	212.3 (1464)	1.00	5 - 10	2.05E-03	2.3E-05
First		181.1 (1249)	0.85		2.41E-03	
Second		171.5 (1182)	0.81		2.21E-03	
Third		173.5 (1196)	0.82		2.73E-03	
Initial	J5PP	489.0 (3372)	1.00	0 - 5	3.40E-05	Impermeable
First		347.1 (2393)	0.71		4.37E-03	
Second		369.1 (2545)	0.75		3.08E-03	
Third		370.0 (2551)	0.76		4.64E-03	
Initial	J6PP	492.7 (3397)	1.00	0 - 5	4.44E-03	5.9E-05
First		316.5 (2182)	0.64		6.06E-03	
Second		292.7 (2018)	0.89		1.49E-03	
Third		305.2 (2104)	0.62		5.72E-03	
Initial	J7PP	307.1 (2117)	1.00	5 - 10	1.38E-03	5.6E-05
First		263.1 (1814)	0.86		1.22E-03	
Second		260.1 (1793)	0.85		1.22E-03	
Third		254.4 (1754)	0.83		8.88E-04	
Initial	J8PP	405.5 (2796)	1.00	5 - 10	2.96E-03	5.7E-05
First		350.0 (2413)	0.86		1.88E-03	
Second		334.0 (2303)	0.82		2.42E-03	
Third		336.7 (2321)	0.83		4.29E-03	

Table 4.8. ECS results: I-84 (E. Pendleton to Emigrant Hill)

ECS Cycle	Sample ID	ECS Modulus (ksi) (MPa)	ECS MR	Visible Stripping (%)	Coefficient of Permeability K (cm/s) (water)	Coefficient of Permeability K (cm/s) (air)
Initial	P1PP	522.8 (3605)	1.00	0 - 5	1.04E-03	5.5E-05
First		272.3 (1877)	0.52		1.42E-03	
Second		288.7 (1991)	0.55		1.45E-03	
Third		245.5 (1693)	0.47		1.32E-03	
Initial	P2PP	305.3 (2105)	1.00	5 - 10	2.86E-03	6.0E-05
First		185.1 (1276)	0.61		2.56E-03	
Second		203.7 (1404)	0.67		2.90E-03	
Third		172.6 (1190)	0.57		1.76E-03	
Initial	P4PP	280.3 (1933)	1.00	0 - 5	7.88E-04	4.8E-05
First		153.4 (1058)	0.55		1.10E-03	
Second		197.2 (1360)	0.70		9.14E-04	
Third		183.9 (1268)	0.66		1.15E-03	
Initial	P5PP	325.5 (2244)	1.00	5 - 10	1.47E-03	6.7E-05
First		265.0 (1827)	0.81		1.47E-03	
Second		208.0 (1434)	0.64		1.25E-03	
Third		216.5 (1493)	0.67		1.99E-03	
Initial	P6PP	226.0 (1558)	1.00	0 - 5	1.17E-03	4.3E-05
First		180.0 (1241)	0.81		1.64E-03	
Second		155.0 (1069)	0.64		1.82E-03	
Third		172.5 (1189)	0.67		1.90E-03	
Initial	P7PP	269.1 (1855)	1.00	0 - 5	2.31E-03	6.9E-05
First		213.5 (1472)	0.80		2.04E-03	
Second		199.4 (1375)	0.69		2.95E-03	
Third		206.4 (1423)	0.76		2.35E-03	
Initial	P8PP	196.6 (1356)	1.00	5 - 10	3.44E-03	5.7E-05
First		146.7 (1011)	0.79		2.66E-03	
Second		147.2 (1015)	0.74		1.94E-03	
Third		157.0 (1082)	0.77		2.34E-04	

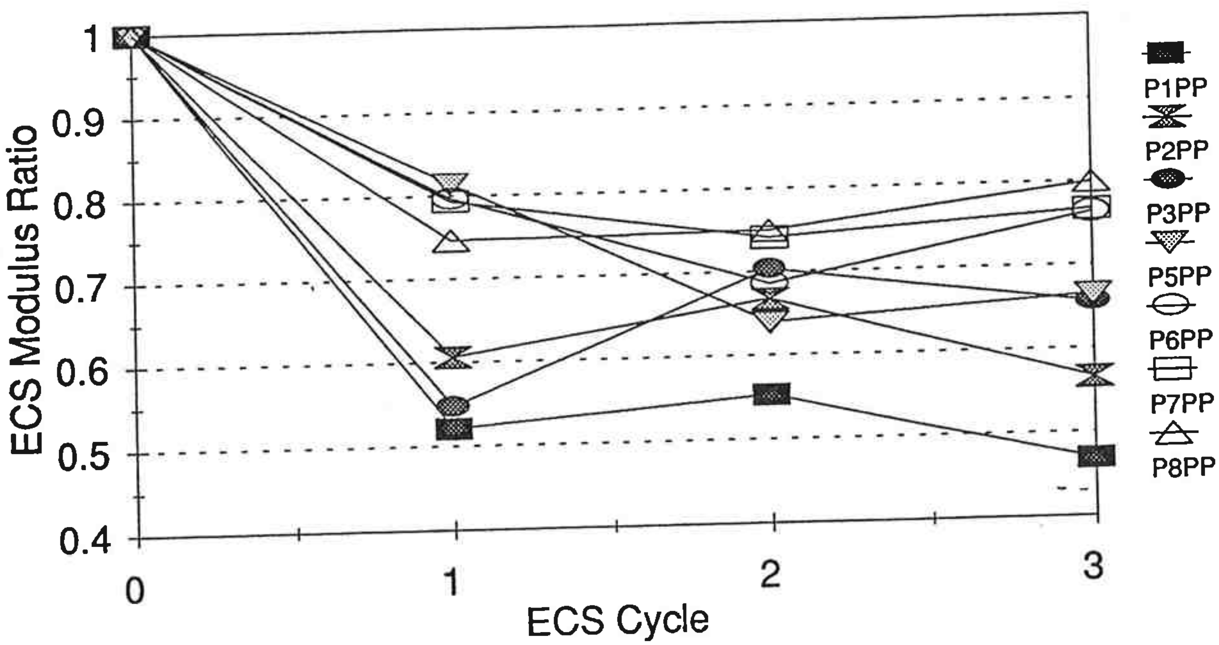


Figure 4.13. Pendleton ECS modulus changes

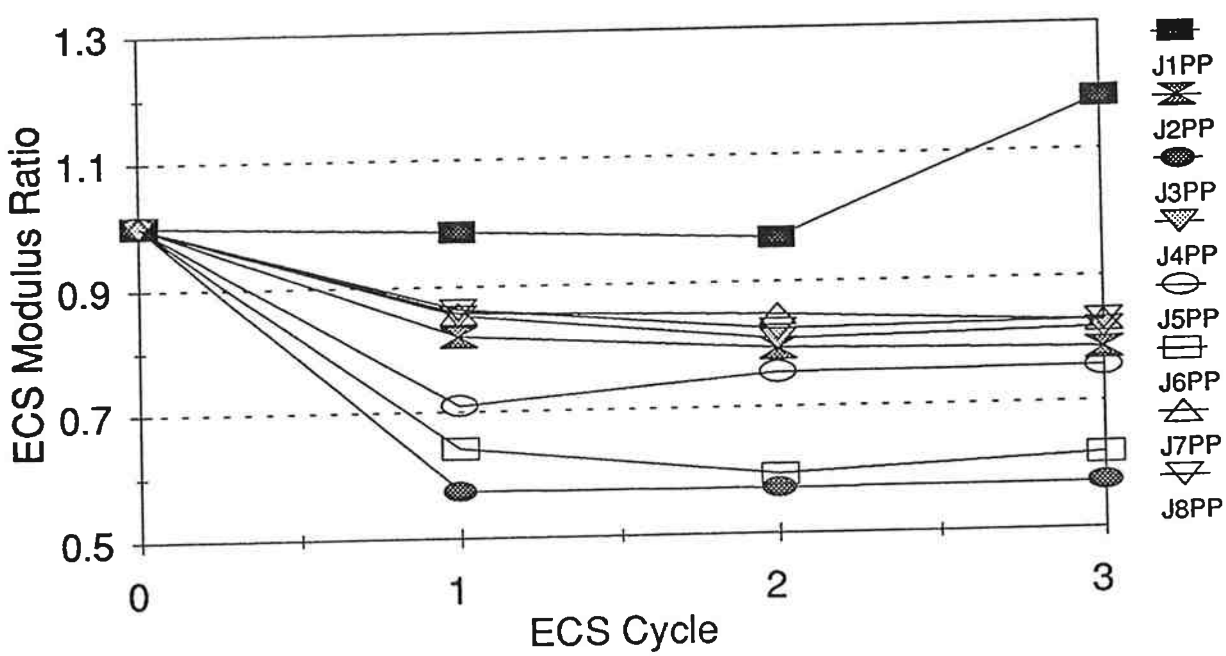


Figure 4.14. Jumpoff Joe ECS modulus changes

Degree of Visual Stripping was also measured from the ECS cores. After a sample had been subject to all four cycles of the ECS test procedure, it was split open diametrally and the stripping of the asphalt from the aggregate was checked visually using the degree of stripping guidelines shown in Figure 4.15. The visual stripping is measured in quantities of severeness where 0-5 means that there was zero to five percent stripping noticeable upon examination. Visual stripping is a subjective measurement and therefore small changes in actual stripping may not be distinguishable. The results for the visual stripping are shown alongside the rest of the ECS data in Tables 4.7 and 4.8. It would make sense that a sample which has shown some problem due to water sensitivity from the ECS test would have more bond loss between the asphalt and aggregate and thus more stripping. Surprisingly, this does not seem to be so when looking at the data. The Jumpoff Joe - N. Grants pass project displayed more water sensitive samples, yet had 4 out of 8 samples showing stripping in the 5-10 percent range. The E. Pendleton - Emigrant Hill project had 5 out of 7 samples exhibit water sensitivity as a result of the ECS modulus, yet had only 3 out of 7 samples with stripping in the 5-10 percent range.

Figures 4.16 and 4.17 provide a graphical representation of the water permeability tests performed on the ECS specimen. The calculation method suggested by Allen (1993) was used to calculate the coefficient of permeability k in cm/s. The figures display the results for the specimen and any changes per cycle can easily be noted by watching the trends on the graphs. The graphs show how sporadic the k values are for these data. As stated by Allen (1993), the plumbing of the ECS permeameter does not provide true permeability values. However, the value of k can be of use in understanding how the water permeability of a sample changes as the water sensivity either increases or decreases.

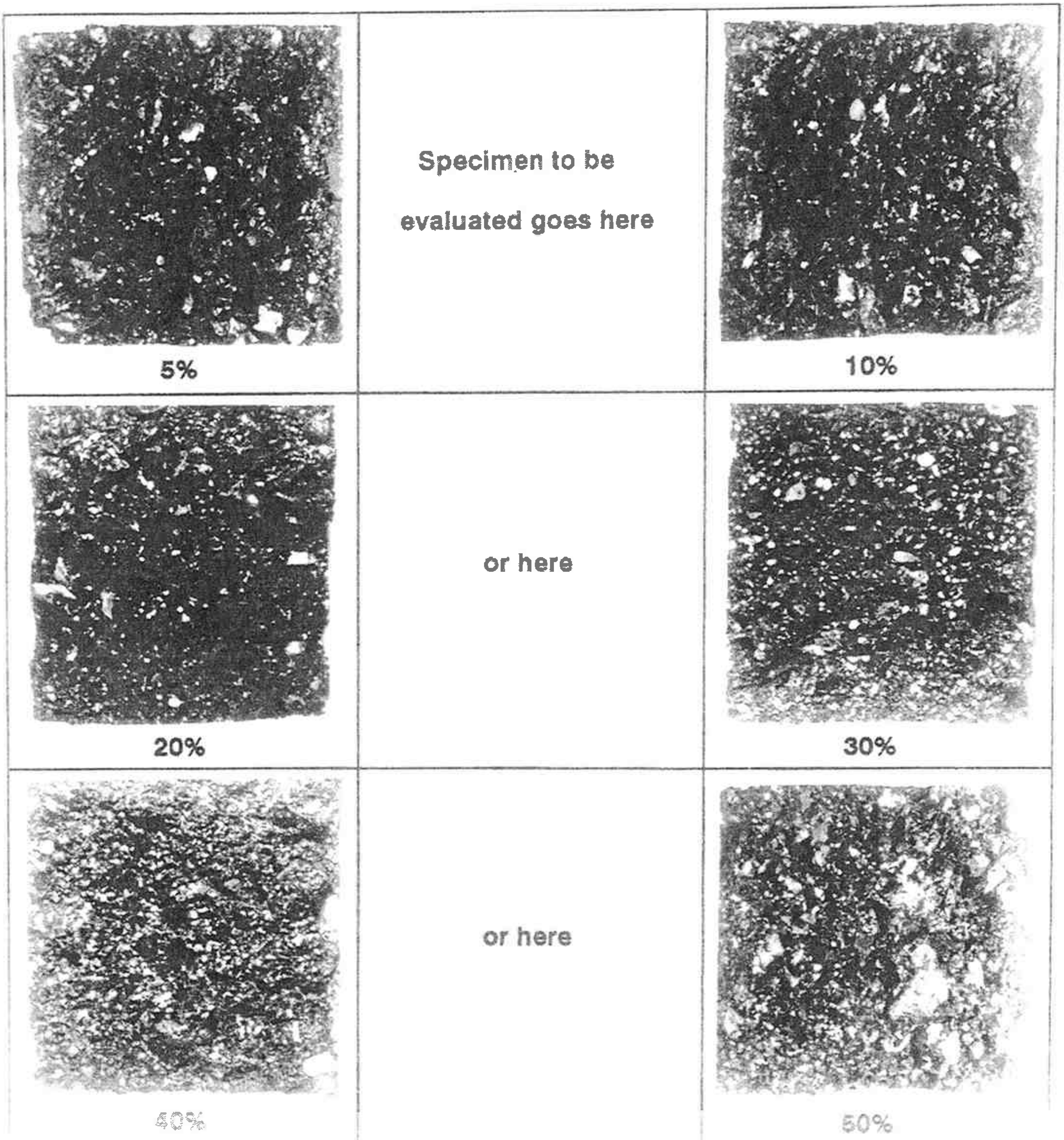


Figure 4.15. Visual stripping guidelines (after Allen, 1993)

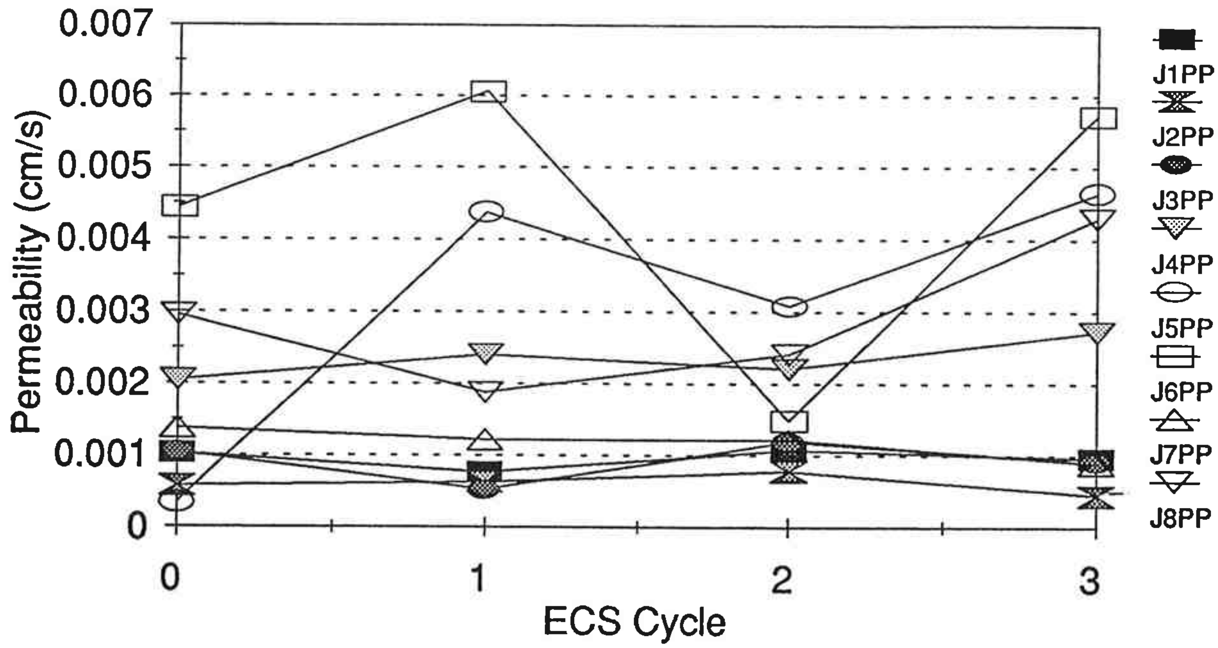


Figure 4.16. Water permeability changes Jumpoff Joe

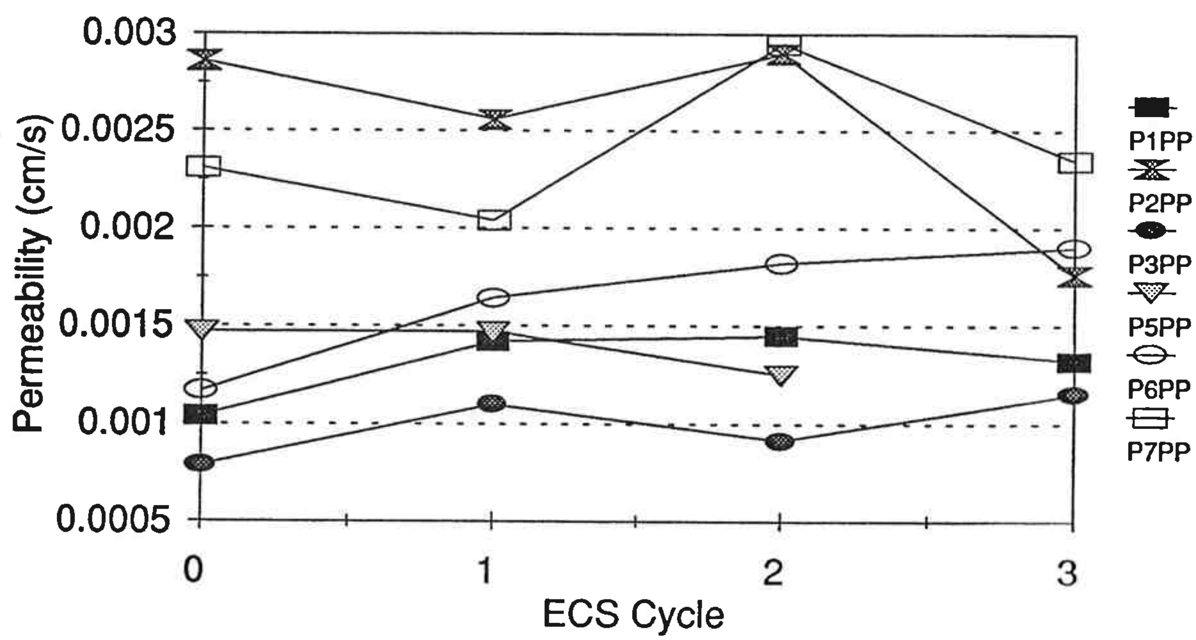


Figure 4.17. Water permeability changes Pendleton

4.4.3 Laboratory Shear Test

Data from the shear test were analyzed for three factors: 1) load failure; 2) amount of shear until failure; and 3) the total energy required for failure. Table 4.9 presents the summary for this data. For each tack coat rate, two tests were performed on the unaged and two on the aged specimens. Figures 4.18 and 4.19 present this data in a graphical mode, where the values from the repeated tests were averaged. These graphs display how the unaged and aged specimens reacted during the test. The increased stiffness of the binder tack coat on the aged specimen is easily shown here, through the increased energy required to shear the specimen, and the higher shear force. It would appear from both the aged and unaged specimen that the 0.10 gal/yd^2 (0.45 l/m^2) spray rate provided the most shear resistance in all tests except for the maximum shear load for the aged specimen. The total failure strain for the aged spray rates are surprisingly equal, around 0.8 in(20 mm). Figure 4.20 shows how the small amount of energy required to shear the unaged 0.15 and 0.20 gal/yd^2 (0.68 and 0.91 l/m^2) specimens shows how high traffic areas with lots of load energy could fail quicker at spray rates other than 0.10 gal/yd^2 (0.45 l/m^2).

4.5 Summary of Results

The results from the laboratory data provide insight into the properties of porous mixes. Significant findings from ECS testing, core gradation, asphalt properties, and tack coat shear testing are discussed below.

The ECS tests show that the pavements for the E. Pendleton - Emigrant Hill project might have some future water sensitivity problems. This would seem to agree with the JMF design, as the Pendleton project showed low Index of Retained Strength (IRS) values. The JMF IRS data for the Jumpoff Joe project provided an IRS above the 70% line which concurs with the ECS results.

Table 4.9. Results of shear testing experiment

Tack Coat Rate		Condition Aged (Y or N)	Failure Load		Max Shear @ Max Load		Total Energy	
(gal/yd²)	(ℓ/m²)		(lb)	(kN)	(in)	(cm)	(lb • n)	(kN • n)
0.05	0.23	N	108	400	0.95	2.4	2300	403
0.05	0.23	N	106	470	0.75	1.9	1500	266
0.1	0.45	N	125	560	0.95	2.4	2500	432
0.1	0.45	N	123	550	0.95	2.4	2400	424
0.15	0.68	N	83	370	0.70	1.8	1100	194
0.15	0.68	N	64	280	0.44	1.1	330	58
0.2	0.91	N	62	270	0.70	1.8	460	81
0.2	0.91	N						
0.05	0.23	Y	172	760	0.92	2.3	3400	602
0.05	0.23	Y	180	800	0.72	1.8	2400	421
0.1	0.45	Y	161	710	0.82	2.1	2800	492
0.1	0.45	Y	176	780	0.92	2.3	3600	638
0.15	0.68	Y	150	670	0.91	2.3	3300	575
0.15	0.68	Y	110	490	0.73	1.9	1500	259
0.2	0.91	Y	99	440	0.94	2.4	2200	391
0.2	0.91	Y	154	680	0.88	2.2	3000	526

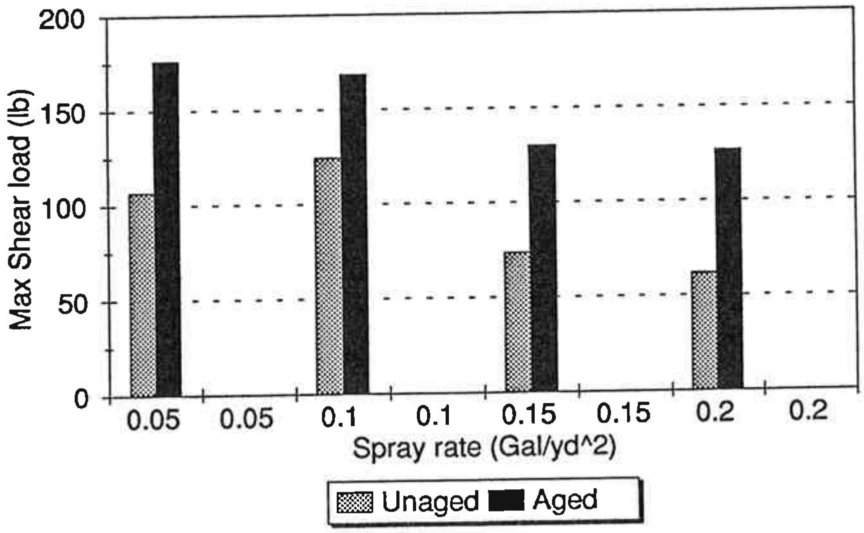


Figure 4.18. Shear load vs. spray rate

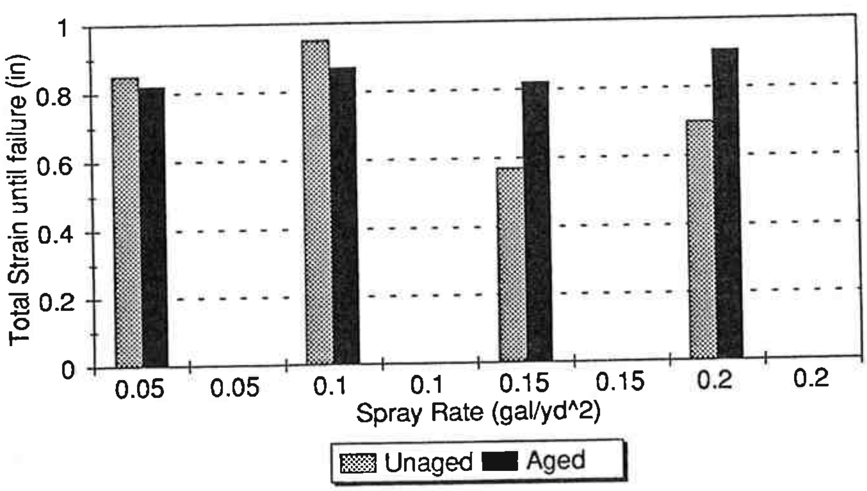


Figure 4.19. Shear strain to failure vs. spray rate

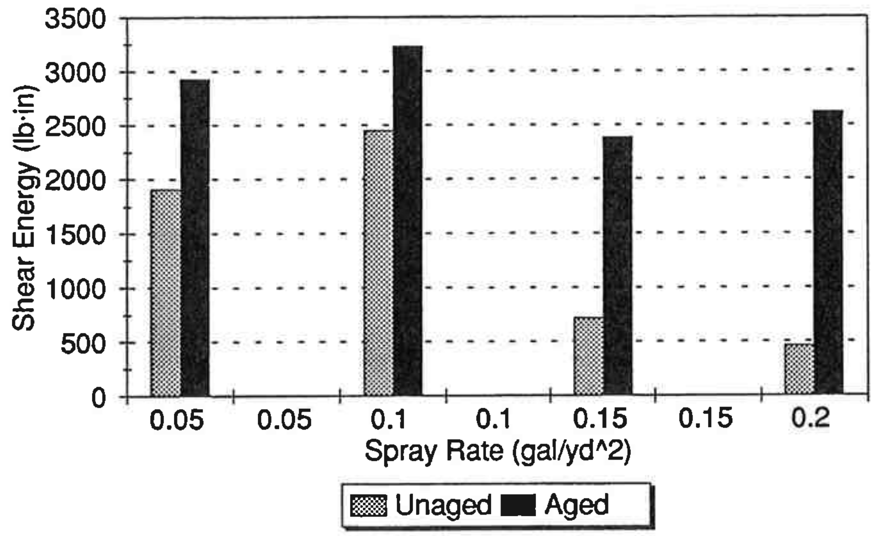


Figure 4.20. Average shear energy vs. spray rate

The results from the core testing for porous mixes showed surprisingly few changes in the gradation curves from the JMF. This would indicate that the infiltration of fines into the porous pavement is not significant. Also, the asphalt properties do not show problems with extensive aging and embrittlement.

Some useful information came out as a result of the tack coat shear testing experiment. The results show that for the normal CSS-1 tack coat emulsion, a 0.10 gal/yd^2 (0.45 l/m^2) spray rate is optimum for a PCC to F-mix bond.

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5.0 NOISE STUDY

This chapter covers the information gathered during the noise study portion of the porous pavements project. Three sections were evaluated for noise properties along Oregon's Interstate-5 freeway. Because the sound measurements had to follow a certain format that allowed comparisons of various pavement types, special sections were studied in this section of the project. Table 5.1 provides information on these sites.

5.1 Test Methods

Two types of noise measurements were taken. The first was roadside noise and the second was interior vehicle noise. Noise measurement testing for roadside noise can often be a difficult task because varying geometric configurations can cause severe changes in the acoustic characteristics from site to site. In order to remove any geometric variables, test sites were chosen where fairly new pavement types existed, and overlays of F-mix were planned in the near future. Tests were then performed before and after overlay at identical locations.

Noise measurements for the roadside study were taken in four 1-hr test periods in an attempt to filter out any anomalies in the data and to make appropriate traffic count matches. These measurements were taken to determine both an A-weighted dB(A) level, and a 1/3 octave band spectrum. Traffic counts were taken during each hour period for large trucks, medium trucks, and autos. Figure 5.1 gives an example of the normal setup for the microphone in relation to the roadway. For all sites in this study the noise measurements were taken 50 feet from the centerline of the closest directional travel lanes. Measurements were performed with a Brüle and Kjær 2221 sound level meter (Figure 5.2) to determine the A-weighted Leq, and a Rion SA-27 1/3 octave band analyzer (Figure 5.3) for the noise spectrum. All equipment was calibrated with a 1000 Hz calibrator at 93.8 dB(A) prior to measurements.

Table 5.1. Noise study site locations

Project Name	Limits	Mix Types	ADT (1992)
Halsey to Lane County Line	203.6 - 216.6	F-mix over 1993 B-mix	25500
BattleCreek to N. Jefferson	244-4 - 249.9	F-mix over 1993 B-mix	39800
Seven Oaks to Jackson	28.9 - 35.8	F-mix over 1994 B-mix over old PCC	33200

Figure 5.1. Typical setup for noise study

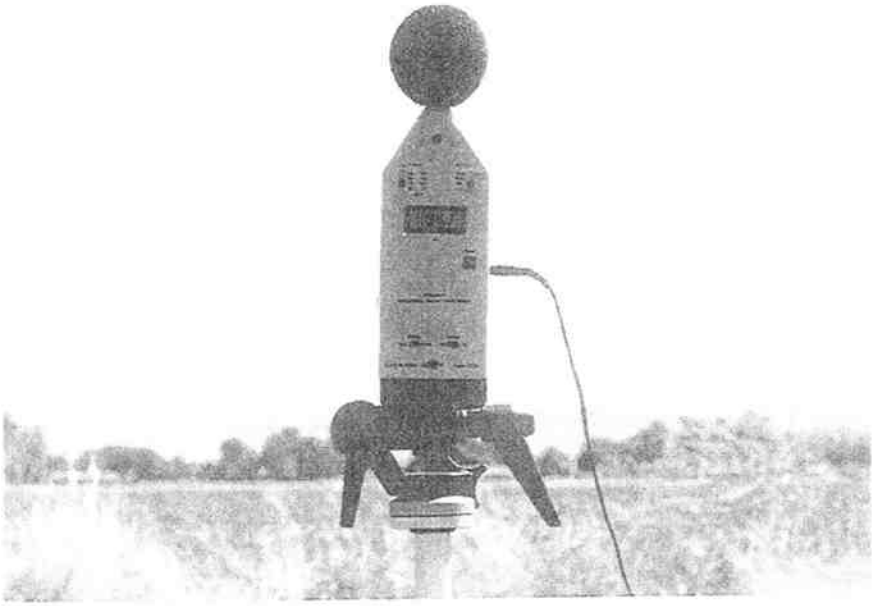

Figure 5.2. Photo of Brüle and Kjær 221 sound level meter

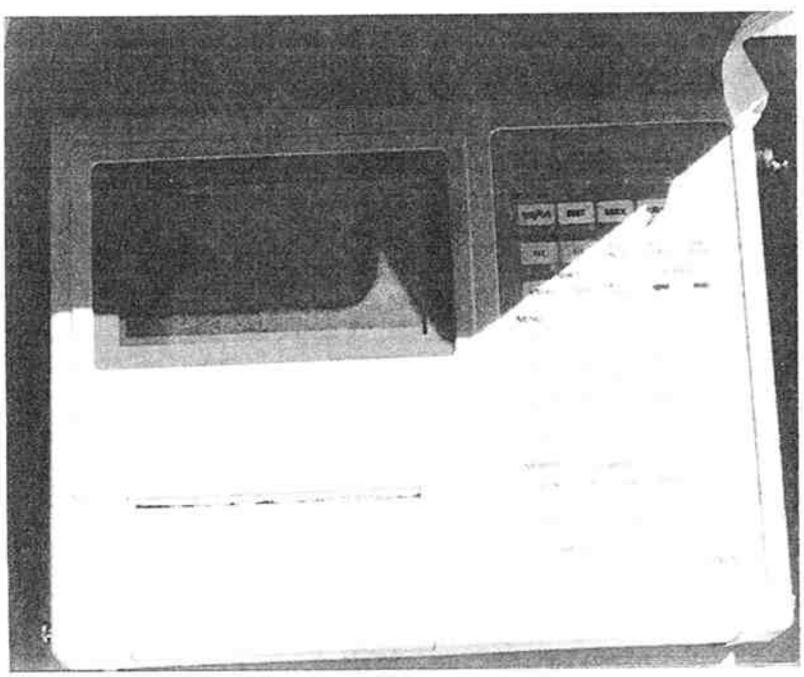

Figure 5.3. Photo of Rion SA-27 1/3 octave band analyzer

The interior noise measurements were taken inside a 1993 Dodge Caravan. The microphone was placed in the middle seat of the vehicle, at an approximate height of ear. Tests were performed at 65 mph (100 km/h). Care was taken that there were no heavy trucks travelling alongside the van during measurements. Noise levels both for an A weighted decibel level and a 1/3 octave frequency spectrum were taken for this format as well. Length of measurement was approximately 2 minutes for each site. There were approximately 3 measurements taken at each site, and these measurements were then calculated into an Leq hourly equivalent.

5.2 Results

The results from the noise study were analyzed for changes in the A-weighted sound levels for both an Leq reading and 1/3 octave band analysis. Analysis for the roadside noise was performed to try to find traffic volumes that were comparable for a single hour, or a combination of hours across the road surface types. The model used for comparison was the FHWA Traffic Noise Prediction Model, Stamina 2.0 (FHWA, 1982). The prediction model computed traffic noise levels by using the highway traffic volumes and speeds that were observed during the measurements, distance to the roadway centerline, and the physical characteristics of the area. The input variables for this program include geometric characteristics of the site and traffic volumes for near and far lanes. The output is a dB(A) Leq level based on the program's built-in prediction model. The Stamina 2.0 Traffic Noise prediction model was used to compare the theoretical noise levels of two comparative traffic characteristics. The difference in the predicted noise levels provided an estimate of the accuracy (in dB(A)) that would result from comparing noise levels with varying traffic situations. The level of accuracy criteria set for this analysis was 0.5 dB(A).

The roadside data was analyzed for traffic "matches" and these matches were then compared. The possible combinations were sought out for comparisons of old PCC, new B-mix and new F-mix (Seven Oaks to Jackson); year-old B-mix to year-old F-mix (Halsey Interchange to Lane County

Line); and a comparison between year-old B-mix and new F-mix (BattleCreek to N. Jefferson). Matches were possible for combinations of hourly traffic columns, and these sets were decibel averaged into the appropriate hourly Leq level (i.e. Leq (2-hr), Leq (3-hr), or Leq (4-hr)). Tables 5.2 to 5.4 show the traffic characteristics and Stamina 2.0 results from each traffic match used.

Tables 5.5 to 5.8 show the results of the A-weighted Leq results for the roadside analysis. In all instances the noise levels are lower for F-mix than either B-mix or the older PCC pavement. As stated in an earlier chapter, the normal range of human hearing can detect a 3 dB(A) change in sound pressure. This means that the change in sound levels for PCC to F-mix pavement is significant for the Seven Oaks to Jackson project, but this same project shows a change in noise level in a range or only about 1-2 dB(A) between the new B-mix pavement and the F-mix pavement. Though this does imply an improvement, it is not significant if the 3 dB(A) criteria is used. The results from the BattleCreek to N. Jefferson and Halsey to Lane County Line projects show a reduction from 4 to 4.5 dB(A) when the one-year-old pavements were overlaid with a new F-mix. A comparison was made with the one-year-old B-mix (northbound) and the one-year-old F-mix (southbound) at the Halsey to Lane County Line project. This comparison was deemed possible as the geometric configurations were fairly close, and the median between lanes was 77 ft (23 m). An average difference of about 1.8 dB(A) was found in this instance.

The 1/3 octave band analysis was used as a method of better understanding the effect of pavement change on the user. It is generally believed that the most sensitive range of human hearing is in the 200 to 6000 Hz range. Although this range varies between individuals, the higher frequencies are usually considered more annoying than the lower levels. The purpose of the 1/3 octave band analysis was to show that this range of frequency sound levels would show an improvement when F-mix pavement was placed as opposed to another pavement type. As the comparison for new F-mix and new B-mix pavements on the north and southbound lanes of the Seven Oaks to Jackson and the comparison of year-old F-mix and B-mix pavements at the Halsey

Table 5.2. MP 34 Seven Oaks to Jackson Street (Medford), Stamina traffic matches

	Study Side	NorthBound auto	NorthBound med	NorthBound heavy	SouthBound auto	SouthBound med	SouthBound heavy	Stamina dB(A) diff
OLD PCC hour 1	South	773	72	116	701	44	110	
New B-mix hour 3	South	996	42	93	676	44	108	0.1
OLD PCC hour 1,3	South	1720	129	250	1376	72	212	
New B-mix 2,3	South	1898	80	221	1258	81	211	0.1
OLD PCC hour 1,3,4	South	2650	179	345	1924	95	302	
New B-mix hour 1,3,4	South	2692	124	314	1976	117	302	0.0
OLD PCC 2,3,4	South	2761	173	372	1845	90	307	
New B-mix 2,3,4	South	2892	122	314	1958	83	304	0.1
OLD PCC 1,4	South	1703	122	211	1249	67	200	
New B-mix 3,4	South	1958	77	187	1376	83	201	0.1
OLD PCC 3	North	660	72	122	747	24	91	
New B-mix 1	North	496	54	120	676	29	83	0.4
New B-mix 2	South	934	45	127	582	37	103	
New F-mix 3	South	1033	48	105	790	63	103	0.4
New B-mix 2,3	South	1896	80	221	1258	81	211	
New F-mix 3,4	South	2117	93	210	1414	98	215	0.2
New B-mix 1	North	496	54	120	676	29	83	
New F-mix 2	North	617	58	122	737	44	121	0.3
New B-mix 1,3	North	1075	115	234	1412	84	184	
New F-mix 1,3	North	1208	110	235	1509	116	242	0.1
New B-mix 1,2,3,4	North	2258	209	463	2813	184	374	
New F-mix 1,2,3,4	North	2553	225	469	2989	222	465	0.3
New B-mix 1,2,4	North	1676	148	349	2077	129	273	
New F-mix 1,3,4	North	1936	167	347	2252	178	344	0.3

Table 5.3. Halsey Interchange to Lane County Line, Stamina traffic matches

	Study Side	NorthBound autos	NorthBound med	NorthBound heavy	SouthBound autos	SouthBound med	SouthBound heavy	Stamina dB(A) diff
New B-mix 2	North	903	62	140	1013	28	115	
New F-mix 1	North	1059	73	135	1055	81	145	0.3
New B-mix 4	North	540	26	134	567	13	101	
New F-mix 1	North	1059	73	135	1055	81	145	0.8
Year old B-mix 2	North	903	62	140	1013	28	115	
Year old F-mix 1	South	817	89	205	795	46	143	0.1
Year old B-mix 3	North	737	45	165	851	18	98	
Year old F-mix 2	South	841	87	173	757	42	164	0.1
Year old B-mix 1	North	919	81	151	911	48	133	
Year old F-mix 4	South	880	71	178	806	41	148	0.3

Table 5.4. MP 248 BattleCreek to N. Jefferson, Stamina traffic matches

	Study Side	NorthBound autos	NorthBound med	NorthBound heavy	SouthBound autos	SouthBound med	SouthBound heavy	Stamina dB(A) diff
New B-mix 1	South	1195	65	166	1058	76	132	
New F-mix 1	South	1150	76	208	1090	99	143	0.6
New B-mix 2	South	1506	86	189	1086	74	188	
New F-mix 2	South	1198	95	248	990	130	197	0.5
New B-mix 2,4	South	2962	178	388	2301	134	366	
New F-mix 1,3	South	2341	168	459	2202	243	259	0.3

Table 5.5. Seven Oaks to Jackson (South Bound) – exterior noise data

PCC to B-Mix Leq Levels				
Matches	Leq Time (hrs)	PCC Leq dB(A)	B-Mix Leq dB(A)	Difference dB(A)
1 to 3	1	76.4	69.8	6.6
1,3 to 2,3	2	76.3	70.0	6.3
1,4 to 3,4	2	75.7	70.0	5.7
1,3,4 to 1,3,4	3	75.9	69.7	6.2
2,3,4 to 2,3,4	3	75.9	70.0	5.9

PCC to F-Mix Leq Levels				
Matches	Leq Time (hrs)	PCC Leq dB(A)	F-Mix Leq dB(A)	Difference dB(A)
2,3 to 3,4	2	76.4	68.9	7.5
1 to 4	1	76.4	68.6	7.8

B-Mix to F-Mix Leq Levels				
Matches	Leq Time (hrs)	B-Mix Leq dB(A)	F-Mix Leq dB(A)	Difference dB(A)
2 to 3	1	70.1	69.2	0.9
2,3 to 3,4	2	70.0	68.9	1.1

Table 5.6. Seven Oaks to Jackson (North) – exterior noise data

PCC to B-Mix Leq Levels				
Matches	Leq Time (hrs)	PCC Leq dB(A)	B-Mix Leq dB(A)	Difference dB(A)
3 to 1	1	76.5	70.7	5.9

PCC to F-Mix Leq Levels				
Matches	Leq Time (hrs)	PCC Leq dB(A)	F-Mix Leq dB(A)	Difference dB(A)
1 to 4	1	76.8	69.2	7.5
1,3 to 3,4	2	76.7	69.2	7.4

B-Mix to F-Mix Leq Levels				
Matches	Leq Time (hrs)	B-Mix Leq dB(A)	F-Mix Leq dB(A)	Difference dB(A)
1 to 2	1	70.7	68.9	1.8
1,3 to 1,3	2	70.5	69.1	1.4
1,2,3,4 to 1,2,3,4	4	70.4	69.1	1.4
1,2,4 to 1,3,4	3	70.5	69.2	1.3

Table 5.7. Halsey to Lane County Line – exterior noise data

B-Mix to F-Mix Leq Levels				
Matches	Leq Time (hrs)	B-Mix Leq dB(A)	F-Mix Leq dB(A)	Difference dB(A)
2 to 1	1	73.3	68.8	4.4
4 to 1	1	72.1	68.8	3.3
2,4 to 1,3	2	76.8	72.6	4.2
1-year-old B-mix to 1-year-old F-mix Leq Levels				
Matches	Leq Time (hrs)	B-mix Leq dB(A)	F-mix Leq dB(A)	Difference dB(A)
2 to 1	1	73.3	71.3	2.0
3 to 2	1	73.4	71.9	1.5
1 to 4		73.3	71.4	1.9

Table 5.8. BattleCreek to North Jefferson – exterior noise data

B-Mix to F-Mix Leq Levels				
Matches	Leq Time (hrs)	B-Mix Leq dB(A)	F-Mix Leq dB(A)	Difference dB(A)
2,4 to 1,3	2	76.8	72.6	4.2
1 to 1	1	76.2	72.2	4.0
2 to 2	1	76.9	72.7	4.2

to Lane County Line projects were deemed the most useful projects by the direct pavement to pavement comparison, the frequency study is most useful when these projects are considered.

Figures 5.4 through 5.6 display a frequency spectrum dB(A) difference as a comparison across pavement types. A positive value shows at which frequencies the F-mix pavement is quieter and a negative value represents a lower dB(A) level for the B-mix pavement. Figures 5.4 and 5.5, which are from the Seven Oaks to Jackson project, show a 4 dB(A) to 1 dB(A) improvement for F-mix pavement in the 500 to 6000 Hz range, with all other ranges showing an improvement for B-mix pavement from 9 dB(A) to 1 dB(A). Figure 5.6, which is from the Halsey Interchange to Lane County Line project, displays a curve that is much different from the new B-mix to new F-mix comparison. This curve shows F-mix improvements from 1 dB(A) to 4.5 dB(A) in the range of 500 to 4000 hz, and an improvement from 0 dB(A) to 1.5 dB(A) in the 25 to 200 Hz range. All other ranges show an improvement for B-mix in the range of 0 dB(A) to 2.5 dB(A).

Data for the interior noise levels were directly averaged for various 2-minute sample times using the decibel addition method. These numbers were compared for pavement types. Table 5.9 displays the Leq data computed for the noise measurements taken. These data seem more sporadic than the data for the roadside noise measurements, and even show a near significant to significant (2 - 2.9) dB(A) change in the favor of the B-mix pavement at the Seven Oaks to Jackson project. Data for the BattleCreek to N. Jefferson and the Halsey to Lane County Line show little to no change (0 to 1.5) dB(A) in the Leq levels.

Frequency sweeps for the interior noise levels, like those for the roadside were displayed for the same location sites. Figures 5.7 to 5.9 display how sporadic the interior measurements came out. The only site that displayed a curve that even remotely compared to those for the roadside measurements was the northbound lane for the Seven Oaks to Jackson project, where there is a 1 to

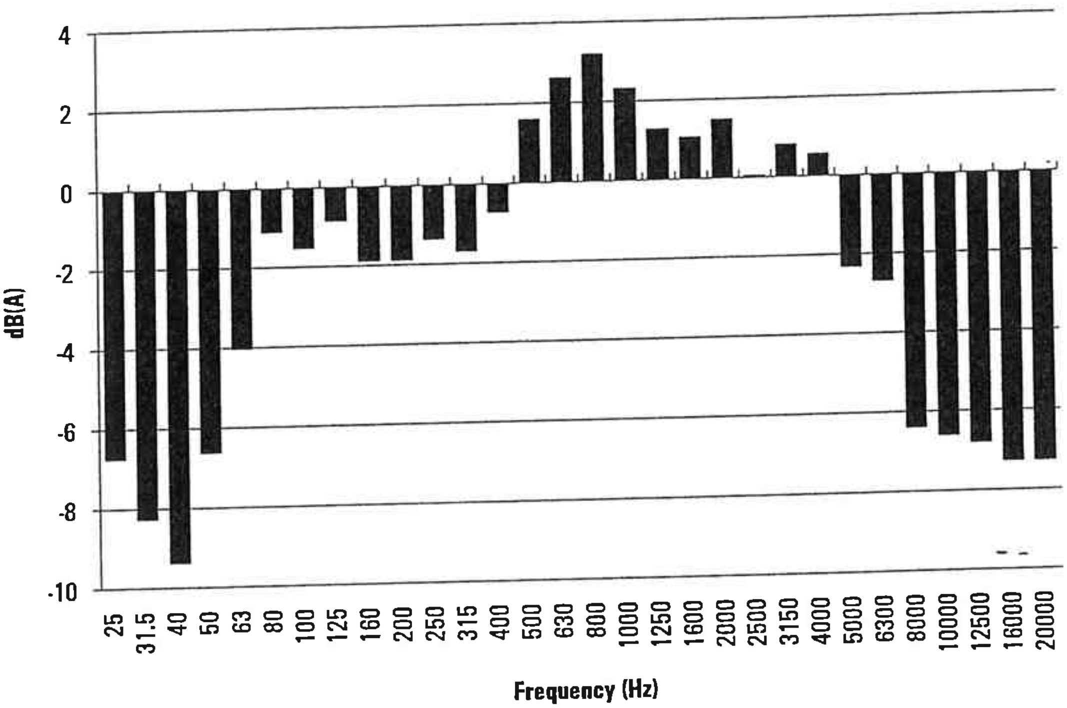

Figure 5.4. Seven Oaks to Jackson (South) new B-mix to new F-mix

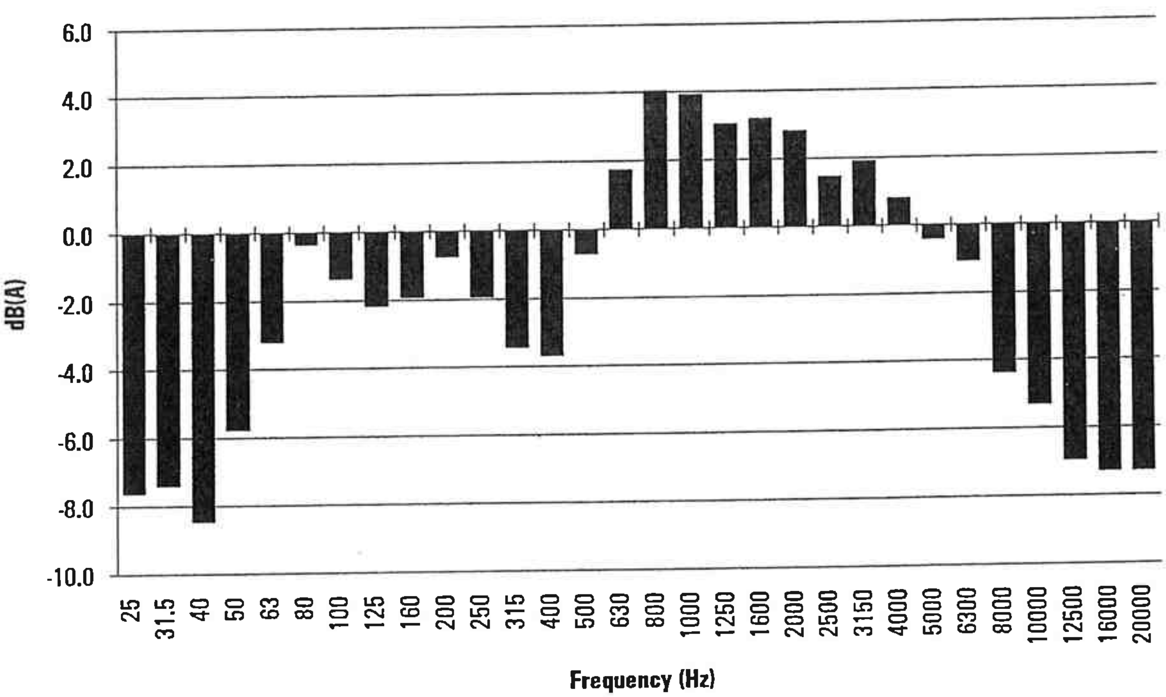

Figure 5.5. Seven Oaks to Jackson (North) new B-mix to new F-mix

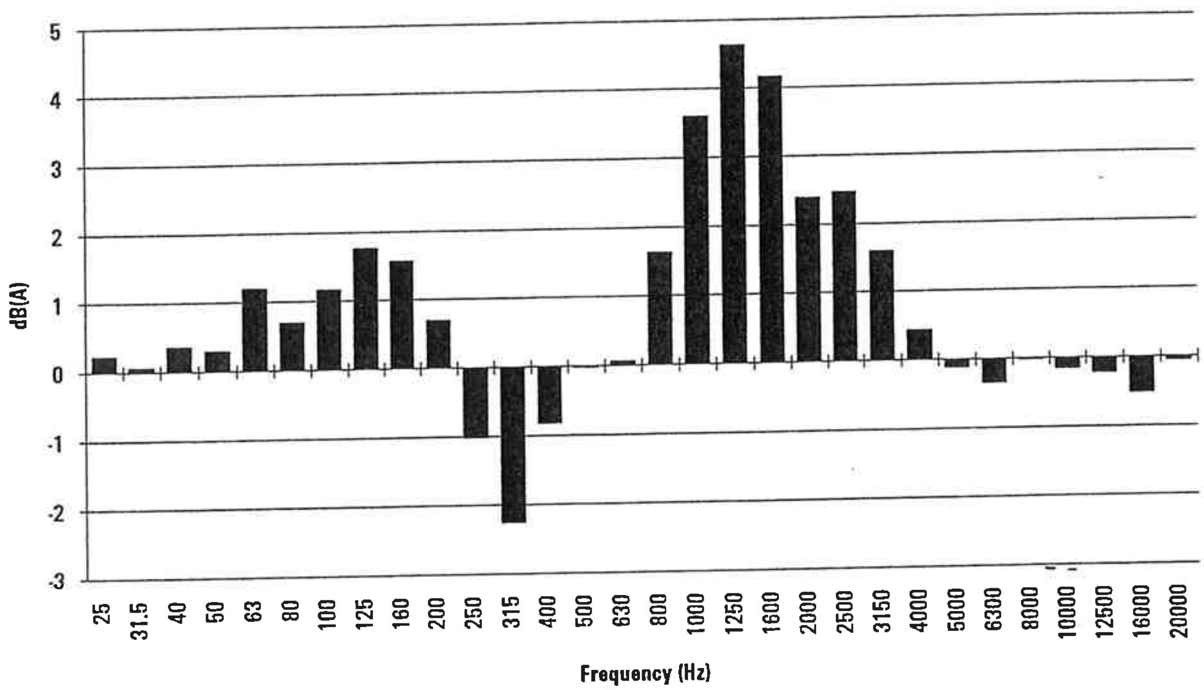

Figure 5.6. Halsey Interchange to Lane County Line 1-year-old B-mix to 1-year-old F-mix

Table 5.9. Interior A weighted sound levels

Seven Oaks to Jackson Interior			
Matches	Leq dB(A)	Leq dB(A)	Difference dB(A)
PCC - B-mix South	75.1	68.5	6.6
PCC - B-mix North	73.5	70.5	3.0
PCC - F-mix South	75.1	71.4	3.7
PCC - F-mix North	73.5	72.5	1.0
B-mix - F-mix South	68.5	71.4	-2.9
B-mix - F-mix North	70.5	72.5	-2.0
BattleCreek to N. Jefferson Interior			
Matches	Leq dB(A)	Leq dB(A)	Difference dB(A)
B-mix - F-mix	72.7	72.0	0.7
Halsey to Lane County Line Interior			
Matches	Leq dB(A)	Leq dB(A)	Difference dB(A)
B-mix - F-mix	72.0	72.0	0.0

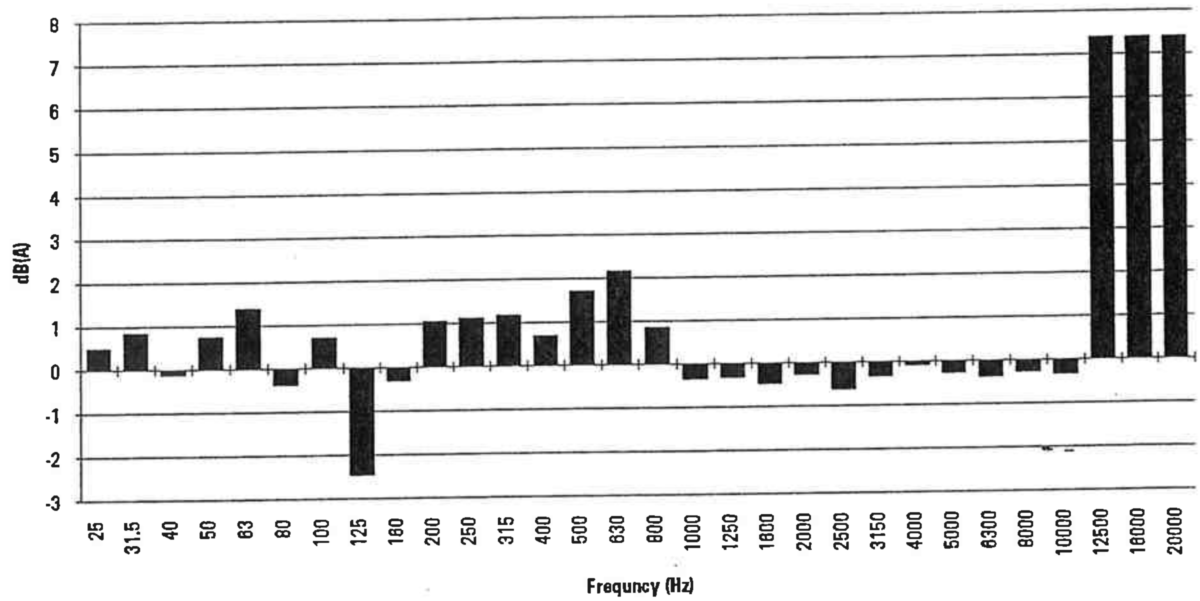

Figure 5.7. Seven Oaks to Jackson (South Interior) new B-mix to new F-mix

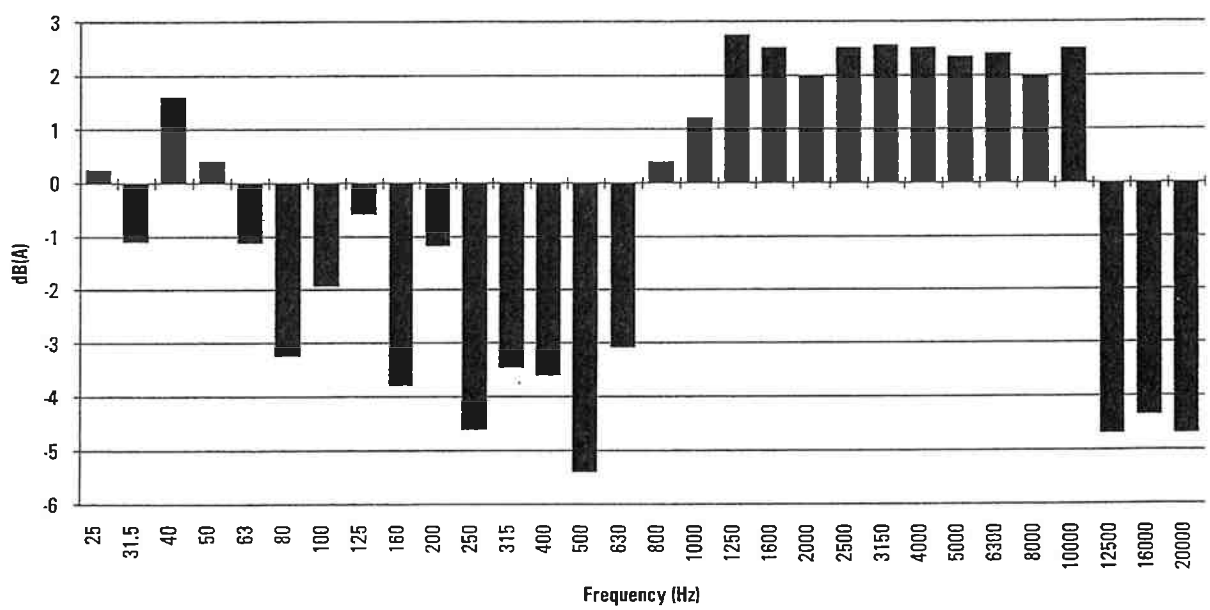

Figure 5.8. Seven Oaks to Jackson (North Interior) new B-mix to new F-mix

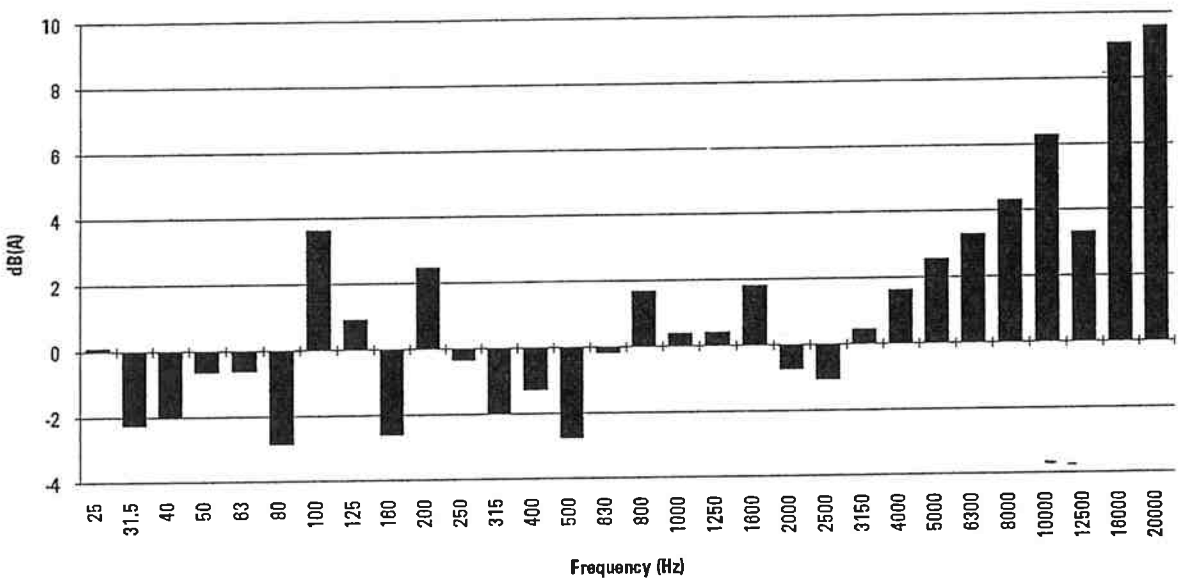

Figure 5.9. Halsey to Lane County Line (Interior) 1-year-old B-mix to 1-year-old F-mix

2.5 dB(A) improvement for the F-mix pavement in the 800 to 10,000 Hz range, and a 0 to 5.5 dB(A) improvement for B-mix at all other frequencies.

5.3 Summary of Findings

The results of the noise analysis confirm the data found in the literature search which indicate porous pavements reduce the noise in the higher frequency zones. This conclusion is supported mostly from the roadside measurements, and not from those taken in the interior of the vehicle. A possible explanation for this is that the higher frequencies are dampened by the vehicle shell. As high frequency noises have a shorter wavelength, they would be more apt to be reflected off the vehicle's thin shell, and would hide some of the data and make F-mix pavements appear a little more noisy inside than outside.

There were three sites studied to determine significant noise changes in pavements of equal age. These were the Seven Oaks to Jackson north and south sites with new pavement, and the Halsey Interchange to Lane County Line with one year old pavement. The data analysis was performed with B-mix to F-mix comparison for these sites. Exterior noise improvement for F-mix over B-mix was 0.9 to 2.0 Leq dB(A), and a 1-4.5 dB(A) change for 1/3 octave band frequency in the 600 to 4000 Hz range. Interior noise showed an improvement for B-mix over F-mix in the range of 2-3 dB(A) range, and little change for frequency levels.

6.0 EVALUATION OF PROJECT DATA

The evaluation of the data for this report encompasses all that was discovered during the study. The significant findings include changes in properties over time, specification change suggestions, and new porous pavement guidelines.

6.1 Significant Findings

6.1.1 Field Studies

The skid data collected as a part of this study provided interesting results. It was never expected that the "wet" skid numbers would be higher (more skid resistant) than those for the "dry" condition. Even though the dry condition is not a true dry condition, there is still quite a lot more rain on the road during a rainstorm. These data are unexplainable, as data were taken at various times in the year, and the data came out to show the anomaly. Again, the hypothesis for this problem is that the water from the sprayer on the friction tester actually loosens any detritus on the road surface, but does not provide enough water or time to wash it away completely.

Data were also collected from the project for pavement permeability, texture depth, and rutting. The data are inconclusive as to whether or not the permeability is truly decreasing over time, as the length of the study was too short and the equipment used wasn't sufficient for F-mix pavements. Texture depth of the pavement is somewhat correlated to the pavement permeability. As for rut depth measurements for the sites, there is no truly noticeable change over the course of a year in the rutting potential of the pavement mat. This is an expected attribute of a good F-mix overlay.

The accident data that were collected as a part of the field study on F-mix pavements were disappointingly inconclusive. The amount of data available, and the relatively short time period available made it difficult to come up with any conclusive evidence that accident rates are lowered after placement of open-graded pavements.

Splash and spray data collected in this study indicate the F-mix to have a spray improvement of at least 10% over B-mix and PCC pavements. Some F-mixes were tested with inadequate cross slopes. These tests proved that poorly graded F-mixes do not perform well during heavy rain storms as the water sits in the low spots instead of draining.

6.1.2 Laboratory Studies

The laboratory data provided interesting insights into the behavior of porous pavements. The ECS results for the two tested projects suggest some differing performance characteristics. The Pendleton project may start to show some water sensitivity problems in the near future based on the results of the ECS test. Whether or not the ECS test is a valid test for F-mixes remains to be seen, and watching this project site should be useful in this determination.

From the results of the field core tests performed at the ODOT labs, it would appear as if the pavements surveyed in this project are holding up fairly well. It would also appear as if there is only a small amount of fines getting down into the pores of the pavement, and that there is little clogging of the pavement.

The shear test data provided some good numbers and information about placing a tack coat on a PCC surface before covering it with an open-graded friction course. Test data suggests that if the normal CSS-1 tack coat emulsion is used, that the 0.10 gal/yd^2 (0.45 l/m^2) spray rate would provide the best tack. These same results would be expected to be seen over time based on the results of artificially aging the sample.

6.1.3 Noise Study

The noise study portion of the project provided useful information on the differences in noise levels for conventional pavements and porous mixtures. The data clearly showed that new porous mixtures used in Oregon provide a 1-2 dB(A) reduction in roadside noise over B-mix. The main area of reduction is in the upper range frequencies, where the human ear is most sensitive for some individuals.

6.1.4 Stress Distributions in Porous Asphalt Mixes

Appendix F is a report completed as a part of this study. The study employed finite element analysis methods to determine the appropriate confining pressures in F-mixes, and the effect on critical stresses of an F-mix over a PCC pavement. The analysis showed 100-300 psi (690-2100 kPa) confining pressure can occur in F-mixes. For the F-mix over PCC pavement, the F-mix does not significantly reduce stress in the PCC.

6.2 Changes in Mix Properties Over Time

An important aspect of any pavement surfacing is how it may change with time. ODOT realized that it is lacking some information in this area for porous pavements, and that is part of the reason for this project.

Many of the findings contained in this report should be monitored to get a better idea as to what is happening with the use of porous pavements with traffic and time. The properties of interest include field permeability, rutting, noise, and splash and spray. Information gathered would help ODOT quantify the benefits of porous pavements used in Oregon.

6.3 Suggested Modifications to Specifications

One of the reasons for this study was to try to come up with some valid changes in the asphalt specifications for porous pavements found in the *Oregon Standard Specifications for Highway Construction* (ODOT, 1991). Based on the literature review and input from paving personnel, as outlined in the interim report (Younger et al., 1994), listed below are some suggested changes to the specifications. A copy of the current ODOT specifications for asphalt concrete is presented in Appendix B.

Section - 00745.42 Preparation of Underlying Surfaces

In order to make sure water does not infiltrate the base course layers, the following is to be added under part (b) <u>All Projects</u>

> For open-graded pavements (type E and F mix), make sure underlying layer is properly sealed with an appropriate dense material that fills in all depressed surface areas.

Section - 00745.48 Hauling, Depositing, and Placing

Because mixture transport distances from batch plant to project site are critical to reduce draindown and chunking of porous pavements, the following is to be added under part (a) <u>Hauling</u>:

> For open-graded pavements haul times greater than 30 minutes, mixes should be tested for draindown according to F-mix design procedures, and adjusted for asphalt content and mix temperature if draindown is greater than 60%.

and under part (b) <u>Depositing</u> section on windrows:

- Reduce any chunking in open-graded mixtures.

6.4 Suggested Guidelines for Use of Porous Mixtures

A number of interesting facts about porous pavements were discovered during the course of this project. The majority of this information was gathered through the literature search, and is thus gleaned from the experience of both ODOT and other agencies. Table 6.1 lists some limitations for porous pavements and the reason why.

Porous pavements are recommended for use in such areas as high volume trafficked areas with high rainfall levels, or in areas where noise reduction is required. The safety benefits of porous pavements make them an attractive paving alternative.

The problems noted in the frictional characteristics of porous pavements directly after placement require additional study.

Another point about porous pavements is that an environmental use zone for Oregon is needed. Due to the many difficulties in using porous pavements in mountainous regions porous pavements should be restricted from use in these areas. Figure 6.1 outlines the uses of porous pavements in Oregon. This figure provides a flow chart of where porous pavements are recommended and where not recommended.

Table 6.1. Limitations of porous pavements

Usage	Reasoning
City streets	Requires a lot of extra time and money to assure drainage occurs properly.
Heavy winter snow areas	Snow plows can damage the pavement surface and the pores can get clogged by sanding debris.
Paving that requires a lot of handwork	Porous pavements are not easy to handle by raking into position, and cost extra for such work.

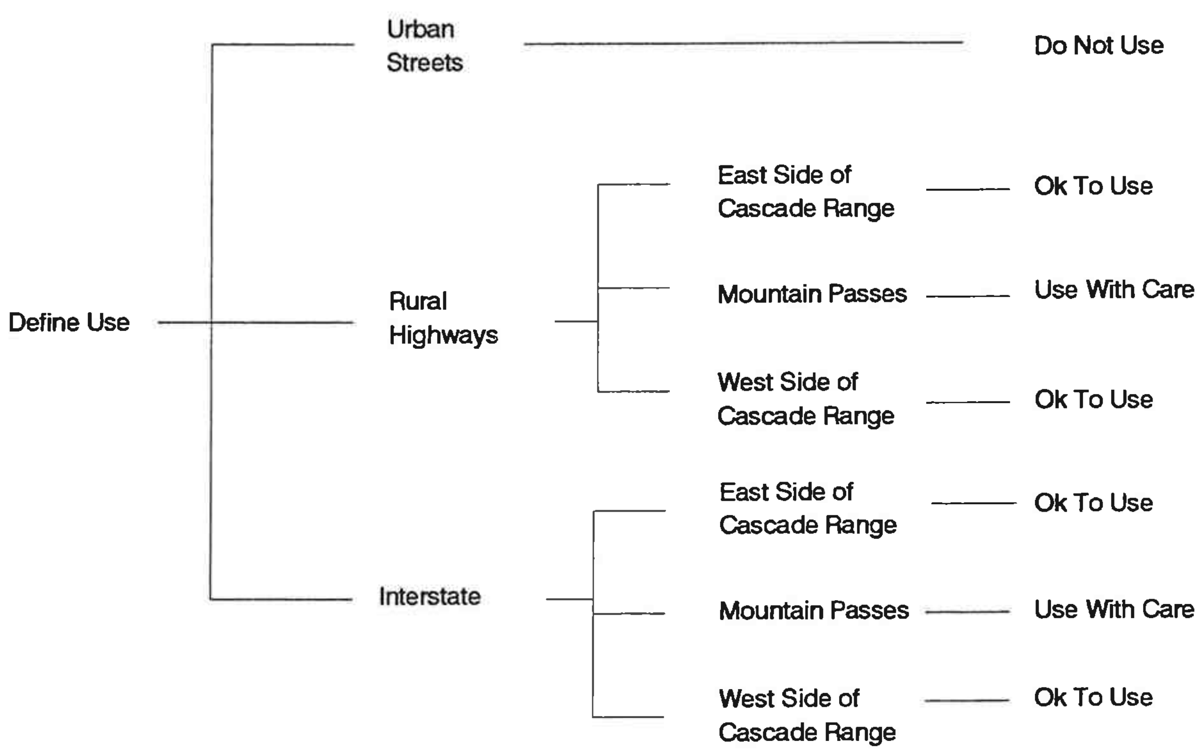

Figure 6.1. Flow chart for porous pavement uses

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

This report is the culmination of two years of data gathering from porous pavements used throughout Oregon. The areas of skid, noise, water sensitivity, safety, properties over time, and tack coats were investigated and reported herein. The culmination of this study provides a fair amount of data dealing with porous pavements. Specific conclusions resulting from this study are given below:

1) Advantages documented in the literature review are: increased wet weather friction resistance, reduced splash and spray, noise reduction, decreased hydroplaning, reduced rutting, and glare reduction.

2) Disadvantages documented in the literature review are: some construction difficulties, winter performance problems, potential oxidation problems, and patching problems.

3) Porous pavements provide good rutting resistance.

4) After several years, F-mix pavements in Oregon do not seem to have a problem with filling of voids.

5) ODOT's F-mix pavement provides a 10% or greater reduction in spray over B-mix or PCC pavements.

6) F-mixes are 1-2 dB(A) quieter than B-mixes for roadside noise and 2-3 dB(A) louder for interior.

7) 1/3 octave band analysis shows that F-mixes are 0-4.5 dB(A) quieter in the 500-4000 Hz range for exterior, yet no change for interior noise.

8) ECS testing shows a possible water sensitivity problem for the Pendleton - Emigrant Hill project, and none for the Jumpoff Joe project.

9) ODOT's F-mix shows little change over time for rutting, permeability, and void levels.

10) Porous pavements should not be used in city streets or in areas that will require a lot of construction handwork

7.2 Recommendations for Implementation

The data presented in this report, along with the specification changes and guideline recommendations provide a good start for ODOT to build on the data base regarding the behavioral properties of porous pavements. Continuing this data collection process will provide ODOT will a data base of information to quantify the benefits of porous pavements. Specific recommendations for implementation include:

1) Use a 0.10 ga./yd^2 (0.45 l/m^2) spray rate for a CSS-1 emulsion tack coat between F-mix overlays on PCC pavements.

2) Change F-mix specifications as suggested in Section 6.3.

3) Adopt the recommended guidelines for use of porous pavements as outlined in Section 6.4.

7.3 Recommendations for Future Study

Though porous mixes seem to be performing well, future studies should look at the following:

1) Continue testing of splash and spray, and other properties of porous pavements over an extended period of time.

2) Monitor the F-mixes tested for water sensitivity in the ECS over an extended period of time. This should determine the applicability of the ECS to identify mixes susceptible to early moisture problems.

3) Review construction procedures of others to improve upon construction methods used in Oregon, including the development of pay incentive/disencentives for porous asphalt mixes.

4) Continue to monitor E. Pendleton-Emigrant Hill project for water sensitivity problems.

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APPENDIX A

List of Oregon F-mixes

Table A.1 F-mix highway log

CONTRACT #	JOB NAME	PLACED	NAME	NUMBER	COUNTY	MILES	M.P.	M.P.	COND	ADT
C 11037	DIST 5 OVERLAY PROJECT	1991	VARIOUS		LANE	2.10	5.23	3.13		
C 10751	DISTRICT 7 OVERLAY PROJECT	1989	VARIOUS		COOS	2.95	274.60	277.55		
C 10763	DIST. 3 PAVING PROJECT	1989	VARIOUS		MARION	6.67	23.36	30.03		
C 11037	DIST 5 OVERLAY PROJECT	1991	VARIOUS		LANE	5.28	20.31	22.59		
C 115-1557	ANTIOCH RD. CRATER LK. HWY.	1985	SAMS VALLEY	271	JACKSON	4.87	12.61	17.48	2	2100
C 10256	DISTRICT 5 PAVING PROJECTS	1986	VARIOUS		LANE	11.88				
C 09812	DAYS CREEK - TRUCK SCALES	1984	TILLER-TRAIL	230	DOUGLAS	2.00	10.80	12.80	2	500
C 09993	S. FRK. COQUILLE RV.- R.R. AVE.	1985	POWERS	242	JACKSON	4.89	12.61	17.48	4	640
C 09783	WILD PARK LANE - REEVES CR.	1984	REDWOOD	25	JOSEPHINE	2.57	22.00	24.57	3	6200
C 10006	DIST. 8 PAVING PROJECT	1985	REDWOOD	25	JOSEPHINE	5.20	6.90	12.10	4	7000
C 10761	DIST. 5 OVERLAY PROJECT	1989	VARIOUS		LANE	12.64				
C 09972	CLOVER LANE - NEIL CREEK RD.	1985	GREEN SPRINGS	21	JACKSON	2.90	1.50	4.40	3	3400
C 11065	JUMP OFF JOE-N.GRANTS PASS	1991	PACIFIC	001	JOSEPHINE	8.91	67.11	58.20	2	23900
C 10941	HAYSVILLE-BATTLE CR.INLAY SAL	1990	PACIFIC	001	MARION	9.59	259.09	249.50	2	39800
C 11038	SANTIIAM RV.(S.B.) BRIDGE	1992	PACIFIC	001	MARION,LINN	1.24	241.44	240.20	4	42500
C 11334	SANTIAM RV,N BOUND BR. SEC	1993	PACIFIC	001	MARION,LINN	1.38	240.60	241.07	4	42500
C 10980	N. JEFFERSN INTCH-N ALBNY INTC	1991	PACIFIC	001	LINN,MARION	9.80	234.23	244.49	2	42500
C 11294	HALSEY INTR. LANE CNTY LINE SE	1993	PACIFIC	001	LINN	12.59	216.14	203.55	9	25500
C 10989	WINCHESTER INT. N.B. RAMPS	1991	PACIFIC	001	DOUGLAS	0.62	129.43	129.21	2	21800
C 10963	SUTHERLIN INT.-GARDEN VLY BLV	1991	PACIFIC	001	DOUGLAS	11.76	124.80	136.27	2	29000
C 10952	W. MARQUAM INT.-N.TIGARD INT.	1990	PACIFIC	001	MULTNOMAH	5.29	294.00	299.50	2	87000
C 10749	SUVER- THOUSAND OAK DR SEC	1989	PACIFIC HWY WEST	01W	BENTN,POLK	7.01	70.50	77.51	3	5200
C 11300	PERRYDALE RD-CROWLEY RD	1993	PACIFIC WEST	01W	POLK,YAMHL	7.9	46.74	54.40	9	3150
C 11138	BELTLINE HWY-BARGER AV(EUGE	1992	PACIFIC HWY WEST	01W	LANE	1.28	118.24	119.52	1	11800
C 10961	BROOKMAN RD-GARLAND RD N.	1991	PACIFIC HWY W.	01W	WASHINGTO	2.0	17.42	19.42	2	20800
C 10939	CORBETT INTCHG.-MULTNOMH FI	1991	COLUMBIA RIVER	002	MULNOMAH	93.01	22.34	31.00	2	14700
C 11087	NE. 181ST AV-TROUTDALE OVERL	1991	COLUMBIA RIVER	002	MULTNOMAH	3.01	13.83	16.84	2	33900
C 11256	RUFUS-ARLINGTON (W.UNIT)	1993	COLUMBIA RIVER	002	GILM,SHRMN	15.54	128.76	129.30	1	7300
C 10949	RUFUS-ARLINGTON (E. UNIT)	1991	COLUMBIA RIVER	002	GILLIAM	12.7	125.5	138.2	1	7300
C 11245	UMATILLA - MCNARY	1993	COLUMBIA RIVER	002	UMATILLA	3.10	182.6	185.7	9	5000
C 10926	RAINIER- TIDE CREEK	1990	COLUMBIA RV. HWY(LOWER	02W	COLUMBIA	11.58	36.50	46.55	2	7300
C 11276	GREEN SPRNGS HWY-MDLND HW	1993	THE DALLES-CALIFORNIA	004	KLAMATH	2.71	277.79	280.50	9	6000
C 11351	KLAMATH FLS,MALIN,GREEN SPRI	1993	THE DALLES/CALIFORNIA	004	KLAMATH	5.47	272.35	277.37	4	6000
C 11351	- - - -	1993	- - - -	004	KLAMATH	11.20	280.50	291.70	9	3300
C 10743	CHEMULT- LENZ RD. SECT.	1990	THE DALLES- CALIFORNIA	004	KLAMATH	17.08	203.85	220.93	2	4350
C 11331	FREMONT JCT. - HACKETT DRIVE	1993	THE DALLES- CALIFORNIA	004	DESCHUTES	7.30	169.90	177.20	9	39000
C 10766	HACKETT DR.- GILCREST	1989	THE DALLES- CALIFORNIA	004	KLAMATH	6.20	177.00	183.20	2	39000
C 10462	MURPHY RD.- LAVA BUTTE	1989	THE DALLES CALIFORNIA	004	DESCHUTES	5.44	141.50	150.80	3	13800
C 11210	NORWOOD RD-PWRS RD(BEND)	1992	THE DALLES-CAL.,POWEL B	004	DESCHUTES	5.24	135.43	140.67	2	26000
C 10672	REDMOND BEND(SOUTH UNIT)	1989	THE DALLES- CALIFORNIA	004	DESCHUTES	2.09	132.66	134.75	2	12300
C 11104	REDMOND-BEND(N. UNIT)	1992	THE DALLES-CALIFORNIA	004	DESCHUTES	9.23	123.18	132.41	1	12300
C 11104	REDMOND-BEND (N.UNIT)	1992	THE DALLES-CALIFORNIA	004	DESCHUTES	9.23	123.18	132.41	1	12300
C 10850	O'NEIL JCT.-REDMOND COUPLET	1990	THE DALLES- CALIFORNIA	004	DESCHUTES	2.09	120.26	118.43	1	7500
C 11009	TERREBONNE-O'NEIL JCT	1991	THE DALLES CALIFORNIA	004	DESCHUTES	3.30	115.2	118.4	1	7500
C 11210	NORWOOD RD-PWRS RD(BEND)	1992	THE DALLES-CAL.,POWEL B	004	DESCHUTES	7.57	0.00	7.57	9	2900

Table A.1 F-mix highway log (continued)

CONTRACT #	JOB NAME	PLACED	NAME	NUMBER	COUNTY	MILES	M.P.	M.P.	COND	ADT
C 09652	LENZ RD. - FORGE RD.	1984	THE DALLES/ CALIFORNIA	004	KLAMATH	18.30	222.90	241.20	2	3800
C 11015	WILLIAMSON RV.-MODOC PNT.	1991	THE DALLES-CALIFORNIA	004	KLAMATH	2.26	253.80	256.20	2	5300
C 10874	FORGE RD-LOBERT(S. UNIT)	1990	THE DALLES-CALIFORNIA	004	KLAMATH	2.70	247.70	251.60	2	3800
C 10972	FORGE RD-LOBERT(N. UNIT)	1991	THE DALLES-CALIFORNIA	004	KLAMATH	8.40	241.22	251.64	1	3800
C 10924	FAREWELL BEND-OLDS FERRY IN	1991	OLD OREGON TRAIL	006	MALHEUR	2.78	355.77	352.99	3	5700
C 11170	DURKEE INTERCHANGE	1993	OLD OREGON TRAIL	006	BAKER	14.97	327.15	342.12	9	5100
C 10930	BALDOCK SLOUGH-S BAKER INTC	1991	OLD OREGON TRAIL	006	BAKER	9.62	297.10	306.72	1	5510
C 11119	E. PENDLETON INTCH.-EMGRANT	1992	OLD OREGON TRAIL	006	UMATILLA	4.69	213.04	217.73	2	4500
C 09645	S. BAKER - DURKEE	1984	OLD OREGON TRAIL	006	BAKER	21.00	306.40	327.40	3	5100
C 10974	FARWELL BEND- OLD FERRY INT.	1990	OLD OREGON TRAIL	006	MALHEUR	2.78	253.30	356.08	3	5200
C 10425	POWLL BUTT JCT-ARNLD ICE CV		CENTRAL OREGON HWY	007	DESCHUTES	814	4.30	12.44	2	1950
C 11048	OCI ACCES RD-STANTON BLVD IN	1991	STANTON BLVD(COUNTY RD	007	MALHEUR	2.23				
C 11296	BROOTN RD-LITTLE NESTUCA RV	1993	OREGON COAST	009	TILLAMOOK	1.65	90.33	91.98	9	4000
C 11253	PLEASANT VLY-GREEN TIMBER R	1993	OREGON COAST	009	TILLAMOOK	1.03	75.08	76.11	9	4500
C 10681	SIMMONS CR.- PLEASANT VLY RD	1989	OREGON COAST	009	TILLAMOOK	1.42	71.57	72.99	1	4500
C 11305	NEDONNA BEACH RD-BARVIEW	1993	OREGON COAST	009	TILLAMOOK	5.40	48.60	54.00	9	6200
C 11205	ARCH CAPE TUNNL-SHORT SND C	1992	OREGON COAST	009	CLAT.,TILLM	3.19	35.91	39.10	1	3100
C 10599	CAPE SABASTION- MYERS CR RD	1988	OREGON COAST	009	CURRY	1.75	334.75	336.50	2	4000
C 11298	DIST 7 OVERLAY PROJECT	1993	VARIOUS	009	COOS	0.28	280.82	280.10	2	4300
C 11298	DIST 7 OVERLAY PROJECT	1993	VARIOUS	009	COOS	2.60	224.40	227.00	2	8600
C 10673	LONGWOOD DR.- WINCHESTER W	1989	OREGON COAST	009	DOUGLAS	1.41	213.60	215.01	2	9600
C 11298	DIST 7 OVERLAY PROJECT	1993	VARIOUS	009	COOS	0.55	1.70	2.25	2	3500
C 11207	PASSMORE RD-BAYSHORE DR	1992	OREGON COAST	009	LINCOLN	7.37	147.38	154.75	1	10800
C 11333	DEPOE BAY RD-NE. 54TH ST	1993	OREGON	009	LINCOLN	9.92	127.60	137.53	9	8400
C 09781	GOLD BEACH - SEBASTION PK. RD	1984	OREGON COAST	009	CURRY	2.00	328.44	330.48	2	11300
C 11034	DIST. 7 PAVING PROJECT	1991	OREGON COAST	009	COOS,CURR	6.49	221.30	255.03	2	11400
C 10870	DOOLEY BR.- CANNON BEACH	1990	OREGON COAST	009	CLATSOP	1.58	22.50	24.50	3	7700
C 09987	EUCHRE CR.- OPHIR REST AREA	1985	OREGON COAST	009	CURRY	2.50	316.98	319.38	2	3400
C 10446	SUTTON LAKE - FLORENCE	1988	OREGON COAST	009	LANE	5.50	184.50	190.30	1	5800
C 10948	IMBLER-ELGIN (PASS LANE)	1992	WALLOWA LAKE	010	UNION	2.21	15.58	17.79	2	3200
C 11213	PACIFIC HWY-42ND ST.(SPRNGFIL	1993	EUGENE-SPRINGFIELD	015	LANE	3.50	4.00	7.50	2	15300
C 11243	MCKENZIE HWY PASSING BAYS	1993	MCKENZIE	015	LANE	1.43	21.98	38.49	3	3300
C 09776	Q ST. - A ST.(SPRINGFIELD)	1984		015	LANE	1.40	0.00	1.40	2	16300
C 10827	MCKENZIE HWY AT MP. 14.5	1990	MCKENZIE	015	LANE	0.45	14.20	14.63	2	4900
C 09978	SPRINGFIELD - LEABURG	1985	MCKENZIE	015	LANE	5.24	2.96	8.31	3	15200
C 11222	SISTERS- TUMALO	1993	MCKENZIE- BEND	017	DESCHUTES	12.6	0.0	12.6	9	6100
C 11270	DESCHUTES RIVER - US 97	1993	MCKENZIE-BEND	017	DESCHUTES	3.13	14.80	18.00	9	9500
C 11271	DESCHUTES RIVER-US 97	1993	MCKENZIE-BEND	017	DESCHUTES	3.13	14.91	18.04	9	8700
C 10770	PASSING LANES HWY 97	1990	MCKNZE-BND,THE DALLES-C	017	DSCHTS,KFA	6.1	112.80	122.30	9	9000
C 10465	LOWER SALT CR.- UPPER SALT C	1987	WILLAMETTE	018	LANE	4.91	36.76	41.70	2	4200
C 10881	RATTLESNAKE CR.- WHEELER RD	1990	WILLAMETTE	018	LANE	0.60	8.80	9.40	3	5600
C 10938	SALMON CR.(OAKRIDGE) BRIDGE	1991	WILLAMETTE	018	LANE	0.55	35.91	36.04	3	4500
C 11331	FREEMAN JCT-HACKETT DR.	1993	THE DALLES-CALIFORNIA	019	KLAMATH	7.23	169.87	177.10	2	1200
C 10704	EMIGRANT CREEK - M.P.4	1989	FAS-A346(DEAD INDIAN RD.)	020	JACKSON	3.1	0.90	4.00	4	22000
C 10760	HAYDEN MOUNTAIN PASS SECT.	1989	GREENSPRINGS	021	KLAMATH	10.30	32.97	43.27	3	460

Table A.1 F-mix highway log (continued)

CONTRACT #	JOB NAME	PLACED	NAME	NUMBER	COUNTY	MILES	M.P.	M.P.	COND	ADT
C 11188	DIST. 8 OVERLAY	1992	GREEN SPRINGS, ROGUE VL	021	JACKSON	.95	20.92	21.87	3	460
C 10239	JENNY CR.- PARKER SUMMIT	1986	GREEN SPRINGS	021	JACKSON	5.30	23.41	28.71	3	460
C 10818	KERN SWAMP RD-WEYRHAUSR R	1990	GREEN SPRINGS	021	KLAMATH	3.00	53.60	56.60	2	3100
C 10433	DISTRICT 8 PAVING	1987	CRATER LK & GREENSPRING	021	JACKSON\KL	2.30	45.50	47.80	3	750
C 15 MISC.	HWY 62- M.P. 40	1993	BUTTE FALLS RD	022	JACKSON	4.00	4.00	8.00	3	27800
C 10649	TRAIL-CASEY(EAST UNIT)	1989	CRATER LAKE	022	JACKSON	2.09	26.90	28.90	1	2900
C 15 MISC.	M.P. 40 CROWFOOT RD.	1992	BUTTE FALLS RD.	022	JACKSON	3.64	4.00	7.64	3	21500
C 11192	MINNIE CR.-BUTCHER KNIFE CR.	1992	REDWOOD	025	JOSEPHINE	5.24	9.08	14.32	1	7000
C 10726	CLACKAMS/BORNG HWY-362ND D	1989	MT. HOOT	026	CLACKAMAS	2.44	19.96	22.74	2	20100
C 10883	CORVALLIS E.C.L.-N.W. RONDO ST	1990	ALBANY-CORVALLIS	031	BENTON	6.10	1.38	7.48	2	9700
C 10833	CORVALLIS E.C.L. NW. RONDO ST	1990	ALBANY- CORVALLIS	031	BENTON	6.10	1.38	7.48	2	9700
C 10917	CORVALLIS BY-PASS (S.UNIT)	1990	CORVALLIS- NEWPORT	033	BENTON,LIN	1.04	56.79	55.75	1	10300
C 10598	GLEN AIKEN CR.- GREY CR.	1988	COOS BAY- ROSEBURG	035	COOS	1.40	15.15	16.55	2	7000
C 10653	CAMAS MT. WAYSIDE- MUNS CRE	1989	COOS BAY- ROSEBURG	035	DOUGLAS	3.54	58.53	62.07	2	3750
C 10846	CAMAS VLY-CAMAS MT WAYSIDE	1991	COOS BAY-ROSEBURG	035	DOUGLAS	4.83	54.23	59.06	2	3750
C 11291	REMOTE CAMPGROUND-SLATER	1994	COOS BAY-ROSEBURG	035	COOS,DUGL	6.12	38.25	46.00	9	3750
C 11110	MYRTLE POINT S.C.L. POWERS JC	1992	COOS BAY\ROSEBURG	035	COOS	1.5	21.83	23.33	2	5100
C 10719	N FORK COQUILLE RV. MYRTLE PO	1989	COOS BAY- ROSEBURG	035	COOS	0.84	19.61	20.45	2	7000
C 10866	GREY CREEK- N. FORK RD.	1990	COOS BAY- ROSEBURG	035	COOS	2.85	16.60	19.45	2	7000
C 11013	COQUILLE REROUTE	1993	COOS BAY-ROSEBURG	035	COOS	1.74	9.60	12.10	3	9300
C 10839	HOOVER HILL RD.-BROCKWAY RD	1990	COOS BAY- ROSEBURG	035	DESCHUTES	1.03	69.40	71.80	1	5300
C 11297	PACIFIC HWY WEST-GATEWAY ST	1993	BELTLINE	036	LANE	6.82	6.11	12.93	3	320
C 10843	SLICK ROCK CR.-SULPHUR CR.	1990	SALMON RIVER	039	LINCOLN	3.98	5.60	9.58	1	5700
C 11228	AIRPORT RD-PACIFIC HWY	1993	SALMON RIVER	039	YAMHILL	4.65	48.00	52.65	1	7700
C 10778	FORT HILL- WALLACE BRIDGE	1990	SALMON RIVER	039	POLK	2.63	24.23	26.86	2	10300
C 10788	ROSELODGE- POLK CNTY LINE	1990	SALMON RIVER	039	LINCN,TLMO	1.90	9.50	11.30	3	21500
C 10991	SALMON RV HWY, THREE RV HWY	1993	SALMON RIVER	039	POLK	.044	22.89	23.33	2	7900
C 10992	SAWTELL RD - M.P. 29	1992	SALMON RIVER	039	POLK	1.61	27.82	29.43	2	10300
C 11364	OCHOCO-SUMMIT-M.P.60.5	1993	OCHOCO	041	WHEELER	10.34	60.5	71.25	1	820
C 11189	MP 34.0 - MP 45.0	1992	OCHOCO	041	CROOK	11.35	34.05	45.40	1	790
C 10432	WEATHERLY CR.- GRAB CR. SEC.	1987	UMPQUA	045	DOUGLAS	2.38	22.75	25.13	2	3500
C 10852	ROCK CR. - ANLAUF SECTION	1990	UMPQUA	045	DOUGLAS	2.23	53.94	56.17	2	4000
C 11035	UMPQUA WAYSIDE-ELKTON	1991	UMPQUA	045	DOUGLAS	4.00	32.07	36.07	1	3500
C 11187	GOLDEN CR.-WEATHERLY CR.	1992	UMPQUA	045	DOUGLAS	2.58	20.10	22.68	2	3700
C 11087	GOLDE CR-WEATHERLY CR.	1991	UMPQUA	045	DOUGLAS	2.58	20.10	22.68	3	3500
C 10863	SCOTTBURG- WELLS CR. SEC.	1990	UMPQUA	045	DOUGLAS	3.0	16.5	19.5	2	3700
C 10923	HANCOCK HILL PASSING LANE	1991	UMPQUA	045	DOUGLAS	1.08	37.10	38.20	2	2800
C 11163	SADDIE MT. JCT.-COAST RANGE	1992	SUNSET	047	CLATSOP	4.2	9.8	14.0	1	3400
C 11302	CEDAR HILLS BLVD INT AUXILRY L	1993	SUNSET	047	WASHINGTO	1.37	68.11	68.67	3	92000
C 11342	MALLER RD-GLENCOE RD SEC.	1993	SUNSET	047	WASHINGTO	5.11	52.30	57.40	9	11800
C 11229	WOLF CR- W. FORK DAIRY CR.	1993	SUNSET	047	WASHINGTO	9.02	37.41	46.43	1	5300
C 10750	COAST RANGE SUMIT-JEWELL JC	1989	SUNSET	047	CLATSOP	7.62	14.05	21.67	2	3500
C 11341	KLAMATH FLS,MALN,LAKVEW,HAT	1993	KLAMATH FALLS-MALIN HWY	050	KLAMATH	1.49	3.78	2.29	9	4500
C 11220	EASIDE BYPASS(KLAMATH FALLS)	1993	KLAMATH FALLS-MALIN	050	KLAMATH	4.78	16.82	12.24	4	4500
C 10780	FROGLAKE- M.P. 83.0 SECT	1990	WARM SPRINGS	053	WASCO	16.98	71.00	83.00	2	3200

Table A.1 F-mix highway log (continued)

CONTRACT #	JOB NAME	PLACED	NAME	NUMBER	COUNTY	MILES	M.P.	M.P.	COND	ADT
C 11269	M.P.66.9 JCT WAPINITA HWY	1993	WARM SPRINGS	053	WASCO	4.35	66.9	62.55	1	3200
C 11270	KAH-NEE-TA JCT-PELTON DAM RD	1993	WARM SPRINGS	053	JEFFERSON	5.91	105.29	111.20	9	4500
C 11360	KAH-NEE-TA JCT.PLTN DAM,W UN	1993	WARM SPRINGS	053	JEFFERSON	2.19	103.01	105.20	9	4250
C 11237	TRAIL-CASEY ST PARK (W.UNIT)	1994	CRATER LAKE	062	JACKSON	4.16	22.75	26.91	3	3550
C 10805	FOREST BOUNDARY- RIVER RD SE	1990	FLORENCE- EUGENE	062	LANE	1.04	12.30	11.30	3	4850
C 10787	PENN RD.-COUGAR PASS SECT.	1990	FLORENCE - EUGENE	062	LANE	0.61	35.00	35.70	2	3600
C 11043	PHOENIS-VLY VIEW RD SEC	1991	ROGUE VALLEY	063	JACKSON	5.15	11.88	17.03	2	10100
C 10210	JACKSON COUNTY OVERLAY	1986	JACKSONVILLE	063	JACKSON	4.09	24.00	28.09	3	1000
C 10455	N.E. WASCO - S.E. DIVISION ST.		CASCADE HWY N.	068	MULTNOMAH	4.00	0.24	4.24	9	21900
C 11194	PACIFIC HWY W.-GATEWAY ST.	1993	BELTLINE	069	LANE	6.68	6.25	12.93	9	38000
C 10620	DISTRICT 6 OVERLAY	1988	NORTH UMPQUA	073	DOUGLAS	4.2	62.00	66.20	3	970
C 10754	FISH CR.- CHINQUAPIN CR.	1989	NORTH UMPQUA	073	DOUGLAS	6.45	56.00	62.45	2	970
C 11165	BOULDER FLAT-FISH CR. BR.	1993	NORTH UMPQUA	073	DOUGLAS	3.30	52.33	55.63	9	970
C 10899	SUSAN CR.- WRIGHT CR. RD.	1990	NORTH UMPQUA	073	DOUGLAS	6.32	27.88	34.20	9	1500
C 11278	SUSAN CR.-USFS BOUNDARY	1993	NORTH UMPQUA	073	DOUGLAS	2.12	28.67	30.79	9	1500
C 10979	STUMP LAKE-WINDIGO	1993	NORTH UMPQUA	073	DOUGLAS	6.70	67.18	73.88	1	970
C 11021	SPRING VLY CR-SALEM TOWNE	1991	SALEM-DAYTON	150	POLK	4.7	12.6	17.3	3	4200
C 10964	N.SANTIAM-ST. PARK-MILL CTY	1992	N. SANTIAM	162	MARION	4.09	24.62	28.71	1	4500
C 10905	SPANGLER HILL-MULINO	1991	CASCADE HWY.S.	160	CLACKAMAS	2.91	8.07	10.71	2	9000
C 11328	PACIFC HWY E.-CLACKMS CNTY L	1993	WOODBURN-ESTACADA	161	MARION	2.59	0.04	2.63	9	5400
C 11303	ECL GATES-LITTLE SWEEDEN SEC	1993	NORTH SANTIAM	162	MARION	4.2	34.20	38.40	9	4000
C 11095	MILL CITY- GATES	1992	N. SANTIAM	162	MARION	3.58	30.03	33.61	1	5500
C 10777	LITTLE N. FORK RD.- M.P.25	1990	NORTH SANTIAM	162	MARION	1.80	23.20	25.00	2	4500
C 10951	FIR GROVE LANE-TOWERS ROAD	1991	NORTH SANTIAM	162	MARION	2.9	17.00	19.70	2	7300
C 10790	MILL CITY- GUN CR. SECT.	1990	NORTH SANTIAM	162	MARION	5.98	29.40	29.60	1	6200
C 10927	LAVA LK MEDOWS RD-SANTIAM SF	1991	N. SANTIAM-SANTIAM	162	LINN	7.71	77.8	80.4	2	4200
C 11254	RIVERSIDE DR.-LAKE CREEK	1993	CORVALLIS-LEBANON	210	LINN	3.26	3.04	6.30	9	16300
C 11152	WILLAMETTE RV.-RIVERSIDE DR.	1992	CORVALLIS-LEBANON	210	LINN	3.33	0.28	3.61	1	21000
C 11304	E.COURTNEY CR. BRIDGE	1993	HALSEY-SWEET HOME	212	LINN	0.24	3.11	3.35	4	3900
C 10601	HENDRICKS RD.- PACIFIC HWY	1988	SPRINGFIELD- CRESWELL	222	LANE	2.96	11.63	14.59	2	2500
C 11285	42ND ST.-MCKENZIE HWY	1993	EUGENE-SPRINGFIELD	227	LANE	2.49	7.47	9.96	9	15000
C 11287	DIST 6 OVERLAY PROJECT	1993	UMPQUA & ELKTON-SUTHER	231	DOUGLAS	3.75	0.00	3.75	9	1400
C 11324	SAMS VLY HWY JCT-SHADY CVR.N	1993	CRATER LAKE	234	JACKSON	2.94	18.56	15.62	3	21000
C 11265	CATCHING SLOUGH BRIDGE	1993	COOS RIVER	241	COOS	0.72	1.74	2.46	3	4000
C 10566	CRATER LAKE HWY- BROWNSBOR	1988	LAKE OF THE WOODS	270	JACKSON	8.22	0.00	8.22	1	3750
C 10607	SAMS VLY. HWY, TABLE ROCK	1988	SAMS VALLEY	271	JACKSON	0.14	10.68	10.82	2	2500
C 10600	JOHNSON CR.- CAMERON RD.	1988	JACKSONVILLE	272	JSPHN,JKSN	14.8	9.20	24.00	3	1900
C 10864	APPLEGATE RV. BRIDGE MP.9.2	1990	JACKSONVILLE	272	JOSEPHINE	3.02	6.18	9.20	3	3350
C 10867	NCL JACKSONVILLE-RIVERSIDE	1990	JACKSONVILLE	272	JACKSON	4.77	34.03	38.80	3	9200
C 10757	POORMANS CR. SECT.	1989	JACKSONVILLE	272	JACKSON	3.1	25.90	29.60	2	5000
C 11077	KIWA SPRINGS-MT. BACHELOR	1992	CENTURY DRIVE	372	DESCHUTES	10.47	21.62	11.15	2	730
C 11077	KIWA SPRING-MT BACHELOR	1992	CENTURY DRIVE	372	DESCHUTES	10.47	11.15	11.15	2	730
C 11351	- - - -	1993	&CHILOQUIN	422	WHEELER	5.34	0.00	5.34	9	5000
C 11197	DIST. 7 OVERLAY	1992	ORE COAST,COOS BAY-ROS	009, 035	COOS	0.52	234.50	235.02		
C 11197	DIST. 7 OVERLAY	1992	ORE COAST,COOS BAY-ROS	009,035	COOS	3.20	281.30	284.50		

Table A.1 F-mix highway log (continued)

CONTRACT #	JOB NAME	PLACED	NAME	NUMBER	COUNTY	MILES	M.P.	M.P.	COND	ADT
C 11162	YOUNGS BAY BR-WARRENTON	1992	ORE COAST & LOWR COLUM	009 & 02W	CLATSOP	4.62	4.15	97.07		
C 10840	PLTN DAM,RIMRK RANCH,JEFRSN	1990	THE DALLES-CAL., WRM SPI	004, 053	JEFFERSON	23.8	91.90	115.70		
C 09799	S.P.R.R. O'XING SEC.	1984	SPRINGFIELD - CRESWELL	222	LANE	14.4	0.0	14.41		
C 11044	DIST 4 OVERLAY PROJECT	1991	PACIFC W.-CORVALLIS NWP	01W, 033	BENTON	5.31	79.75	82.60		
C 11044	DIST 4 OVERLAY PROJECT	1991	PACIFIC W.-CORVALLIS NWF	01W, 033	BENTON	2.36	53.49	51.03		