FINAL REPORT

EFFECT OF MOISTURE AND AGING ON ASPHALT PAVEMENT LIFE

Part 2 - Effect of Aging

HP & R Study: 083:5157

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16. Abstract

This report presents the results of a study to evaluate the effect of oxidative aging on asphalt mixtures. The results of tests on field core samples from 8 projects representing different regions in Oregon were compared with those from laboratory mixture samples for selected projects subjected to accelerated aging tests. The study also involved laboratory tests on asphalt used in those projects selected for the laboratory aging process for mixtures.

The procedure selected for aging laboratory mixtures involved using a Pressure Oxygen Bomb (POB), a sealed container in which mixture and/or asphalt samples were subjected to pure oxygen at 100 psi pressure at 140°F (60°C), for periods of up to five days. The asphalt samples were aged on a Fraass plaque to achieve minimum disturbance of the sample, and the degree aging assessed by change in the Fraass breaking temperature.

The results of this study showed that the POB was an effective means of producing measurable changes in both mixtures and asphalt samples. However, the mixture properties were substantially different to those measured for the field core samples, while the asphalt properties were similar. The study also indicated that aging rate is a function of the air voids in the mixture, and the amount of asphalt and its properties. The study therefore confirms the well established principle that low air voids and thick asphalt films are required to produce durable asphalt mixtures.

17. Key Words

Oxidative aging, asphalt mixtures, cores, pressure oxygen bomb, Fraass breaking temperature aging rate, durability

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DISCLAIMER

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1.0 INTRODUCTION

1.1 Problem Statement

Paving projects in Oregon and other states often show early problems although improved technology and specifications have reduced their occurrence. A previous OSU/ODOT project (1,2,3) evaluated the effects of some mix variables on pavement life, and determined that mix density was the most important, with aggregate gradation and asphalt content of lesser importance. In view of these results, and continued observations on new paving projects further investigations were carried out to include some new variables and to extend the range of some of the variables previously studied. To this end a two phase study to investigate the effects of moisture and aging was undertaken which is outlined below. The results of the first phase concerning the effects of moisture were presented in the accompanying report, Part 1 - Effect of Moisture (4). The first phase was necessitated because of the increased presence of moisture in freshly mixed materials due to recent changes in materials and due to widespread use of dryer drum mixing plants. Associated with the moisture problem, the use of lime and one proprietary antistripping agent was investigated.

The second phase of this study was an investigation of oxidative aging of asphalt mixtures. The results of tests on field core samples from eight projects were compared with those from laboratory mixture samples for selected projects subjected to accelerated aging tests. The study reported herein also involved laboratory tests on asphalt used in those projects selected for the laboratory aging process for mixtures.

1.2 Purpose

The purpose of this phase of the study is to provide a better understanding of the occurrence of long-term aging problems in asphalt pavements in Oregon. The specific objectives of this phase are:

a) To evaluate the change of properties of cores from projects of about five years old and ten years old representing all areas of Oregon,

b) To evaluate the effect of accelerated aging in the laboratory on mechanical properties of asphalt mixtures similar to the cores, including resilient modulus, fatigue life, and Fraass breaking temperature of asphalt cement, and

c) To provide guidelines to minimize the effects of aging on pavement performance.

1.3 Research Approach

The research includes tests on both laboratory-prepared mixtures from three projects and cores from eight projects. Oregon DOT provided cores, core data, all the needed mix design data, materials, and asphalt testing. Oregon State University was responsible for performing all testing to evaluate the effects of aging on cores and laboratory mixtures, i.e., all dynamic tests to evaluate resilient modulus, fatigue, and breaking temperature of asphalt cement. Both as-compacted and aged samples were tested. The aging procedure utilized a modified Pressure Oxygen Bomb (POB) (5).

Following completion of both phases, the final report has been prepared in two parts by Oregon State University summarizing the results for both phases and presenting the conclusions and recommendations resulting from the study.

This report (Part 2) deals specifically with the second phase of the study. It presents (Chapter 2) the experiment design, including projects evaluated, variables considered, test program and methods, and sample preparation. Chapters 3 and 4 present the test results and discussion. Chapter 3 presents the data for core tests, mix design, modulus, fatigue, and Fraass breaking temperature of asphalt cement. Chapter 4 contains the discussion of the effects of aging on the cores and mix performance. Finally, Chapter 5 presents the conclusions and recommendations for this phase.

2.0 EXPERIMENT DESIGN

2.1 Background

Premature failure or poor performance of asphalt pavements often results from weakening of the binding forces in the pavement by the action of moisture and/or mechanical stresses, and/or aging of asphalt cement. Aging is the change of properties of asphalt pavements (6). Petersen (7) indicated that three fundamental composition-related factors govern the changes that could cause hardening of asphalts in pavements:

1) loss of the oily components of asphalt by volatility or absorption by porous aggregates;
2) changes in the chemical composition of asphalt molecules from reaction with atmospheric oxygen; and
3) molecular structuring that produces thixotropic effects (steric hardening).

Oxidation of asphalt is generally considered as a major factor contributing to the hardening and embrittlement of asphalt pavement (8-10). This oxidation occurs both in the preparation and laydown of hot mix pavements and from environmental aging while in service. Excessive hardening of the asphalt cement has been considered undesirable because it often leads to problems associated with pavement embrittlement and cracking. The rate of hardening is affected by the chemical composition of asphalt, light, aggregate properties, and the ambient temperature (11-14).

After the development of the recovery test by Abson in 1933 (15) provided a means of recovering the asphalt from hot plant mixtures immediately and after periods of aging in the pavement, numerous artificial aging methods to predict the properties of aged asphalt cement have been developed. Many aging methods attempt to correlate short-term laboratory aging with the change of asphalt properties occurring after long exposure in the pavement as well as during mixing operations.

Methods used to correlate the aging by various investigators include use of high temperatures, light, chemical oxidation agents, and oxidation in solution under oxygen pressure. Table 2.1 shows the various test conditions, such as temperature and exposure time, used for aging methods along with evaluation parameters. Most of the evaluation parameters consist of measurements of consistency of the asphalt cement, such as penetration, viscosity, and ductility, as presented in Table 2.1.

For asphalt mixtures, Pauls and Welborn (24) used the compressive strength of the weathered mixtures as the evaluation parameters. Kemp and Predoehl (32) used the resilient modulus of the weathered briquettes as the evaluation parameter.

2.2 Selection of Aging Method

After review of the various aging procedures used previously (Table 2.1), it was decided that the method adopted in this study should attempt to reproduce oxidative aging occurring after construction of a pavement. The research approach was to compare artificially aged laboratory prepared samples with samples aged in the field. Also, since mixture samples and asphalt samples were to be tested, an aging device suitable for both samples should be used.

The pressure oxygen bomb (POB), originally developed in Britain (5) and recently used by Edler, et al. (34) in South Africa, was selected as the most suitable aging device. This device can contain two mixture samples or one mixture sample and several Fraass samples. As reported by Thenoux, et al. (35), the use of Fraass samples for aging asphalt has the advantage of minimal

Table 2.1. Laboratory Accelerated Aging Test and Evaluation Methods.

Date	Investigator(s)	Test Method	Evaluation Method
1903	Dow (16)	18,24 hours, 325°F (163°C)	Change in weight, penetration of residue
1903	Dow (reference unknown)	Mixture aged for 30 min., 300°F (149°C)	Recovered asphalt - change in penetration
1937	Nicholson (17)	Air blowing, 15 min., 425°F (229°C)	Penetration, ductility
1937	Raschig and Doyle (18)	Air blowing, 15 min., 400°F (204°C)	Change in penetration
1937	Hubbard and Gollomb (19)	Mixture, time and temperature varied	Recovered asphalt - change in penetration
1939	Lang and Thomas (20)	Mixture, aging oven, outdoor exposure	Change in mix properties, abrasion, strength, etc.
1940	Shattuck (21)	Mixture oven aging 30 min., 325°F (163°C)	Recovered asphalt - penetration ductility, soft point
1940	Lewis and Welborn (22)	1/8-inch film oven test 5 hr., 325°F (163°C) TFOT	Change in weight, penetration, ductitlity
1946	Lewis and Halstead (23)	1/8-inch film oven test 5 hr., 325°F (163°C)	Change in weight, penetration, ductility
1952	Pauls and Welborn (24)	Mixture oven aged 325°F (163°C) Thin-film oven test	Compressive strength, recovered asphalt, TFOT residues
1955	Griffin, Miles, and Penther (25)	Shell Microfilm test - 5 micron (.0002-in.) film, 2 hr., 225°F (107°C)	Viscosity before and after aging - aging index
1958	Heithaus and Johnson (26)	Road tests - laboratory aging - microfilm test	Recovered asphalts Microfilm Aging Index
1960	Traxler (27)	TFOT and microfilm 15 micron (.0006 in.) film, 2 hr. 225°F (107°C)	Microviscosity at 77°F (25°C) compared
1961	Halstead and Zenewitz (28)	TFOT and 15 micron film (.0006 in.) film, 2 hr., 225°F (107°C)	Microviscosity at 77°F (25°C) compared
1963	Hveem, Zube and Skog (29)	Shell microfilm test modified - 20 microns (.0008 in.) 24 hrs at 210°F (99°C)	Microviscosity at 77°F (25°C) before and after aging
		Rolling thin-film oven test (RTFOT) and TFOT 325°F (163°C), 50 min.	Viscosities of RTFOT, TFOT, and recovered asphalts compared
1969	Schmidt and Santucci (30)	Rolling microfilm test 20 microns (.0008 in. bottle) 210°F (99°C)	Microviscosity of residue
1976	Petersen, Ensley, Plancher, and Harnes (31)	Asphalt-coated aggregates, 302°F (150°C), 4.5 hr.	Chemical composition of extracted asphalts
1981	Kemp and Predoehl (32)	Tilt-oven durability test 168 hr. 235°F (113°C) Mixtures weathered in field	Penetration 77°F (25°C) Ductility 77°F (25°C) Resilient Modulus (M_R)

Table 2.1. Laboratory Accelerated Aging Test and Evaluation Methods (Continued).

Date	Investigator(s)	Test Method	Evaluation Method
1983	McHattie (33)	Extended RTFOT (ERTFOT) 100 hr. 239°F (115°C)	Penetration 77°F (25°C) Kinematic viscosity 275°F Resilient Modulus EAL (Equivalent Axle Load) Life
1985	Edler, et al. (34)	Weatherometer ERTFOT Pressure Oxidation Bomb (POB) 96 hr. 149°F (65°C) Modified TFOT	Viscosity Oxidation level - Infrared spectra High molecular weight constituents

disturbance to the asphalt which is tested on the "container" on which it is aged.

The POB's used in this study were of a slightly different design to that originally used (5) and were operated with the samples contained in a pure oxygen environment at 100 psi and at 60°C. These levels were selected after due consideration of safety concerns regarding the use of pressurized oxygen.

2.3 Projects Evaluated

For the aging study reported herein, eight projects constructed in Oregon, four of which are approximately 10 years old [Arnold Ice Caves-Horse Ridge (1973), Idylwood Street (1974), S.F. Malheur River-Malheur Caves (1974), Elk River-Port Orford (1976)] and four of which are approximately 5 years old [Pacific Highway (1980), Plainview Road-Deschutes River (1980), Grande Ronde-Wallace Bridge (1980), and N. Klamath Falls Junction-Green Springs Highway (1981)] were selected. The location of each project is indicated in Figure 2.1. Of those eight projects, three projects [Idylwood Street (1974), Plainview Road-Deschutes River (1980), Arnold Ice Caves-Horse Ridge (1973)] were selected for the laboratory simulation aging study.

2.4 Core Sampling

For each project selected for the study, seven 4-in. (102 mm) diameter and six 6-in. (152 mm) diameter cores were taken from the wheel track in the travel lane as well as from one foot outside the fog line at each of two stations. The condition of pavement, ADT (average daily traffic) and percentage of truck traffic were recorded.

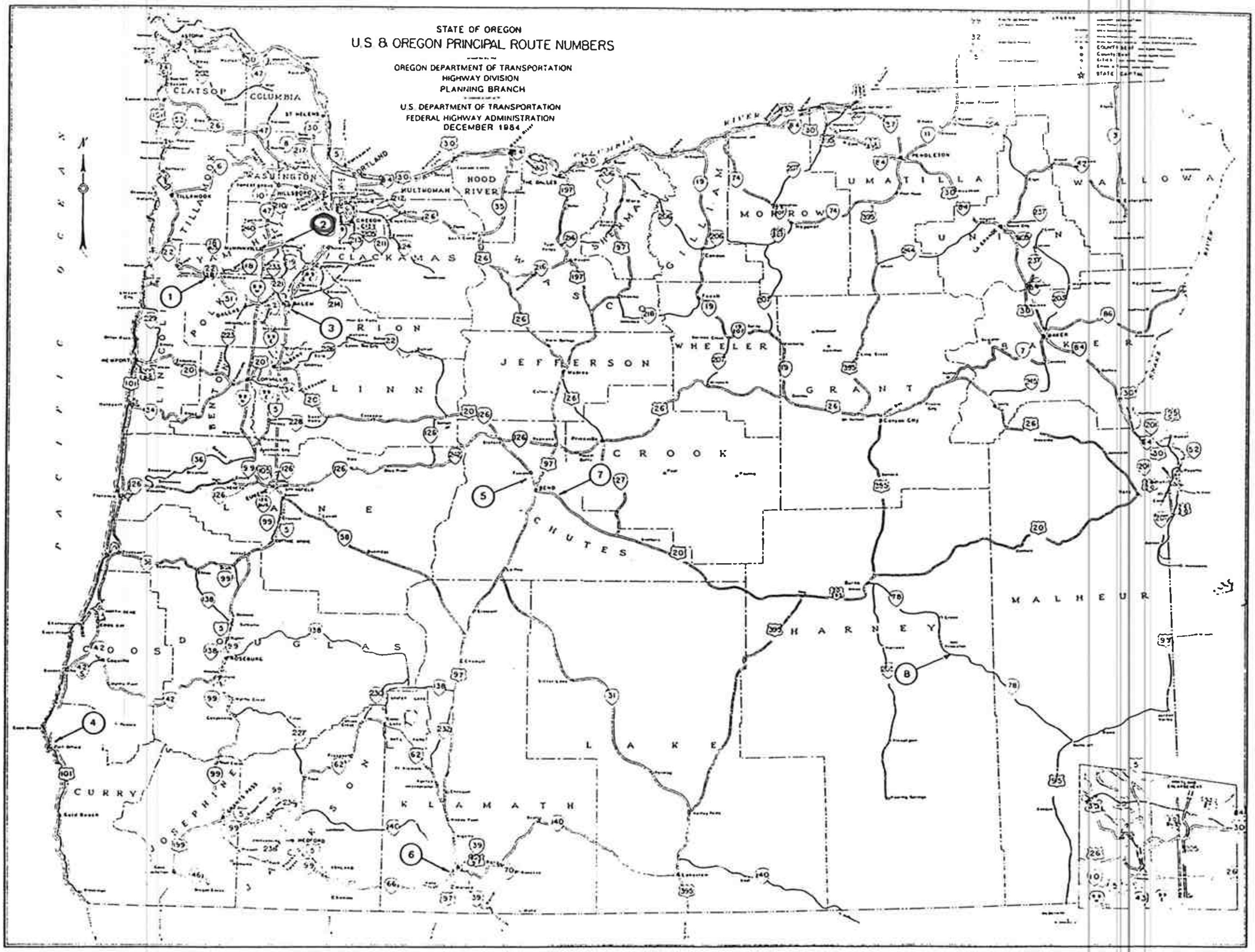


Figure 2.1. Map of Project Locations.

2.5 Variables Considered

The variables considered in the laboratory mixture preparation were:

1) Compaction level: 94%, 88%;

2) Aging period: 0, 1, 2, 3, and 5 days.

Each of the above variables was studied relative to a standard mix consisting of the original mix design used for the projects studied.

2.6 Test Program

2.6.1 Cores

A flow chart of the test program followed in this study is given in Figure 2.2. The Oregon Department of Transportation (ODOT) testing program included the tests for aggregate gradation, asphalt cement contents, air voids, and recovered asphalt cement properties. The repeated load diametral test for modulus and fatigue life of cores was performed by Oregon State University.

2.6.2 Laboratory Mixtures

Following the standard ODOT procedure (36), 4-in. (100 mm) diameter by 2.5-in. (63 mm) high specimens were fabricated for three projects [Idylwood Street, Plainview Road-Deschutes River and Arnold Ice Caves-Horse Ridge] by using the same asphalt cement and same mix design employed at the time of construction. A minimum of 10 specimens for each compaction level were prepared for each of the three projects. All 60 specimens were tested for resilient modulus and fatigue. All tests were run for tensile stress levels of 20 psi and 40 psi for 88% compaction level and 94% compaction level, respectively.

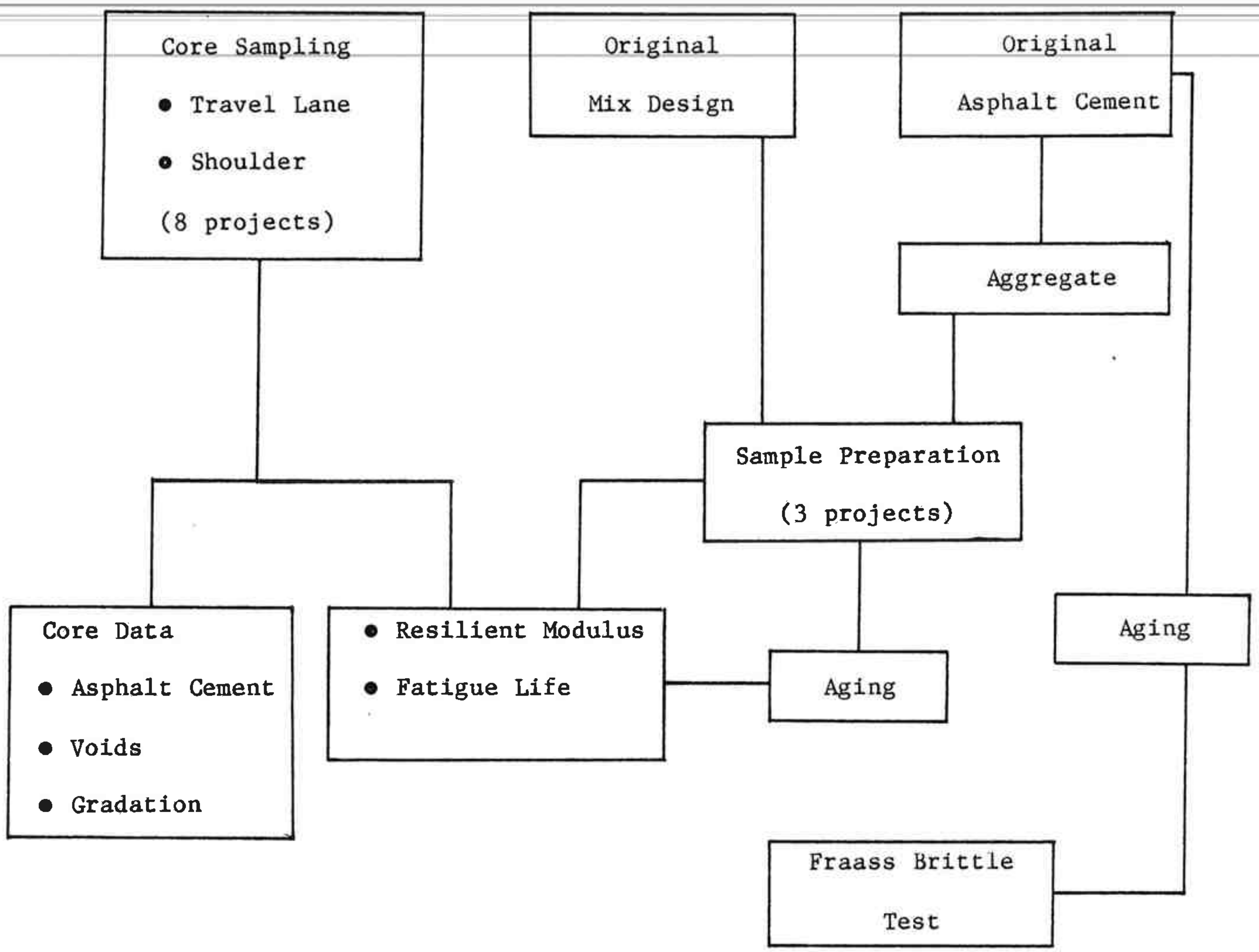


Figure 2.2. Flow Chart of the Test Program.

2.7 Test Methods

2.7.1 Repeated Load Diametral Test

The resilient modulus and fatigue tests were performed using the repeated load diametral test apparatus (ASTM D 4123-82). The test procedures employed are essentially the same as used in previous studies (2,3,4). In summary:

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

The parameters recorded during the repeated load diametral test are the maximum load applied, the horizontal elastic deformation, and the number of repetitions to failure. During the tests, the dynamic load duration was fixed at 0.1 sec and the load frequency at 60 cycles per minute. A static load of 10 pounds (4.5 kg) was applied to hold the specimen in place. The tests were carried out at 70.7 ± 1.6°F (21.5 ± 0.9°C).

Resilient Modulus

The maximum load applied and the horizontal elastic tensile deformation were recorded to determine the resilient modulus using the following equations:

$$M_R = \frac{P}{\Delta Hxt} (.2692 + .9974\nu) \tag{2.1}$$

where:

M_R = Resilient modulus, psi;
ΔH = Horizontal elastic tensile deformation, inches;
P = Dynamic load, lbs;
t = Specimen thickness, inches; and
ν = Poisson's ratio.

Poisson's ratio was assumed constant and equal to .35, which simplified Eq. (2.1) to:

$$M_R = \frac{0.6183P}{\Delta H x t} \tag{2.2}$$

Fatigue Life

Fatigue has been defined (37) as "the phenomenon of fracture under repeated or fluctuating stress having a maximum value generally less than the tensile strength of the material." However, the failure, or the end point of a fatigue test, in the laboratory has been defined by investigators in many ways. It may be the point corresponding to complete fracture of the test specimen, the point at which a crack is first observed or a certain width of crack in the specimen is detected, or the point at which the stiffness or some other property of the specimen has been reduced by a specific amount from its initial value (38).

For this study, the number of load repetitions to fatigue failure was defined as the number of repetitions required to get a vertical crack approximately 0.25 inch (.64 cm) wide in the specimens. To stop the test at the specified level of specimen deformation, a thin aluminum strip was attached to the sides of the specimens, along a plane perpendicular to the plane formed by the load platen. The aluminum strip is connected to a normally closed relay, which controls the dynamic load system. As the specimen deforms, the aluminum strip is stressed. When the specimen deformation exceeds a certain level, the aluminum strip breaks and opens the relay, which shuts off the test. Proper calibration of the length of the aluminum strip will cause the test to stop for specific specimen crack width.

In fatigue testing of asphalt mixtures, the logarithm of the life expressed in number of repetitions to failure is a linear function of the logarithm of the initial strain or stress:

$$N_f = K \left(\frac{1}{\varepsilon_t}\right)^m \tag{2.3}$$

$$N_f = C \left(\frac{1}{\sigma_t}\right)^n \tag{2.4}$$

where:

N_f = Number of load repetitions to failure;

K, m, C, n = Regression constants;

ε_t = Initial horizontal elastic tensile strain; and

σ_t = Tensile stress, psi.

The initial horizontal elastic tensile strain, ε_t, is calculated from the following equation:

$$\varepsilon_t = \Delta H \left[\frac{.03896 + .1185\nu}{.0673 + .2494\nu}\right] \tag{2.5}$$

where:

ε_t = Initial horizontal elastic tensile strain;

ΔH = Horizontal elastic tensile deformation, inches; and

ν = Poisson's ratio.

Assuming again the Poisson's ratio is constant and equal to .35, Eq. (2.5) becomes:

$$\varepsilon_t = \Delta H \text{ x } .5203 \qquad (2.6)$$

The tensile stress is calculated from the following equation:

$$\sigma_t = \frac{2P}{\Pi D t} \qquad (2.7)$$

where: D = Specimen diameter.

2.7.2 Aging Procedure

The modified pressure oxidation bomb, which was developed originally in England (5), consists of a cylindrical pressure vessel fitted with a screw-on cover containing a safety blow-off cap, a pressure gauge, and a stopcock. Figure 2.3 shows a diagram of the pressure oxidation bomb (POB) used for this study.

The following are the main steps in the use of the POB:

1) Samples (asphalt mixtures or asphalt cement) are prepared.
2) The samples are placed in a bomb.
3) The vacuum [26 in. (66 cm) Hg.] is applied for 20 minutes.
4) The bomb is filled via the stop cock from an oxygen cylinder to 100 psi (689.5 kPa). This pressure is held for 30 minutes to ensure leak-free joints.
5) The bomb is then placed in an oven maintained at 60°C (140°F) for a period of time such as 1,2,3, and 5 days.
6) At the conclusion of the test the stopcock is opened, the cover is removed, and the aged mixtures and/or asphalt cement samples are cooled for one day and two hours under room temperature, respectively.

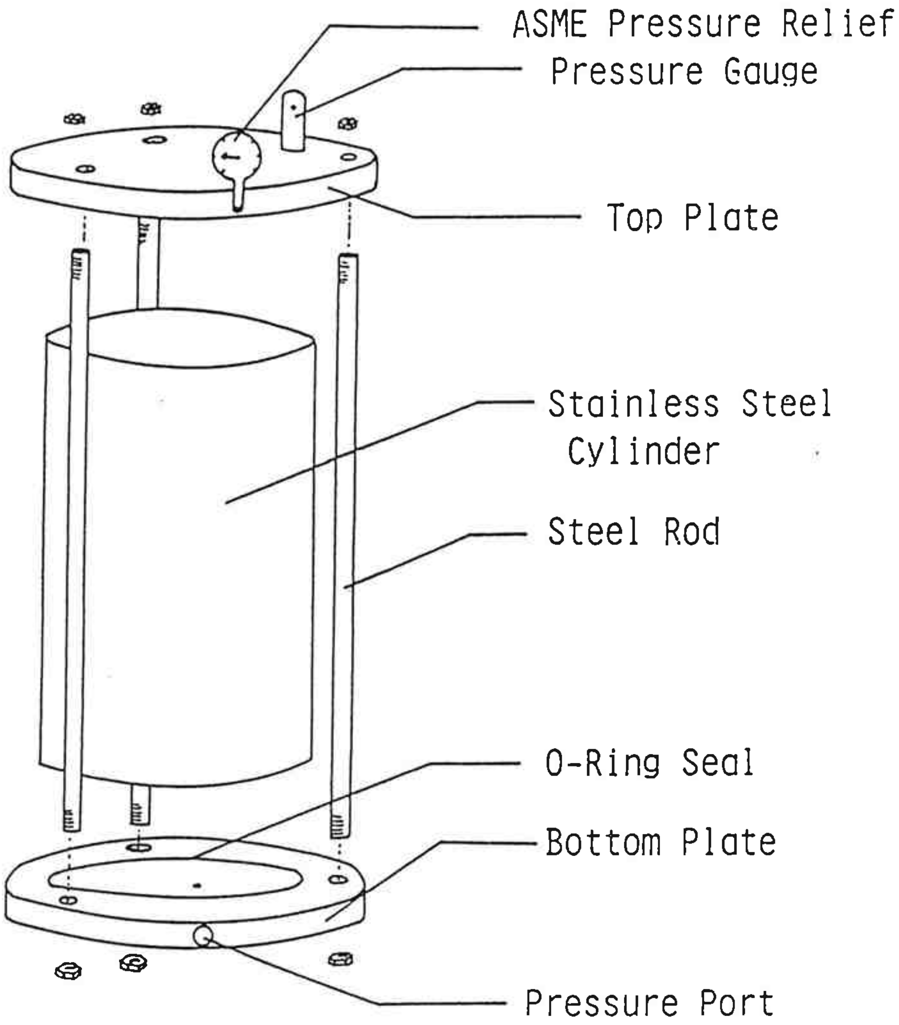


Figure 2.3. Pressure Oxidation Bomb (POB).

2.7.3 The Fraass Brittle Test

The Fraass breaking point (39) is the temperature at which a binder first becomes brittle as indicated by the appearance of cracks when a thin film of binder on a metal plaque is cooled at the rate of 1°C/min. and flexed at a constant rate. This test (outlined in the Appendix) was described by Fraass in 1937, and has been standardized through the world including Canada, Finland, Germany, Italy, Norway, and Sweden. Figure A1 in the Appendix shows the component parts of the apparatus. The following are the main steps used for this study.

1) The sample (0.4 gr) is prepared as described in the Appendix.
2) A standard steel plaque [1.6 in. x 0.8 in. (41 mm x 20 mm)] is coated with a thin layer of asphalt cement [0.02 in. (0.5 mm)].
3) The steel plaque coated with asphalt cement is in a closed chamber, and the temperature of the plaque is lowered steadily at a rate of 1°C/min (1.8°F/min), adding solid carbon dioxide to the acetone bath which surrounds chamber where the plaque is located.
4) The steel plaque is repeatedly bent to a given extent in a standard time. The temperature at which one or more cracks appear is recorded as the breaking point ("brittle temperature"). The Appendix gives a more detailed description of the apparatus, the sample preparation and test procedure.

3.0 TEST RESULTS

3.1 Cores

3.1.1 Mix Design

The summaries of the original mix design for the projects of Idylwood Street, Plainview Road-Deschutes River, and Arnold Ice Caves-Horse Ridge are presented in Table 3.1. For the other projects, asphalt cement grade and mix type used are summarized in Table 3.2.

3.1.2 Core Data

Two core sampling stations were selected for each project as shown in Table 3.3. For each station, wheel track wear, surface condition and average daily traffic (ADT) were recorded. Cores from the top layer and bottom layer of each station were collected. Six 6-in. (152 mm) diameter cores were tested for aggregate gradation, asphalt cement content, air voids, moisture contents and recovered asphalt cement properties by ODOT.

The core aggregate gradations for the top mixes are shown in Table 3.4. The diametral test was run by OSU to measure the moduli values and fatigue life characteristics with six 4-in. (102 mm) diameter cores from one of the two stations sampled for each project. The moduli values of the cores from the travel lane of each project and from the shoulder of two projects (the Plainview Rd.-Deschutes River and Arnold Ice Caves-Horse Ridge) are presented in Table 3.2. After resilient modulus was measured, the fatigue test was run at fixed tensile stress ranging from 30 psi (207 kPa) to 60 psi (414 kPa). The fatigue life of asphalt mixtures is a function of applied tensile strain or stress and may be expressed by Eq. (2.3) or (2.4). The constants K and m or C and n were determined by linear regression analysis following completion of a test.

Table 3.1. Original Mix Design Data

a) 3. Idylwood Street

	Mix Properties				
Asphalt Content (%) (Chevron AR 4000)	6.0	6.5	7.0	7.5	8.0
Asphalt Film Thickness	Dry-Suff	Suff	Suff-Th	Thick	U.Thick
Stability Value @ Fist Compaction	39	45	42	40	40
Cohesion Value @ First Compaction	241	245	248	275	286
Bulk Sp. Gravity @ First Compaction	2.23	2.25	2.27	2.29	2.39
Percent Voids @ First Compaction	9.3	7.8	6.2	4.6	2.9
Stability Value @ Second Compaction	51	54	50	49	48
Bulk Sp. Gravity @ Second Compaction	2.29	2.31	2.33	2.35	2.35
Prcent Voids @ Second Compaction	6.9	5.3	3.7	2.1	1.2
Real Gravity @ 0% Voids (Rice Method - AASHTO T-209)	2.46	2.44	2.42	2.40	2.38

Recommended Asphalt Content (wearing surface): 7.0%

b) 5. Plainview Road-Deschutes River

	Mix Properties				
Asphalt Content (%) (Chevron 9R4000 W)	4.5	5.0	5.5	6.0	6.5
Asphalt Film Thickness	Dry	Dry-Suff	Suff	Suff	Suff-Th
Stability Value @ Fist Compaction	44	44	42	42	43
Cohesion Value @ First Compaction	-	-	-	-	-
Bulk Sp. Gravity @ First Compaction	2.34	2.36	2.38	2.39	2.40
Percent Voids @ First Compaction	7.9	6.3	4.8	3.6	2.8
Stability Value @ Second Compaction	51	53	51	49	47
Bulk Sp. Gravity @ Second Compaction	2.39	2.42	2.44	2.45	2.46
Prcent Voids @ Second Compaction	5.9	4.0	2.4	1.2	0.0
Real Gravity @ 0% Voids (Rice Method - AASHTO T-209)	2.54	2.52	2.50	2.48	2.46

Recommended Asphalt Content (wearing surface): 5.5%

c) 7. Arnold Ice Caves-Horse Ridge

	Mix Properties				
Asphalt Content (%) (Douglas 120/150)	5.5	6.0	6.5	7.0	7.5
Asphalt Film Thickness	Dry	Dry-Suff	Suff	Suff-Th	Thick
Stability Value @ Fist Compaction	45	46	48	51	52
Cohesion Value @ First Compaction	183	235	238	353	389
Bulk Sp. Gravity @ First Compaction	2.30	2.32	2.34	2.36	2.38
Percent Voids @ First Compaction	8.7	7.2	5.6	4.1	2.5
Stability Value @ Second Compaction	51	52	59	52	43
Bulk Sp. Gravity @ Second Compaction	2.36	2.38	2.40	2.42	2.43
Prcent Voids @ Second Compaction	6.3	4.8	3.2	1.6	0.4
Real Gravity @ 0% Voids (Rice Method - AASHTO T-209)	2.52	2.50	2.48	2.46	2.44

Recommended Asphalt Content (wearing surface): 6.5%

Table 3.2. Summary of Core Data.

Project	Thickness (in.)	Max. Sp. Gr.	BSG	Air Voids (%)	A/c (%)	A/C Retention (%)	H_2O (%)	Asphalt Supplier & Grade	Mix Type	M_r (ksi)
1. Grande Ronde-Wallace Bridge (1980)	1.72	2.476	2.20	11.1	5.0	0.1	0.89	Chevron AR-4000W	B-mix	862.67 (128.72)*
2. Pacific Hwy W. Dayton-Lafayette (1980)	2.44	2.580	2.36	8.5	5.7	0.0	0.45	Chevron AR-4000W	B-mix	1103.19 (123.83)
3. Idylwood St. (1974)	1.90	2.459	2.17	11.8	5.9	0.0	0.59	Chevron AR-4000	B-mix	771.87 (106.81)
4. Elk River-Port Orford (1976)	1.44	2.421	2.30	5.0	7.0	0.1	0.47	Douglas AR-4000	B-mix	281.94 (74.47)
5. Plainview Rd.-Deschutes River (1980)								Chevron AR-4000	B-mix	
a. Travel	1.41	2.497	2.29	8.3	5.8	0.2	0.29			568.97 (105.63)
b. Shoulder	1.83	2.484	2.26	9.0	5.6	0.2	0.36			703.78 (37.80)
6. N. Klamath Falls Jct. (1981)	2.49	2.535	2.38	6.1	5.2	0.0	0.44	Witco AR-2000	B-mix	1031.63 (197.02)
7. Arnold Ice Cave-Horse Ridge (1973)								Douglas 120/150 pen	B-mix	
a. Travel	1.55	2.444	2.34	4.3	6.7	0.0	0.52			243.70 (31.82)
b. Shoulder	1.92	2.434	2.33	4.3	6.9	0.0	0.40			186.30 (20.55)
8. S.F. Malheur R. (1974)	1.44	2.158	1.97	8.7	7.6	0.6	0.89	Shell AR-2000	C-mix	621.94 (85.51)

*One standard deviation

Table 3.3 Core Sampling Location and Pavement Conditions.

	Project	Highway	Location of Cores	Number of Cores	Wheel Track Wear (Depth)	% of Surface with Defect	Comments
1.	Grande Ronde-Wallace Bridge (1980)	Salmon R.	+ M.P. 25.9, E.B., 9.6' Rt. of CL outside wheel track	7-4" 6-6"	N/A	N/A	ADT 5100-8900
2.	Pacific Hwy. W-Dayton Jct & Dayton Jct-Lafayette (1980)	Salmon R. & Pacific W.	+ M.P. 50.02, W.B., 13.5' Lt. of CL (travel lane)	7-4" 6-6"	N/A	N/A	
3.	Idylwood St. (City of Salem) (1974)	City Street	• E.B. 14.2' S. of CL, 165' E. of Center of Fir Dell Dr. & Idylwood Int.	7-4" 6-6"	N/A	N/A	
			• E.B. 8.5' S. of CL, 165' E. of Center of Fir Dell Dr. & Idylwood Int.	7-4" 6-6"	N/A	N/A	
			+ W.B. 8' N of CL & 9' E. of Idylwood/Lone Oak Int.	7-4" 6-6"	N/A	N/A	
			• W.B. 13' N of CL & 9' E. of Idylwood/Lone Oak Int.	7-4"	N/A	N/A	
4.	Elk River-Port Orford (1976)	Oregon Coast	• M.P. 298.8, N.B., 20' Rt. of CL in Rt. wheel track	7-4" 6-6"	N/A	N/A	
			• M.P. 298.8, N.B., 1' Rt. of fogline	7-4" 6-6"	N/A	N/A	
			+ M.P. 297.8, S.B., 21' Rt. of CL in Rt. wheel track	7-4" 6-6"	N/A	N/A	
			• M.P. 297.8, S.B., 1' Rt. of fogline	7-4" 6-6"	N/A	N/A	
5.	Plainview Road-Deschutes River (1980)	McKenzie-Bend	+ M.P. 14.5, W.B., 16' from CL (travel lane)	7-4" 6-6"	1/4"	5% raveled 5% cracked	3550 ADT, 13.3% truck 4" over old pavement
			+ M.P. 14.5, W.B., 20' from CL (shoulder)	7-4" 6-6"	N/A	N/A	3550 ADT, 13.3% truck 4" over old pavement
			• M.P. 10, E.B., 9' from CL (travel lane)	7-4" 6-6"	1/8"	5% raveled 5% cracked	3550 ADT, 13.3% truck 4" over old pavement
			• M.P. 10, E.B., 13' from CL (shoulder)	7-4" 6-6"	N/A	N/A	3550 ADT, 13.3% truck 4" over old pavement
6.	N. Klamath Falls Jct.-Green Springs Hwy. (1981)	The Dalles-Calif.	+ M.P. 274, S.B., 21' from CL	7-4" 6-6"	0.0	5% raveled 15% cracked	4680 ADT, 36.8% truck
			• M.P. 274, S.B., 24' from CL (shoulder)	7-4" 6-6"	N/A	N/A	4680 ADT, 36.8% truck
			• M.P. 276, N.B., outside lane, 26' from CL	7-4" 6-6"	1/4"	3% raveled 5% cracked	9600 ADT, 36.8% truck
			• M.P. 276, N.B., outside fogline 30.5' from CL	7-4" 6-6"	N/A	N/A	9600 ADT, 36.8% truck

Table 3.3 Core Sampling Location and Pavement Conditions (Continued).

Project	Highway	Location of Cores	Number of Cores	Wheel Track Wear (Depth)	% of Surface with Defect	Comments
7. Arnold Ice Caves - Horse Ridge (1973)	Central Or.	+ M.P. 16, W.B., 9' from CL	7-4" 6-6"	1/2"	25% shoved or rutting	1350 ADT, 14.8% truck
		+ M.P. 16, W.B., 13' from CL (shoulder)	7-4" 6-6"	N/A	N/A	1350 ADT, 14.8% truck
		• M.P. 14, E.B., 9' from CL	7-4" 6-6"	1/4"	10% cracking	1350 ADT, 14.8% truck
		• M.P.14, E.B., 13' from CL (shoulder)	7-4"	N/A	N/A	1350 ADT, 14.8% truck
8. S. Fork Malheur River-Malheur Caves (1974)	Steens	+ M.P. 48.01, E.B., outside wheel track, 9.4' Rt	7-4" 6-6"	1/4"	95-100% trans. cracks 10-15% spalling	190 ADT, 5% truck
		• M.P. 48.01, E.B., 1' outside fog stripe, 13.4' Rt	7-4" 6-6"	N/A	5% raveling 1~3% flushing	190 ADT, 5% truck
		• M.P. 51.96, N.B., outside wheel track, 9' Lt	7-4" 6-6"	3/8"	5% raveling 1~3% flushing	190 ADT, 5% truck
		• M.P. 51.96, N.B., 1' outside fog stripe, 13' Lt	7-4" 6-6"	N/A	N/A	190 ADT, 5% truck

\+ = cores tested

N/A = Not available

Table 3.4. Core Gradation Data.

Project Size	1. Grande Ronde-Wallace Bridge	2. Pacific Hwy. W-Dayton Jct & Dayton Jct-Lafayette	3. Idylwood St.	4. Elk River-Port Orford	5. Plainview Rd-Deschutes River	6. N. Klamath Falls Jct.-Green Springs	7. Arnold Ice Caves-Horse Ridge	8. S.F. Malheur Rd-Malheur Caves Rd.
Passing								
1"	-	-	-	-	-	-	100 (100)	-
3/4"	99	100	100	100	100 (100)*	100	97 (99)	-
1/2"	85	90	89	99	90 (87)	91	87 (91)	100
3/8"	80	78	78	89	76 (74)	84	78 (81)	90
1/4"	69	65	71	69	61 (60)	74	66 (69)	72
#4	55	54	60	58	51 (50)	63	56 (59)	60
#10	30	31	35	34	28 (28)	36	36 (37)	35
#40	13	15	15	16	13 (12)	16	19 (19)	15
#200	5.3	7.0	5.4	4.7	6.8 (6.3)	7.7	8.2 (7.1)	6.5

( )*: shoulder

Table 3.5 presents the constants K, m, C, and n along with the coefficient of correlationship (r). The original and recovered asphalt cement properties are presented in Table 3.6.

3.2 Laboratory Mixtures

For the laboratory aging simulation study, the original mix design given in Table 3.1 were used. In order to obtain the moduli values of each project, two specimens were tested as compacted and the other eight specimens were tested both before and after aging for each compaction level. The aged specimens were cooled for one day before modulus was measured. The modulus test results including specific gravity, air voids, and maximum specific gravity (AASHTO T-209) are summarized in Table 3.7. The aging effect assessed by the modulus ratio (the ratio of modulus after aging to the modulus before aging) is shown in Figure 3.1. After resilient modulus was measured, the fatigue test was run at fixed tensile stress of 20 psi (137.9 kPa) for the specimens of 88% compaction level and 40 psi (275.8 kPa) for those of 94% compaction level. Figure 3.2 illustrates the effect of aging on fatigue life.

3.3 Fraass Test

The asphalt cement used for the laboratory mixture as well as the projects was exposed to the pure oxygen on Fraass samples using POB at 60°C (140°F) and 100 psi (68 kPa) during the different periods of time. A minimum of 24 samples for each project were prepared for the Fraass test. Eight samples for each project were tested before aging. Eight samples for two days and the other eight samples for five days were aged. The average breaking temperatures of the original asphalt cement (before aging) and aged asphalt cement (for 5 days) are given in Figure 3.3 for the three projects studied in depth.

Table 3.5. Fatigue Data of Cores.

Project	$N_f = K(\frac{1}{\varepsilon_t})^m$			$N_f = C(\frac{1}{\sigma_t})^n$		
	K	m	r	C	n	r
1. Grande Ronde-Wallace Bridge	1.517×10^{-4}	2.181	0.914	6.564×10^{10}	3.672	0.971
2. Pacific Highway	1.804×10^{-7}	2.686	0.976	2.995×10^{9}	3.187	0.964
3. Idylwood Street	1.195×10^{-11}	4.091	0.748	6.873×10^{17}	7.558	0.826
4. Elk River-Port Orford	N/A	N/A	N/A	N/A	N/A	N/A
5. Plainview Rd.-Deschutes River	2.250×10^{-4} (1.145×10^{-17})*	2.069 (5.513)	0.925 (0.957)	9.354×10^{8} (4.976×10^{13})	2.923 (5.573)	0.949 (0.990)
6. N. Klamath Falls Jct.	6.166×10^{-8}	2.706	0.718	-	-	-
7. Arnold Ice Caves-Horse Ridge	4.510×10^{-9} (1.173×10^{-11})	3.454 (4.301)	0.960 (0.980)	6.309×10^{11} (7.348×10^{11})	5.091 (5.214)	0.994 (0.983)
8. S.F. Malheur R.	3.272×10^{-14}	4.607	0.959	3.572×10^{12}	5.088	0.868

( )*: shoulder

Table 3.6. Asphalt Cement Data

a. Physical Properties (Original (O) and Recovered (R))

	1. Grande Ronde-Wallace Bridge			2. Pacific Highway			3. Idylwood St.			4. Elk River-Port Orford		
	O	RTFOT*	R	O	RTFOT	R	O	RTFOT	R	O	RTFOT	R
Penetration												
at 25°C (77°F)	73	39	11	73	39	15	139	66	10	134	65	51
at 4°C (39.2°F)	18		2	18		4	50		9	49		26
Penetration Ratio (4°C/25°C)	0.247		0.182	0.247		0.267	0.360		0.90	0.366		0.510
Absolute Viscosity (60°C, Poises)	1552	4191	59284	1552	4191	13299	1169	4306	225129	1110	4344	3403
Kinematic Viscosity (135°C, C.S.)	352	572	1933	352	572	572	353	608	3952	340	633	399
Flash Point (Closed Cup, °C)	307			307			199			235		
Loss on Heating (%)	0.63			0.63			1.77			1.43		

	5. Plainview Rd-Deschutes River			6. N. Klamath Falls Jct.			7. Arnold Ice Caves-Horse Ridge			8. S.F. Malheur R.		
	O	RTFOT*	R	O	RTFOT	R	O	RTFOT	R	O	RTFOT	R
Penetration												
at 25°C (77°F)	80	46	22 (26)**	85	62	27	140	66	63 (80)	84	60	11
at 4°C (39.2°F)	20		14 (12)	17		11	46		32 (31)	15		6
Penetration Ratio (4°C/25°C)	0.250		0.636 (0.462)	0.20		0.407	0.329		0.508 (0.388)	0.18		0.545
Absolute Viscosity (60°C, Poises)	1504	3858	13584 (11755)	1052	1876	4440	762	2524	5542 (3611)	992	2051	25104
Kinematic Viscosity (135°C, C.S.)	368	494	885 (853)	201	255	330	236	393	745 (616)	190	260	665
Flash Point (Closed Cup, °C)	252			266			229			265		
Loss on Heating (%)	0.34			0.66			0.20			0.50		

*RTFOT: After Rolling Thin Film Oven Test

**( ): Shoulder

Table 3.6. Asphalt Cement Data (Continued)

b. Chemical Composition (ASTM D-4124)

	3. Idylwood St.				5. Plainview Rd-Deschutes River				7. Arnold Ice Caves-Horse Ridge			
	0	2 days*	5 days**	R	0	2 days	5 days	R	0	2 days	5 days	R
Asphaltenes	23.70	27.20	29.01	37.25	16.13	22.35	24.27	25.26	24.90	25.60	27.83	28.00
Saturates	7.89	8.20	7.22	5.64	8.59	7.62	7.57	6.62	10.21	10.31	9.84	10.81
Napththene-Aromatics	21.83	22.98	23.37	24.70	26.81	25.81	25.08	25.78	25.46	27.40	26.78	19.70
Polar Aromatics	44.70	39.44	37.90	32.40	46.96	42.58	40.82	41.81	38.12	36.01	33.74	39.70
Total	98.12	97.82	97.50	99.99	98.49	98.36	97.74	99.47	98.59	99.33	98.19	98.21

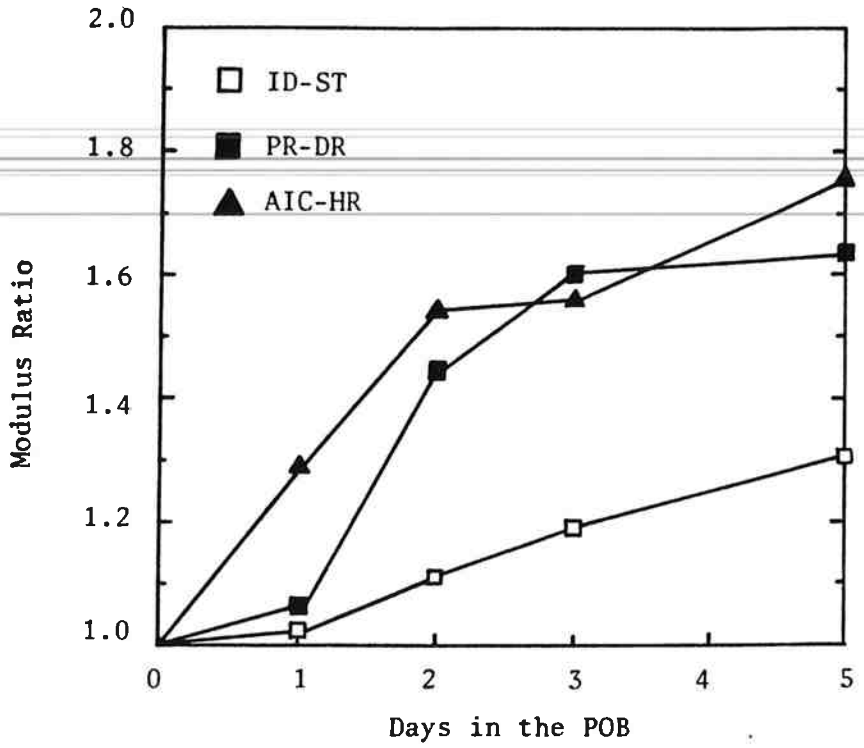
*aged with POB for 2 days

**aged with POB for 5 days

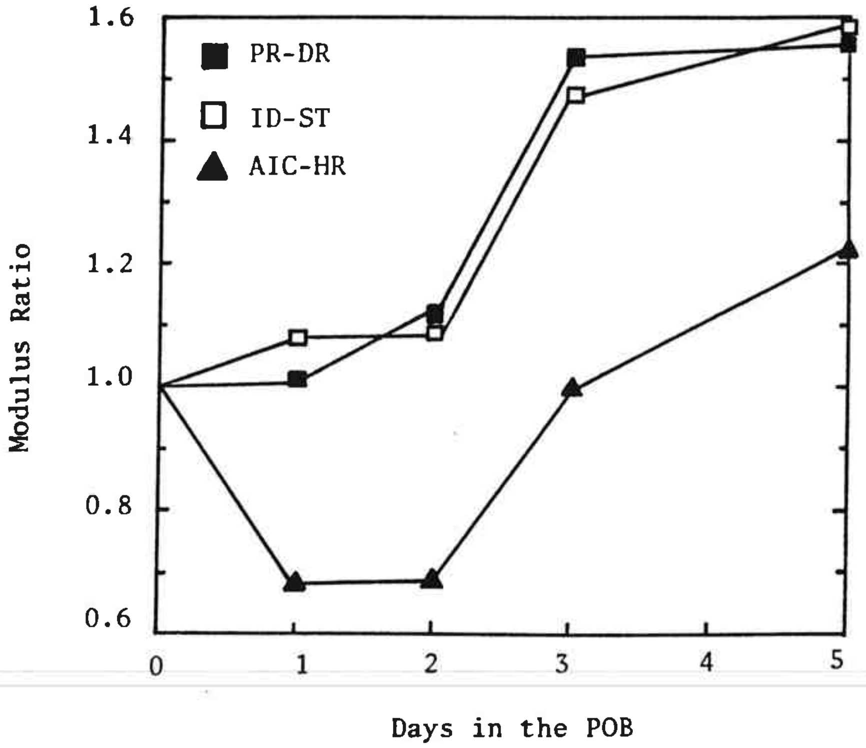
Table 3.7. Laboratory Mixture Aging Test Data.

Project	Days	M_r (ksi)	Ratio	N_f	Ratio	Bulk Sp. Gr.	Max. Sp. Gr.	Air Voids (%)
3. Idylwood St.	0	74.14	1.00	305	1.00	2.275	2.407	5.5
	1	80.17	1.081	309	1.013			
	2	80.96	1.092	444	1.456			
	3	109.96	1.483	958	3.141			
	5	117.61	1.586	1192	3.908			
	0	51.99	1.00	600	1.00	2.137		11.2
	1	53.44	1.028	752	1.253			
	2	58.36	1.122	1006	1.677			
	3	59.02	1.194	1319	2.198			
	5	68.02	1.308	1442	2.403			
5. Plainview Road-Deschutes River	0	237.37	1.00	711	1.00	2.292	2.455	6.6
	1	240.90	1.015	805	1.132			
	2	265.13	1.117	2033	2.859			
	3	365.63	1.540	3449	4.851			
	5	372.50	1.569	5081	7.146			
	0	148.87	1.00	N/A*	--	2.160		12.0
	1	157.92	1.061	2571				
	2	214.03	1.438	5404				
	3	237.83	1.598	6047				
	5	242.42	1.628	7643				
7. Arnold Ice Cave-Horse Ridge	0	105.08	1.00	N/A	--	2.349	2.447	4.0
	1	72.26	0.688	188				
	2	72.51	0.690	219				
	3	105.16	1.001	601				
	5	127.72	1.215	949				
	0	53.20	1.00	1137	1.00	2.202		10.0
	1	68.81	1.293	1181	1.039			
	2	81.86	1.539	1577	1.387			
	3	82.98	1.556	2031	1.786			
	5	93.50	1.758	2206	1.940			

Pure Oxygen Pressure = 100 psi
Aging Temperature = 60°C
N/A*: Not Available

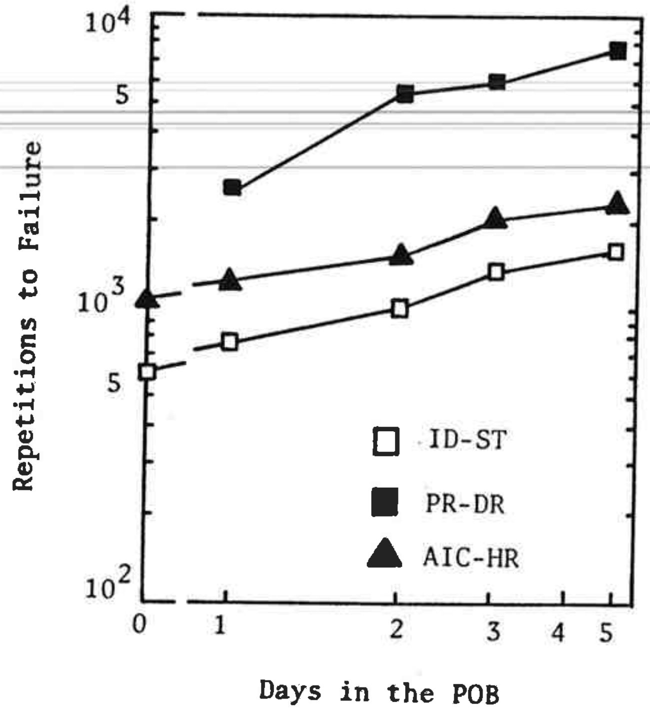


(a) At 88% Compaction Level

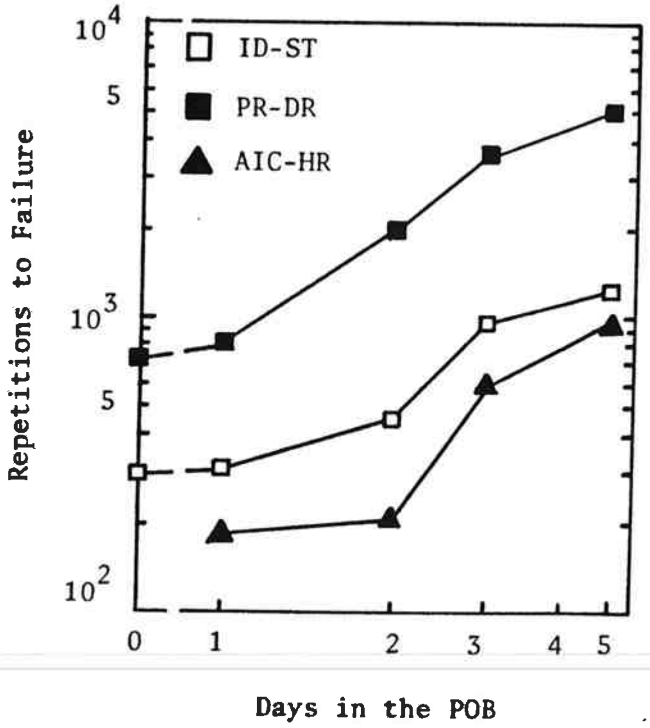


(b) At 94% Compaction Level

Figure 3.1. Aging Modulus Ratio.



(a) At 88% Compaction Level



(b) At 94% Compaction Level

Figure 3.2. Fatigue Life of Aged Specimens.

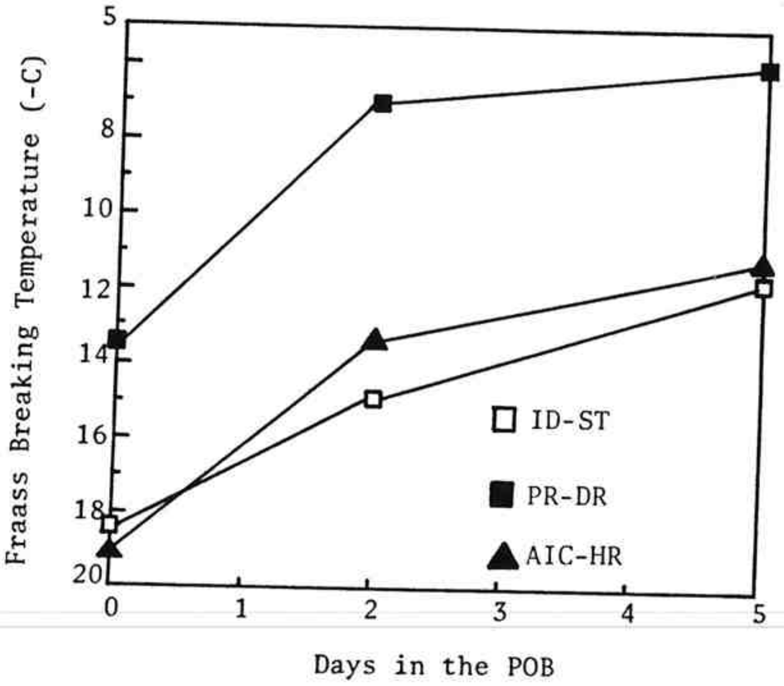
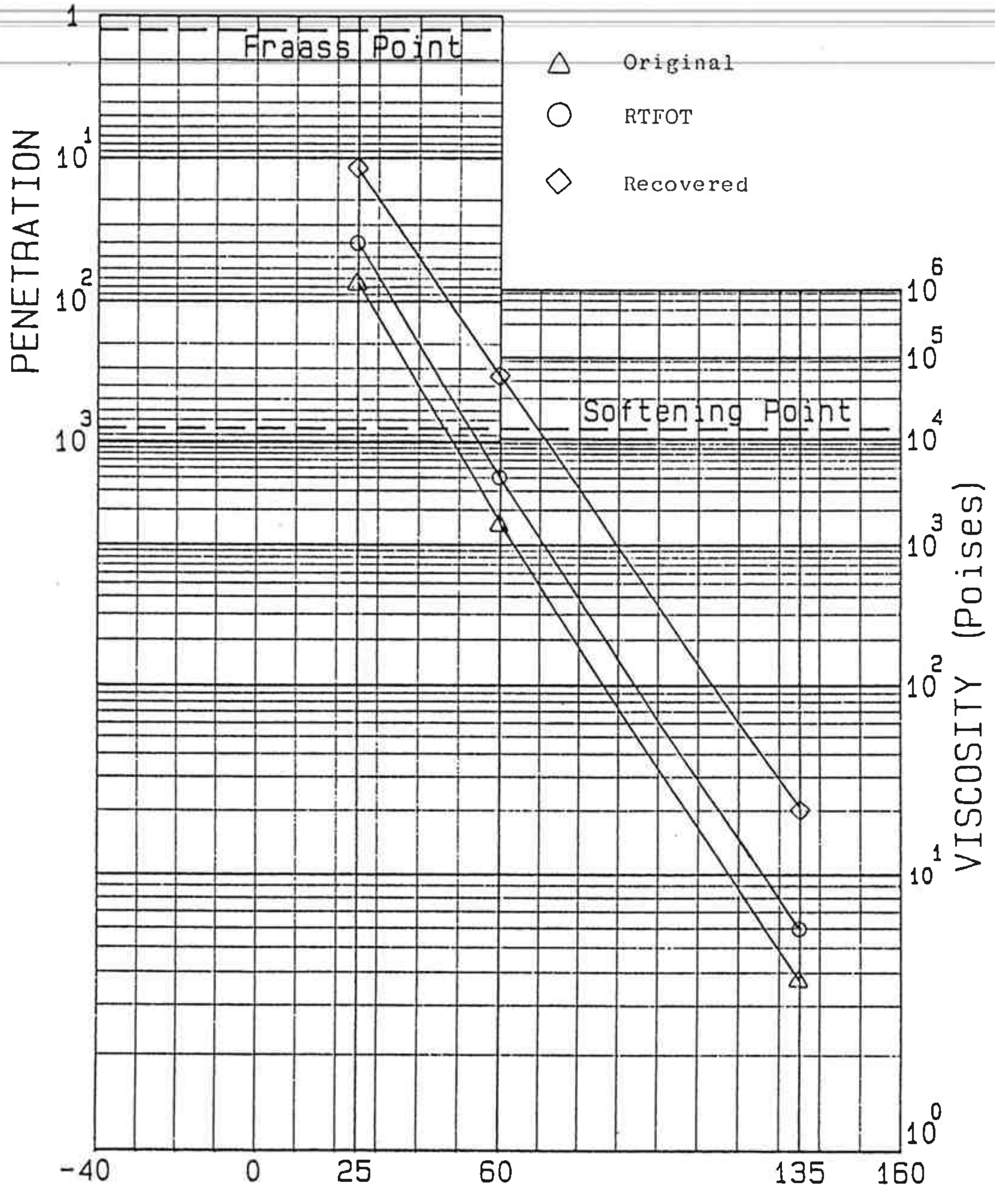


Figure 3.3. Effect of POB Aging on Fraass Temperature of Asphalt Cement.

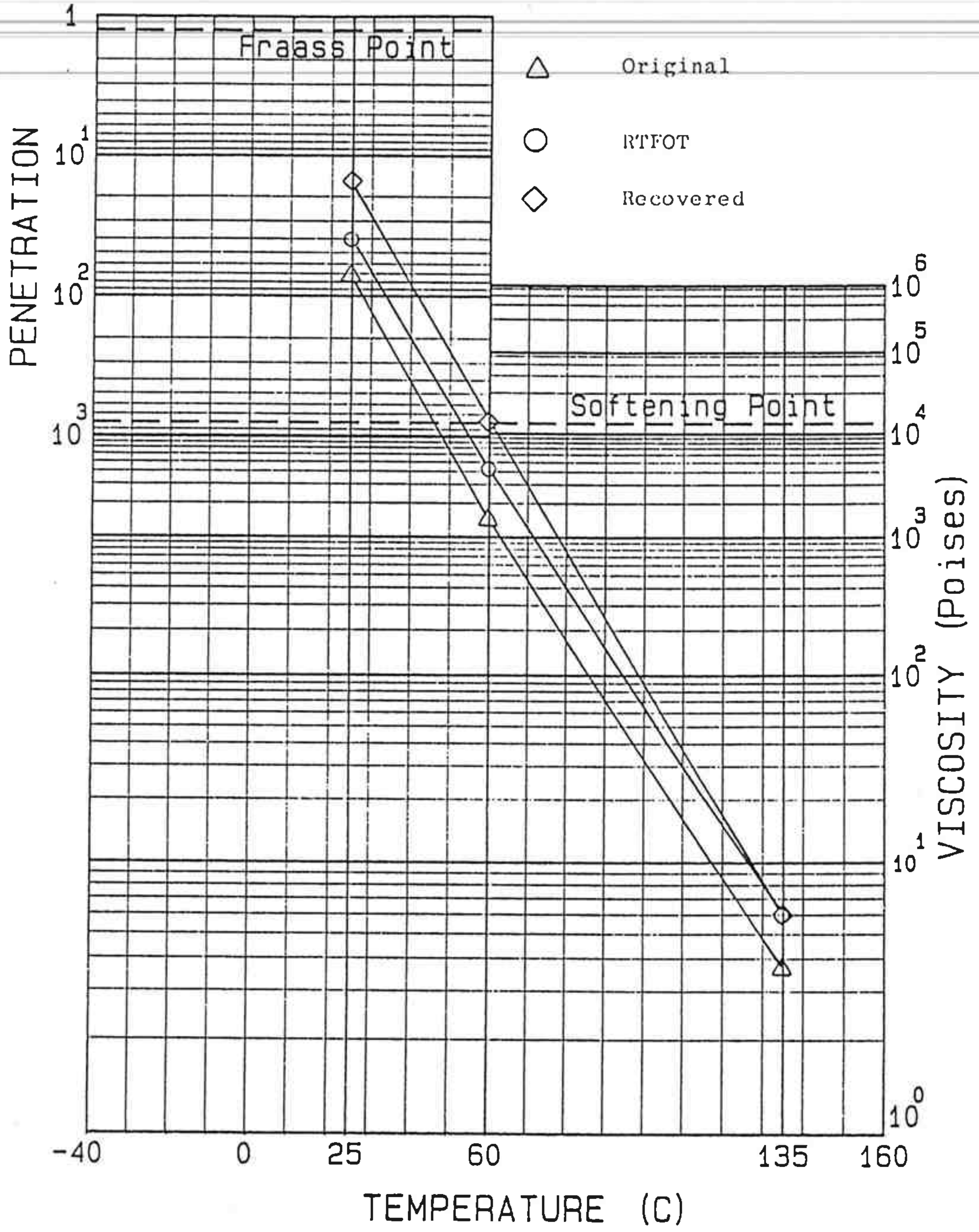
3.4 Summary of Asphalt Consistency Data

The consistency data given in Table 3.6a and in Figure 3.3 is summarized in Figures 3.4a through 3.4h. These show the Bitumen Test Data Chart with all available asphalt consistency data plotted for each project evaluated in this study.



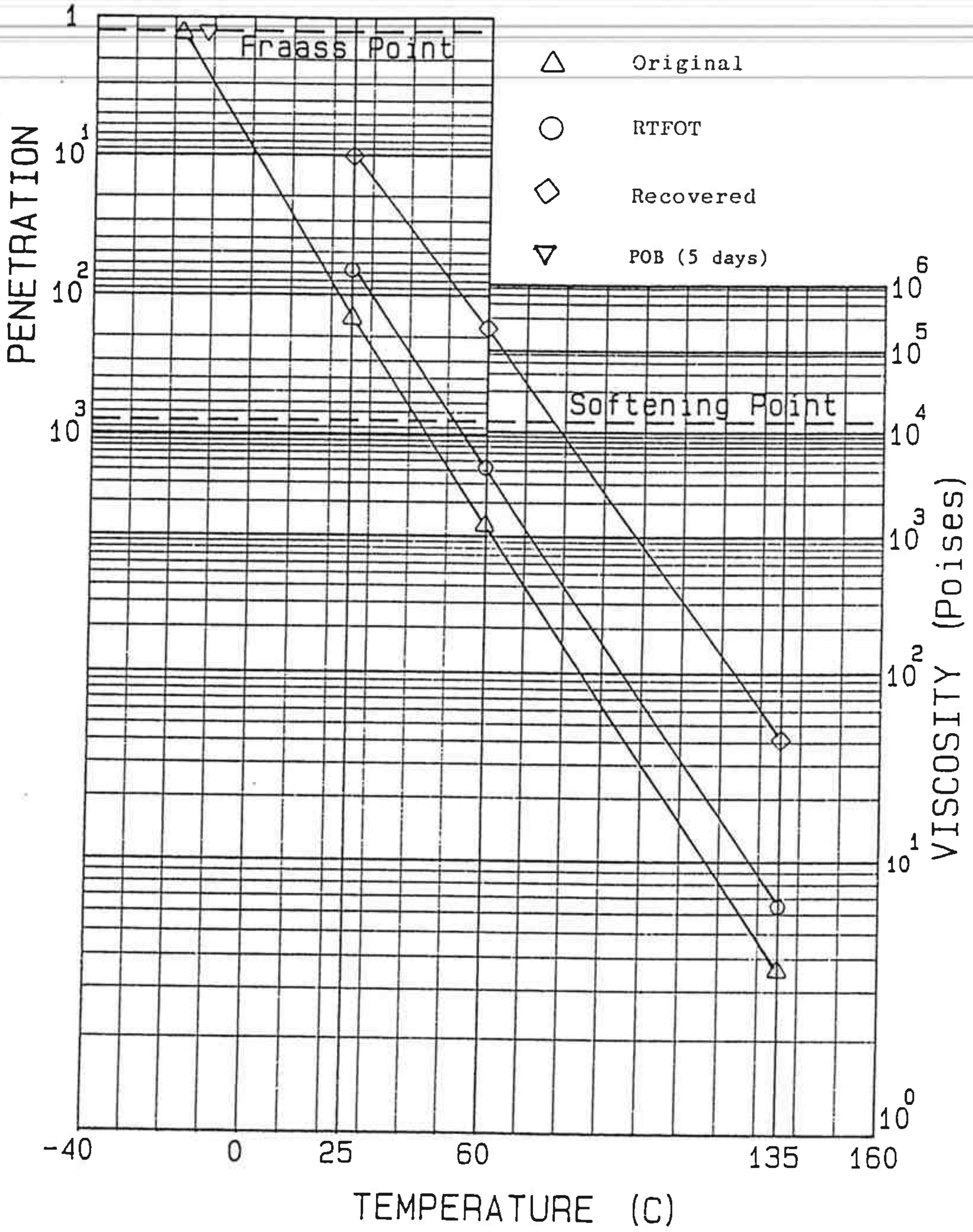
a)1. Grande Ronde-Wallace Bridge

Figure 3.4. Asphalt Consistency Data for Each Project.



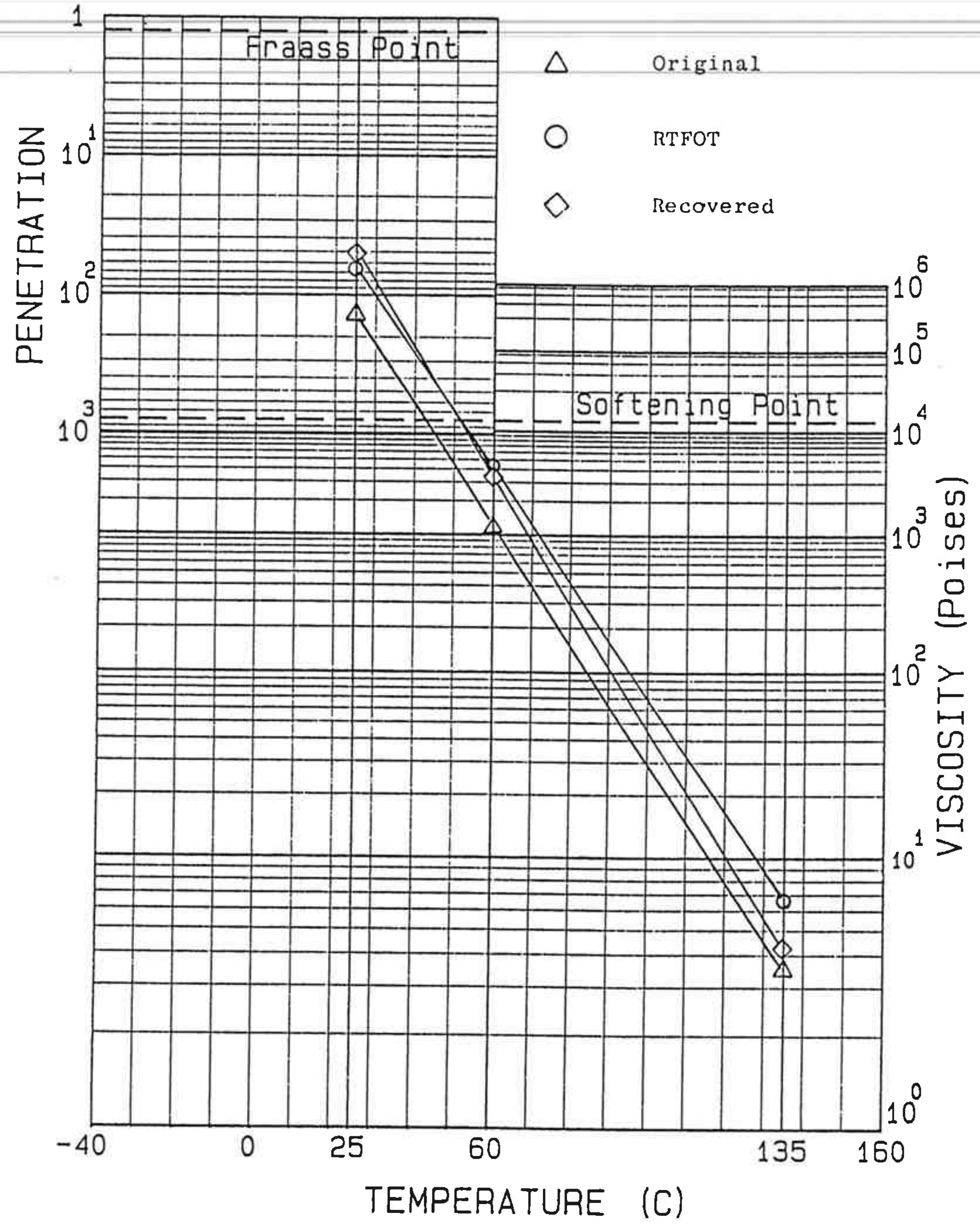
b)2. Pacific Highway

Figure 3.4. Asphalt Consistency Data for Each Project.



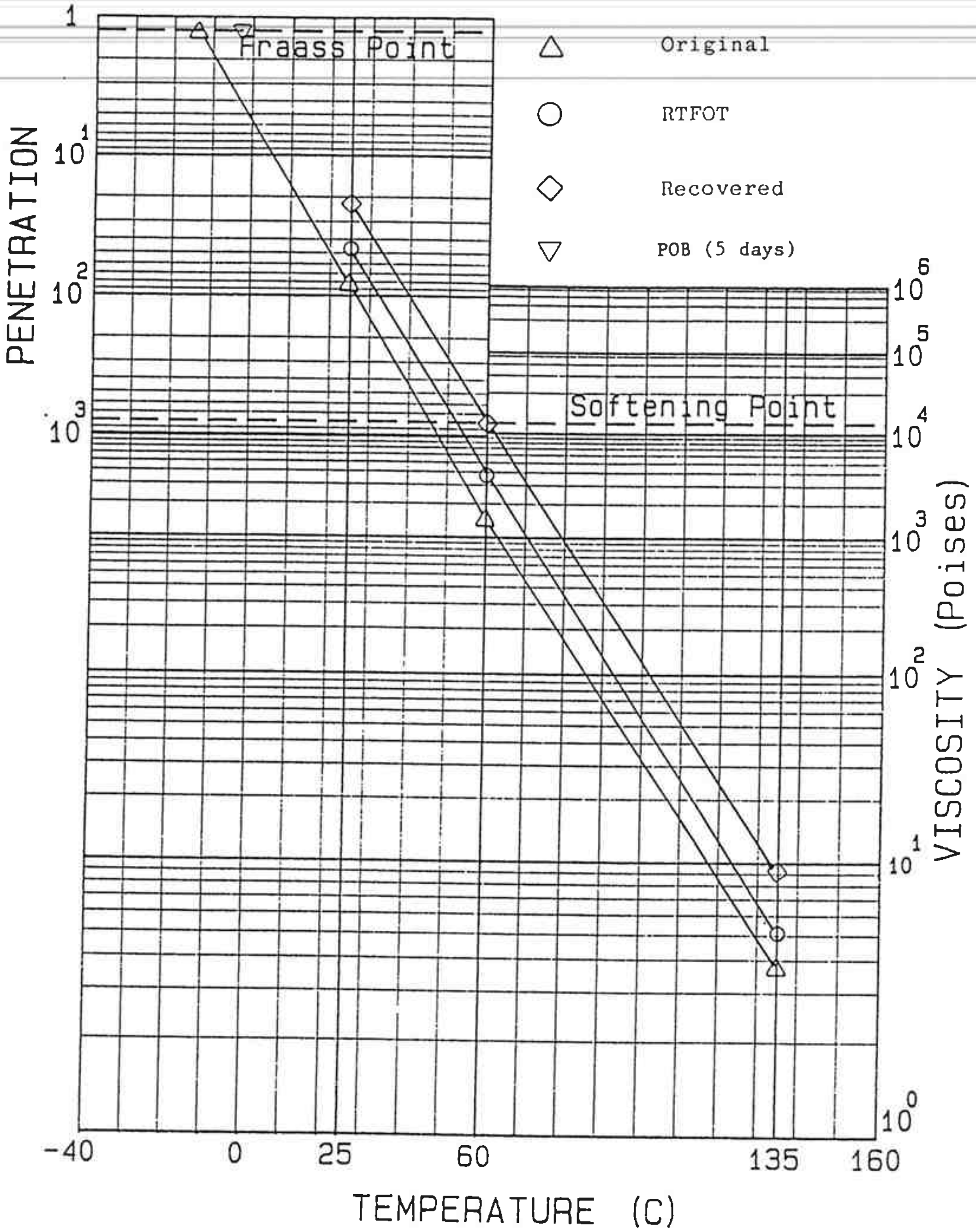
c)3. Idylwood St.

Figure 3.4. Asphalt Consistency Data for Each Project.



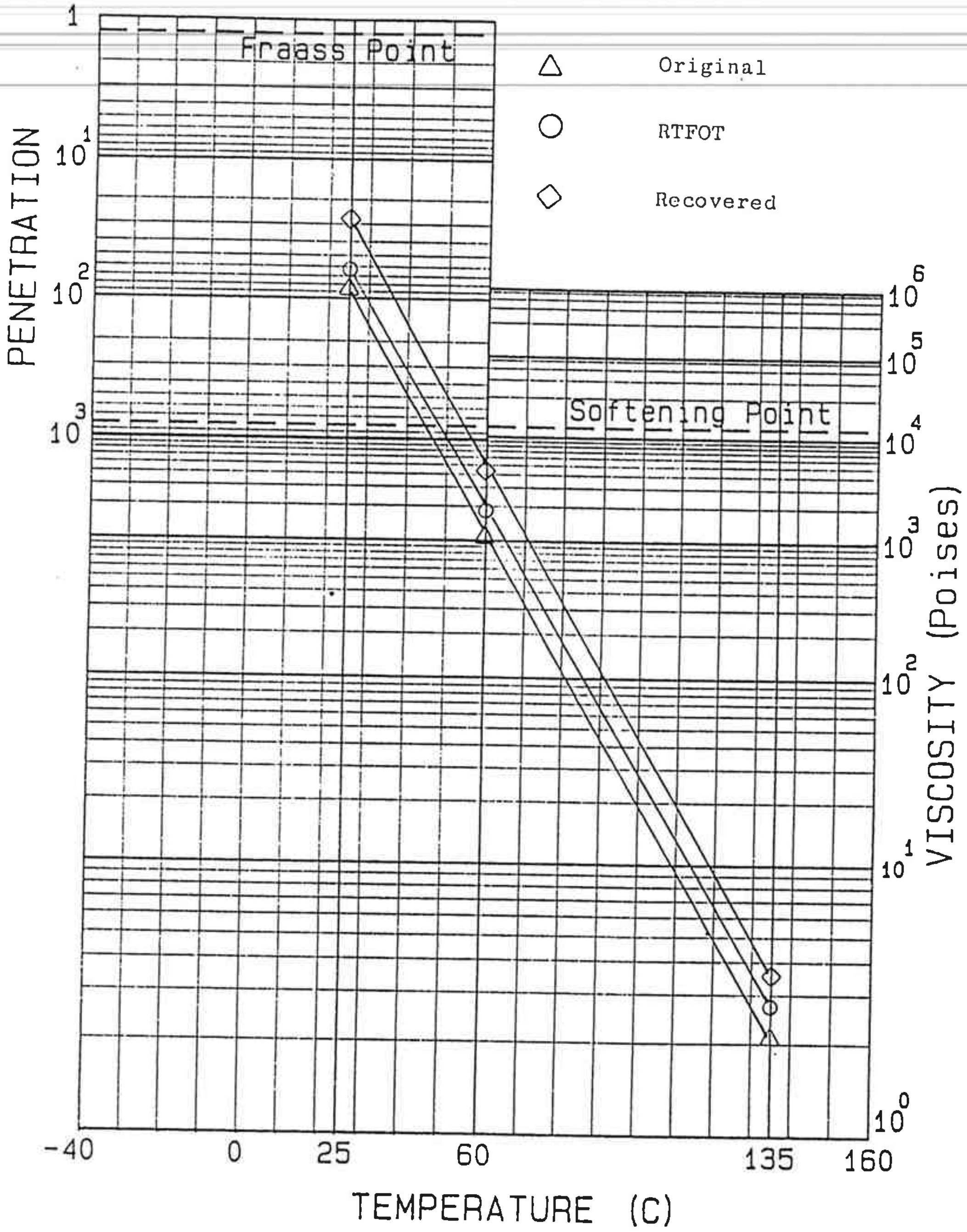
d)4. Elk River-Port Orford

Figure 3.4. Asphalt Consistency Data for Each Project.



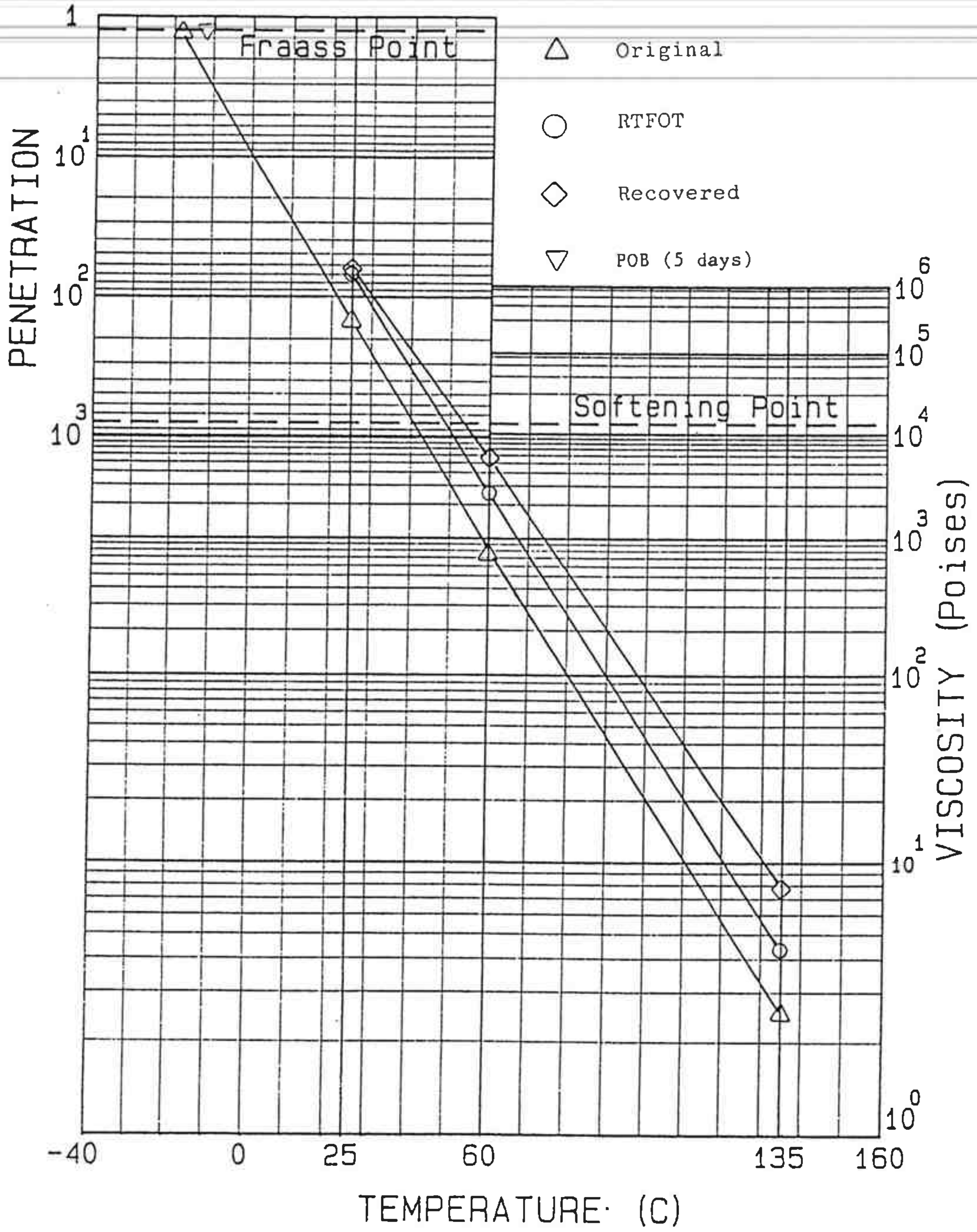
e)5. Plainview Rd.-Deschutes River

Figure 3.4. Asphalt Consistency Data for Each Project.



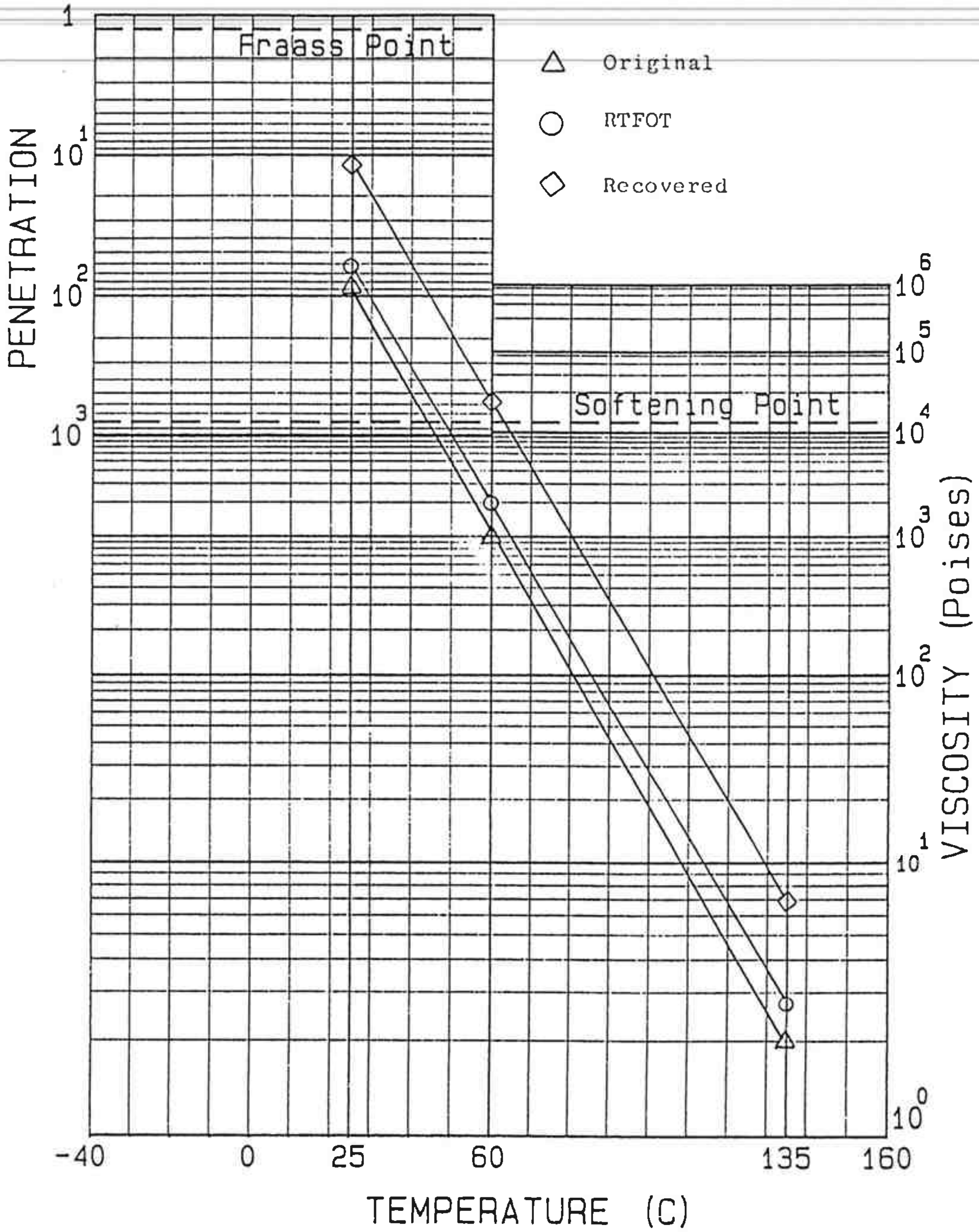
f)6. N. Klamath Falls Jct.

Figure 3.4. Asphalt Consistency Data for Each Project.



g)7. Arnold Ice Caves-Horse Ridge

Figure 3.4. Asphalt Consistency Data for Each Project.



h)8. S.F. Malheur R.

Figure 3.4. Asphalt Consistency Data for Each Project.

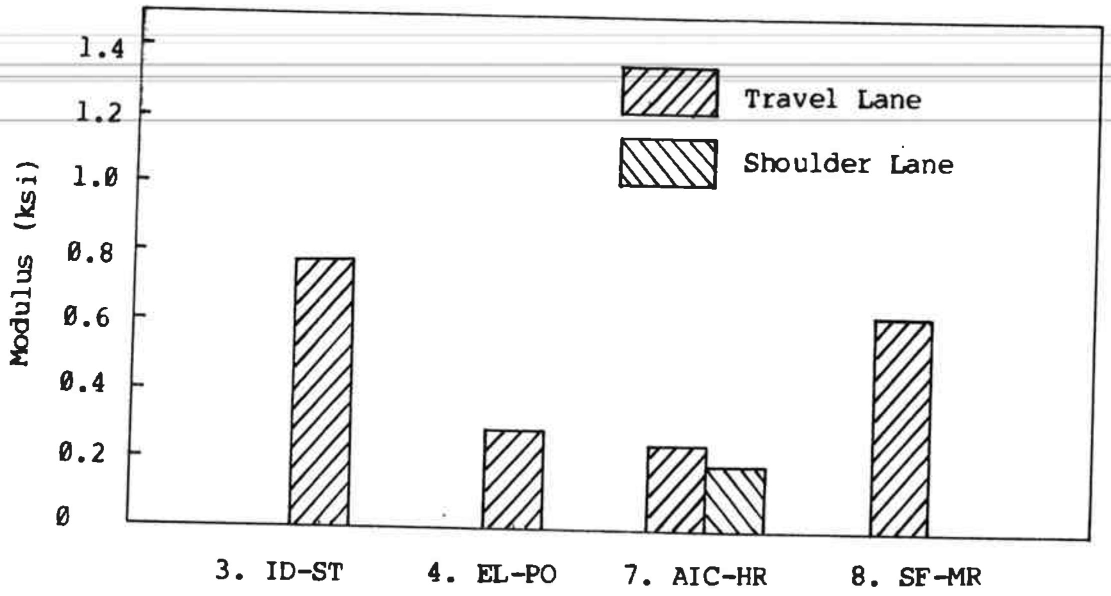
4.0 DISCUSSION OF RESULTS

The results are discussed in three major parts: 1) the cores, 2) the effects of laboratory aging on mixtures, and 3) the effects of laboratory aging on asphalt cement. The effect of laboratory aging is assessed by comparing properties measured with "as compacted (before aging)" and "aged samples (after aging)."

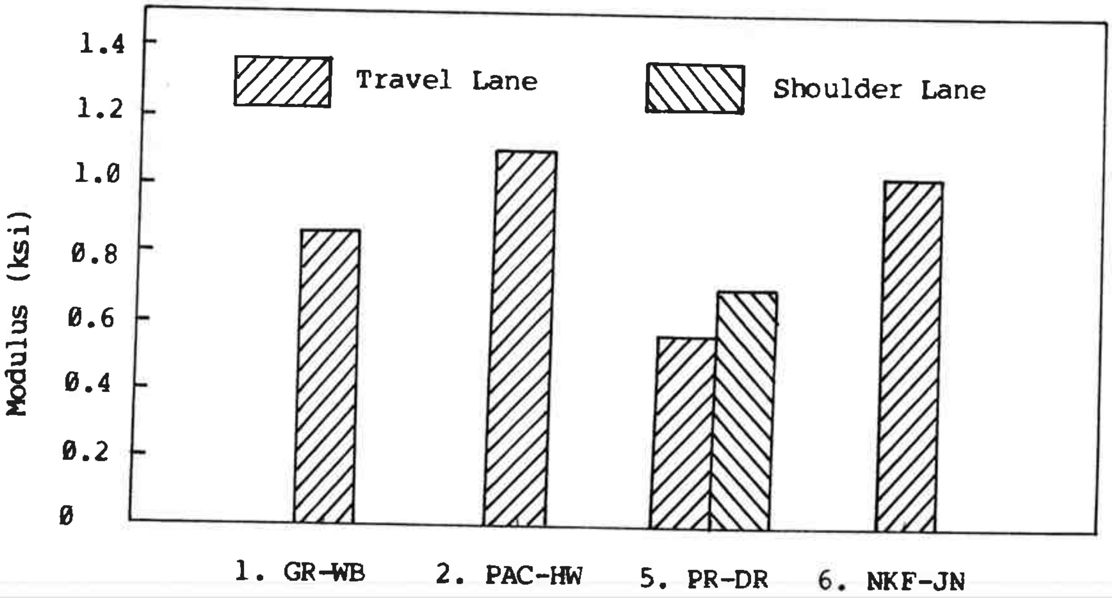
4.1 Cores

4.1.1 Resilient Modulus Results

The moduli values of cores for each project (with one standard deviation) are presented in Figure 4.1. In general, the moduli values of cores for five-year-old projects, which ranged from 560 ksi (3861 MPa) to 1100 ksi (7585 MPa), are much higher than those for 10-year-old projects which ranged from 180 ksi (1241 MPa) to 700 ksi (4827 MPa). This result is partially explained by considering the grade of asphalt cement used and asphalt cement contents. For the ten-year-old projects, the grades of asphalt cement used were 120/150 pen and AR-2000 for the Arnold Ice Caves-Horse Ridge (Project #7) and the S.F. Malheur River project (Project #8), respectively. For the five-year-old projects, however, AR-4000 (or AR-4000W) or 85/100 pen asphalts were used. An additional reason for the difference between the two aging groups is the asphalt cement content of each project. For the ten-year-old projects the asphalt cement contents ranged from 5.9% to 7.6%. The asphalt cement contents for the five-year-old projects ranged from 5.2% to 5.5%. The effect of compaction due to the traffic while in service on resilient modulus of pavement seems not to be significant between the five-year-old pavements and the ten-year-old pavements.



(a) Ten-Year-Old Projects



(b) Five-Year-Old Projects

Figure 4.1. Modulus of Cores.

For the Arnold Ice Caves-Horse Ridge project (Project #7) the modulus of cores from the travel lane is greater than those from the shoulder as expected. For the Plainview-Deschutes River project (Project #5), however, shoulder cores gave higher modulus than travel lane cores, even though shoulder cores had lower bulk specific gravity than travel lane cores.

Among the ten-year-old projects, the modulus of the Elk River-Port Orford project (Project #4) is much lower than that of the S.F. Malheur project (Project #8) and is almost the same as that of the Arnold Ice Caves-Horse Ridge project (Project #7) while the asphalt cement used for the Elk River-Port Orford project is stiffer grade (AR-4000) than that for the Arnold Ice Cave-Horse Ridge project (120/150 Pen.) or S.F. Malheur project (AR-2000). This result can be explained by the physical properties of the asphalt cement presented in Table 3.6. The data from the recovered asphalt cement show that the asphalt cement used for the Elk River-Port Orford project is softer than that for the S.F. Malheur project. Another possible reason why the cores of the S.F. Malheur project (Project #8) obtain relatively high average modulus is that the aggregate used for the project absorbs as much as 0.6% of the asphalt cement, as presented in Table 3.2. The nonabsorbed components of asphalt cement tend to be the heavier components and tend to have a higher viscosity than the original asphalt and cause a high mix modulus.

Even though the asphalt cments used for the Idylwood St. (Project #3) project and the Elk River-Port Orford (Project #4) project have almost identical original properties, as presented in Table 3.6, they have different sources. The properties of the recovered asphalt cement and the moduli values of two projects are substantially different (Table 3.6 and Figure 4.1). The average modulus of the cores for the Idylwood St. project is much greater than that for the Elk River-Port Orford project, probably because the recovered asphalt cement of the former project (Project #3) is hardened more than that of the later project (Project #4). However, cores from the Idylwood St. project show the highest current air voids among the ten-year-old projects. This result may be explained by the relation between the aging rate (or the degree of aging) of the asphalt cement and the air voids content of the mixture. With higher air voids, an asphalt cement is more susceptible to hardening than with lower air voids.

Of the five-year-old projects, both the Pacific Highway project (Project #2) and the Grande Ronde project (Project #1) used the same asphalt cement (Chevron AR-4000W). However, after five years of service the modulus of the Pacific Highway project having higher bulk specific gravity (lower air voids) is much greater than that of the Grande Ronde project even though the physical property data (Table 3.6) indicate that the recovered asphalt cement for the Grande Ronde project is stiffer than that for the Pacific Highway project as explained above (i.e., higher air voids, more hardened).

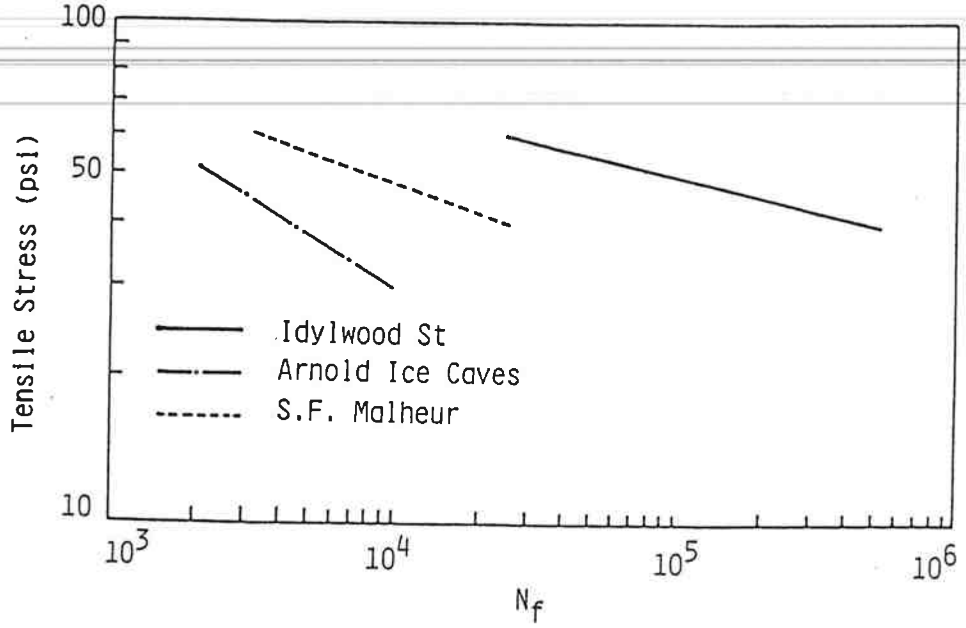
In summary, these results show that themodulus of mixtures relates strongly to the asphalt cement contents, the physical properties of asphalt cement, and the air voids of mixtures.

4.1.2 Fatigue Results

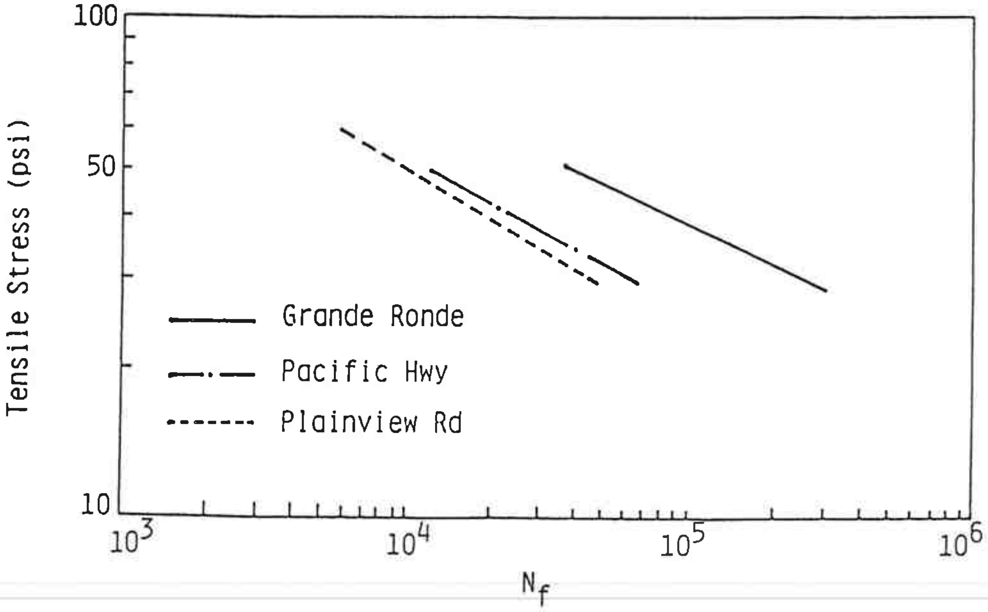
The fatigue lives of the cores from each project in terms of K, m (2.3) or C, n (2.4) are presented in Table 3.5 except the Elk River-Port Orford project (Project #4). Since the top surface of cores was stiffer than the bottom due to more aging, as reported earlier (40), it was very difficult to run the repeated load test to measure the fatigue lives of cores. During the fatigue test, the cores were not deformed evenly, that is, the surface part, which was stiffer than the bottom part due to the effect of age hardening, was not deformed at all while the bottom part deformed gradually. This phenomenon was severe for the Elk River-Port Orford project.

Due to the different moduli and heights of the cores among the projects the fatigue test was run on the basis of the initial tensile stress obtained from Eq. (2.7) rather than the initial tensile strain. Of six projects, the Idylwood Street project (Project #3 - 10 years old) shows the longest fatigue life in the range of tensile stress applied (Figure 4.2). This project used AR-4000 asphalt cement and a B-mix as presented in Table 3.2. The bulk specific gravity is low and asphalt cement content is 5.9%. The modulus of the Idylwood Street project is the highest of the 10-year-old projects and similar to those of the five-year-old projects (Figure 4.1). However, the Idylwood Street project keeps the longest fatigue life as demonstrated in Figure 4.2. The reason of this result can be explained by the properties of asphalt cement used. Even though the grade of asphalt cement for the Idylwood Street project is similar to that of the other projects (AR-4000, AR-4000W, or 85/100 pen.) the aging rate is less as shown in Figures 3.1 and 3.3. Also, this asphalt cement contained relatively more volatile components (heating loss: 1.77% in Table 3.6). The property data (Table 3.6) shows that original asphalt cement for the Idylwood Street project is much softer than that for the other projects to use the same grade asphalt cement but the recovered properties are harder.

From the fatigue test results, the asphalt cement used for the Idylwood Street project (Project #3) might have the best cohesion/adhesion property. Of the ten-year-old projects, the S.F. Malheur project (Project #8) shows better fatigue life performance than the Arnold Ice Caves project (Project



(a) Ten-Year-Old Projects



(b) Five-Year-Old Projects

Figure 4.2. Fatigue Life of Cores.

#7). The moduli (Figure 4.1) and the fatigue life for the S.F. Malheur project is much higher. This result is due to the different mix design (C mix for S.F. Malheur project and B mix for Arnold Ice Caves project) and different asphalt cement content (7.6% and 6.7%) even though they used the almost same asphalt cement grade. Unfortunately, the fatigue life result of the Elk River-Port Orford project was not obtained due to problems with testing thin aged cores, as explained above.

Of the six projects represented in Figure 4.2 the Arnold Ice Caves-Horse Ridge project shows the poorest results (lowest modulus and shortest fatigue life). This might be due in part to the asphalt cement properties.

Of the five-year-old projects, the Grande Ronde project (Project #1) shows the longest fatigue life (Figure 4.2). Even though the Grande Ronde and Pacific Highway projects (Project #2) use the same asphalt cement grade, the fatigue life of the Grande Ronde project is much longer than that of the Pacific Highway project while the modulus of Grande Ronde project is lower than that of the Pacific Highway project. The current higher air voids of cores from the Grande Ronde project produces higher stiffness in the asphalt cement (Table 3.6), as discussed in the previous section. The shorter fatigue life of the Pacific Highway project may be caused by the differences in the asphalt properties or may be due to differences in the aggregate gradation (i.e. passing No. 200) and aggregate source. The Plainview project (Project #5) shows the shortest fatigue life among the five-year-old projects and the modulus of this project is the lowest.

In summary, for a longer fatigue life of the pavement, mixtures should obtain high modulus from good mix design (i.e. with low voids and high asphalt content) and construction practices, and also the properties of the asphalt cement have an important role.

Of the mechanical properties of asphalt mixtures the fatigue life behavior might be the most rational indicator of good mix characteristics. This is because the fatigue life of a mixture is a function of not only modulus of a mixture but also of the adhesion of asphalt cement over time.

4.2 Laboratory Aging of Mixtures

The mixtures fabricated in the laboratory using the kneading compactor were artificially aged with pure oxygen in the POB as described in Chapter 2 for periods of 1, 2, 3, and 5 days. After aging the mixtures were cooled for one day at room temperature before the modulus and fatigue tests were run.

4.2.1 Resilient Modulus Results

In order to measure the degree of mixture aging, the modulus ratio (the ratio of the modulus after aging to that before aging) for each time period was calculated. The values obtained are shown in Figure 3.1.

The mixtures for the Idylwood Street project (Project #3) with 12% air voids aged at a constant rate, while the mixtures for the other two projects aged very rapidly during the first two or three days. The aging rate (slope in Figure 3.1) of the mixtures for the Plainview Road-Deschutes River project (Project #5) changed little after the third day while that of the Arnold Ice Caves-Horse Ridge project (Project #7) still increased. Even though a similar asphalt cement grade (AR-4000 or AR-4000W) was used for both the Idylwood Street and the Plainview Road-Deschutes River project the trend of mixture aging is significantly different. This difference is probably because the physical properties of the original and recovered asphalt cement were substantially different (Table 3.6).

The results from the mixtures with 6% air voids show a slightly different trend. Unlike the mixtures with 12% air voids, the mixtures with 6% air voids for the Idylwood Street and Plainview Road-Deschutes River projects were aged little during the first two days and then aged rapidly between the second and third days. This rapid aging was followed by a period of slow aging between the third and fifth days. This result may show that it takes some time for the oxygen to penetrate into mixtures with low air voids. If so, the permeability of a mixture is an important factor in the aging rate as suggested by Goode (41) and Kumar (42). It is noted that the aging rate with 6% air voids for both the Idylwood Street project and the Plainview Road-Deschutes River project was very similar while the aging rates of the mixtures with 12% air voids for these projects are significantly different.

One unexpected result for the mixtures with 6% air voids was that the samples for the Arnold Ice Caves-Horse Ridge project appeared to soften during the first 3 days. This may have been caused by a slight loss of cohesion of the sample at the high temperature used in the POB (60°C). However, the aging rate (slope) after the second day is similar to that with 12% air voids.

Finally it should be noted that the modulus results from the laboratory accelerated aging procedure (POB) performed for five days under 100 psi (688.5 kPa) and 60°C (140°F) were not comparable with the modulus results from the cores of five or ten years old. Figure 4.3 shows the moduli of the laboratory aged samples for both air void levels after 5 days, and the core modulus values. As can be seen, the core modulus values are approximately twice those of the low air void content laboratory samples.

The change of modulus ratio through each time interval described above is related to the aging trend of the asphalt cement as well as to the air voids

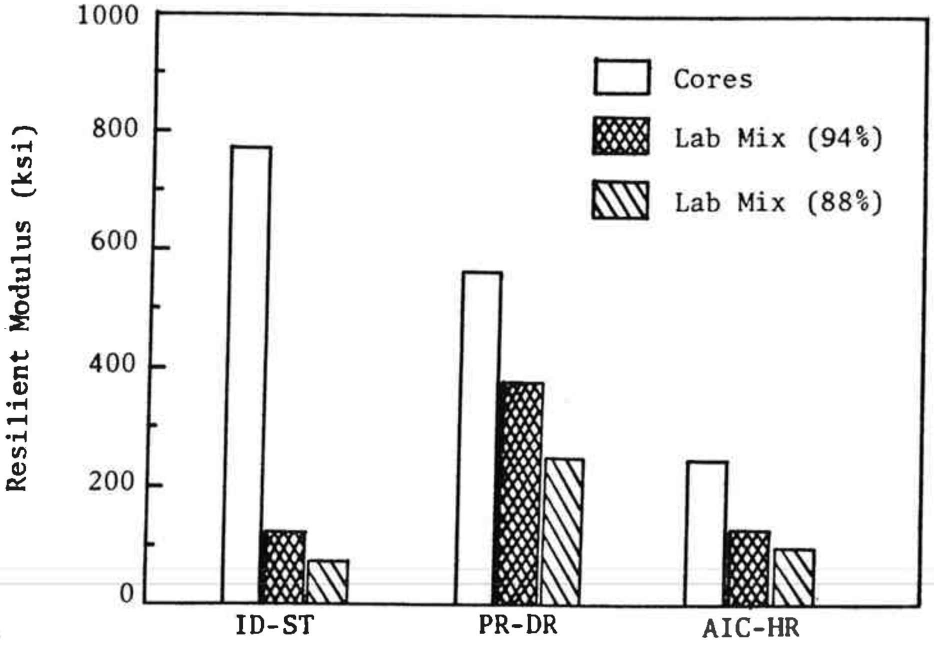


Figure 4.3. Comparison of Moduli Between Cores and Aged Mixtures.

(permeability of the mixture) and asphalt film thickness. This will be discussed further in Section 4.3.

4.2.2 Fatigue Results

After the modulus test, the fatigue test was run with applying 20 psi (137.9 kPa) and 40 psi (275.8 kPa) of tensile stress for 12% air voids mixtures and 6% air voids mixtures respectively.

Fatigue results shown as a function of applied stress best represent the effect of the differences in modulus shown in Table 3.7 and result in a wide variety of fatigue performance as shown in Figure 3.2. The fatigue results show that the aged mixtures of the Plainview Road-Deschutes River project (Project #5) obtain the longest fatigue life for both levels of air voids, since this project has the highest moduli values as presented in Table 3.7.

In general, the resistance to fatigue failure (slope in Figure 3.2) of mixtures decreases after the third day regardless of the difference of compaction levels.

The fatigue characteristic of mixtures with different air voids was changed slightly through the aging time. For 12% air voids, the slopes of each project are relatively constant through time, while slopes of the mixtures with 6% air voids changed in each time interval. These results show that the air voids of mixtures affect the resistance to fatigue failure through their service period, and that high modulus is an important factor in achieving long fatigue life. Also, the change of fatigue life based on the stress level between each time interval is similar to the change of modulus ratio between the corresponding time interval. The changes of fatigue life of the Idylwood Street and the Arnold Ice Caves-Horse Ridge projects increase slightly during the first two days. This result can be explained by the time for oxygen to penetrate into the mixture as discussed in the previous section, since the actual air voids of Idylwood Street (5.5%) and Arnold Ice Caves-Horse Ridge project (4.0) are much lower than that of Plainview Road-Deschutes River project (6.6%).

In this study the mixtures were evaluated at fixed initial stress levels. This approach was discussed at length in Reference 4 that different mixtures are compared at a standard loading condition but means that high modulus materials will suffer a lower applied strain. Therefore, unless they are considerably inferior in some other regard (e.g. asphalt adhesion), they will have longer fatigue lives.

4.3 Fraass Breaking Temperature

The trends of increased Fraass breaking temperature of each project (Figure 3.3) are similar to those of increased modulus ratio of mixtures with 12% air voids (Figure 3.1). For the Idylwood Street project (Project #3) the breaking temperature of the asphalt cement (AR-4000) increases very slow as the aging period increases. For the Plainview Road-Deschutes River project (Project #5) which used the same grade of asphalt cement (AR-4000) the breaking temperature of asphalt cement increases much more rapidly during the first two days aging. The original asphalt cement (AR-2000) used for the Arnold Ice Caves-Horse Ridge project (Project #7) has almost the same breaking temperature and a similar aging rate as that for the Idylwood Street project. Also, these asphalt cements have almost the same physical properties except for flash point and loss on heating as presented in Table 3.6. Again, it can be seen that even though asphalt cements may have the same grade, they show significant differences in behavior such as for the AR-4000 asphalts used for the Idylwood Street and Plainview Road-Deschutes River projects.

The change in Fraass breaking temperature of an asphalt cement indicates a general change in the consistency properties of the asphalt cement which can be estimated from a Bituminous Test Data Chart (BTDC, (23)) as shown in Figures 3.4 (c, e and f) for each project. If the temperature susceptibility of asphalt cement changed little after aging (i.e. if the lines drawn through the original and aged property data were parallel), it might be possible to predict the long term asphalt properties with the one point of Fraass breaking temperature of asphalt aged in the POB. Hence, the Fraass breaking temperature may be a valuable indicator of the durability of asphalt cement as well as defining a low temperature consistency.

The rolling thin film oven test (RTFOT, ASTM D-2872) is used to measure the anticipated hardening of the asphalt during hot-mix plant operations in several western states, and therefore will probably not give an indication of hardening due to long term aging. The consistency data for asphalt from each project after RTFOT (Figure 3.4) are not adequate to predict the long term asphalt properties due to oxidation. The data shown in Figure 3.4 illustrate that asphalt recovered from field cores had higher consistencies to RTFOT aged asphalt, and that POB aging (5 days) may be similar to field aged materials with regard to the Fraass point. Clearly more data is required to support this suggestion.

For the Idylwood Street project the Fraass breaking temperature lies between the projected line (dashed line in Figure 3.4 of RTFOT and recovered as shown in Figure 3.4 (c), that is, the asphalt cement aged in the POB for 5 days was aged more than the asphalt from the RTFOT and aged less than that of cores. However, for the Plainview Road-Deschutes River project the Fraass breaking temperature of asphalt aged in the POB for 5 days lies close on the projected line of recovered asphalt in Figure 3.4 (e). The result of the Arnold Ice Caves-Horse Ridge project is similar to that of the Plainview Road-Deschutes River project as shown in Figure 3.4 (f).

4.4 Effectiveness of POB

In general, a major cause of asphalt cement hardening is oxidation, a process that occurs most readily at high temperature and with thin asphalt films. The POB developed in England (5) was modified to age both asphalt cements and asphalt mixtures for the aging study at OSU. The modified POB was used with 100 psi and 60 C. Previously, higher pressures and temperatures were used (5,34), but the lower levels were adopted because of safety considerations and preservation of the shape of the mixture samples.

The modulus ratios obtained from original mixtures and weathered mixtures after four years aging at four different weathering sites in California (32) are presented in Table 4.1. The modulus ratio ranges from 0.67 to 1.67, excluding the high air voids (7 to 12%) mixture using Santa Maria asphalt and non-absorbent aggregate. Even though the moduli values of aged laboratory mixtures were substantially different from those of cores tested in the study reported herein, it can be seen that the POB causes similar changes in modulus, i.e., results in similar modulus ratios (Figure 3.1) to those observed in the California study (Table 4.1).

The results of asphalt cement chemical composition tests (43) done in cooperation with this aging study, indicate that the use of the POB to age the asphalt cement on Fraass plaques (0.5 mm film thickness) produced a similar chemical composition to that of asphalt extracted from the cores. The results presented in Table 3.6 show that asphalt cement aged in the POB for 5 days can have a similar composition to that from the cores. However, as with the

Table 4.1. Modulus Ratio of Field Weathering Mixtures
(original and 4 year field weathering, Ref. 10)

A. Non Absorbent Aggregate

Asphalt Source	Air Voids	Original*	Weathering Site** A	B	C	D
	3-5	950	0.86	0.76	1.16	0.81
Valley	7-9	880	0.85	0.84	1.05	0.82
	10-12	1000	0.73	0.76	0.81	0.62
LA Basin	7-9	740	0.78	0.73	1.11	0.96
	3-5	330	1.21	1.21	1.24	1.67
Santa Maria	7-9	160	2.31	2.13	2.44	3.13
	10-12	150	1.93	2.47	2.13	3.40

B. Absorbent Aggregate

Asphalt Source	Air Voids	Original*	Weathering Site A	B	C	D
	3-5	810	0.79	0.80	1.12	0.86
Valley	7-9	590	1.09	0.83	1.20	1.07
	10-12	730	0.92	0.70	1.11	0.95
Santa Maria	3-5	430	1.28	1.00	1.28	1.61
	10-12	270	1.37	1.44	1.63	2.07

Original*: Resilient Modulus of Original Mixture, ksi
Weathering Site**: A: Fort Bragg B: Sacramento
C: So. Lake Tahoe D: Indio

consistency data, there is a significant difference in the POB and recovered properties for the Idylwood Street project.

The component fractions of asphalt cement samples (aged for 5 days) from the Plainview Road-Deschutes River project were very similar to those obtained for asphalt extracted from the cores. Also, for the Arnold Ice Caves-Horse Ridge project the fractions of asphaltenes and saturates of the asphalt cement aged for 5 days in the POB are close to those of the cores. The chemical composition of the aged asphalt (for 5 days) of the Idylwood Street project shows that the asphalt in the POB was less aged than that of cores as discussed in the previous section. The equivalent aging period in the laboratory to a field may vary with grade and source of asphalt cements as well as mixture properties (particularly air voids and asphalt film thickness) and environmental conditions.

The POB can be used effectively to age asphalt cements in thin films with high pressure and/or temperature to give an indication of the long term asphalt properties with one point of Fraass breaking temperature on the BTDC. However, to effectively evaluate mixture aging, representative mixtures must be tested, and if possible these should be core samples obtained shortly after construction, rather than laboratory compacted mixtures which as shown in this study and others do not represent field mixtures. There may be a large difference between the laboratory mixtures and plant mixtures due to the differences in production method and differences in the compaction method.

Reference 5 indicates that one day's aging of asphalts in the POB at a pressure of 20 atmospheres (300 psi) and at temperatures of 50 to 60 C is equivalent to half a year on the road in Holland. However, it is extremely important to fit an appropriate pressure relief device to the POB when working at these elevated levels. In this study only 100 psi at 60 C was applied after consideration of safety concerns regarding the use of pressurized oxygen and in order to preserve the shape of the mixture samples.

4.5 Summary

For the limited number of projects studied, the pressure oxidation bomb (POB) was found to be a very effective device for oxidative aging of both asphalt cements and asphalt mixtures. The test results show that with the exception of one project, and in that case only for modulus, all the properties changed by at least 50 percent after 5 days of POB aging. Higher pressure, temperature and/or longer exposure time could be applied to accelerate the aging, but the user must be aware of safe operating procedures.

The modulus ratio and the Fraass breaking point are good indicators to measure the aging rate of mixtures and asphalt cement, respectively. The aging rate of mixtures varies with the air voids of mixtures. The mixtures with higher air voids are aged more rapidly, although the aging rate depends on asphalt properties.

The limited testing done in this study showed that asphalts aged in the POB for 5 days had similar composition to those recovered from field cores of five to ten years old. This lends some confidence to the use of the Fraass samples for oxidative aging procedures.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The effects of aging of asphalt mixtures and asphalt cements used in the construction of three projects in Oregon were evaluated. Similarly, cores from eight projects (four of five years old and four of ten years old) in Oregon were evaluated. The repeated load diametral test and a Pressure Oxygen Bomb (POB) laboratory accelerated aging method with pure oxygen were used.

The resilient moduli and fatigue lives of cores and laboratory mixtures as well as the physical properties of asphalt cements (including the Fraass brittle temperature) were determined. For the laboratory mixtures the following range of variables were considered:

1) Compaction level: 88%, 94%
2) Aging period: 0, 1, 2, 3, and 5 days.

The following major conclusions are drawn from the findings of this study:

1) The Idylwood Street project shows the best fatigue life characteristics and slowest rate of aging.
2) Even though asphalt cement is specified as the same grade, there can be a significant difference in the performance or durability of mixtures using the same grade as shown in this study with projects using AR-4000 and AR-4000W grades.
3) If a mixture has high air voids, substantial hardening occurs within a relatively short period of time, relative to a mixture with low air voids.
4) Not only high modulus from good mix design but also physical properties of the asphalt cement play a vital role in enhancing the performance of asphalt mixtures.

5) The POB proved to be an effective means of aging mixtures and asphalt cements, since it produced substantial changes in the measured properties of the materials used. However, laboratory-made briquettes had much lower moduli than cores.

6) The change of a Fraass brittle temperature is a good indicator of the aging rate of the asphalt cement (that is a good evaluation parameter to measure the durability of an asphalt cement).

7) The resilient modulus of asphalt mixtures is a rapid indicator of the aging rate of mixtures. However, the fatigue resistance is a better indicator of expected long-term performance.

5.2 Recommendations

The following recommendations can be made from the results of this study:

1) Tests such as the Fraass brittle test for asphalt cement and repeated load tests for mixtures, should be used before and after aging, to measure the durability of asphalt mixtures. Fatigue and modulus tests are recommended to evaluate mixture performance.

2) The POB should be considered as a suitable device to "condition" mixtures to represent field oxidative aging.

3) To evaluate the mixture susceptibility to aging, a minimum "modulus aging ratio" could be required in the mix design process. This should be used where mixtures would be used in regions where oxidative aging is a particular problem.

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APPENDIX

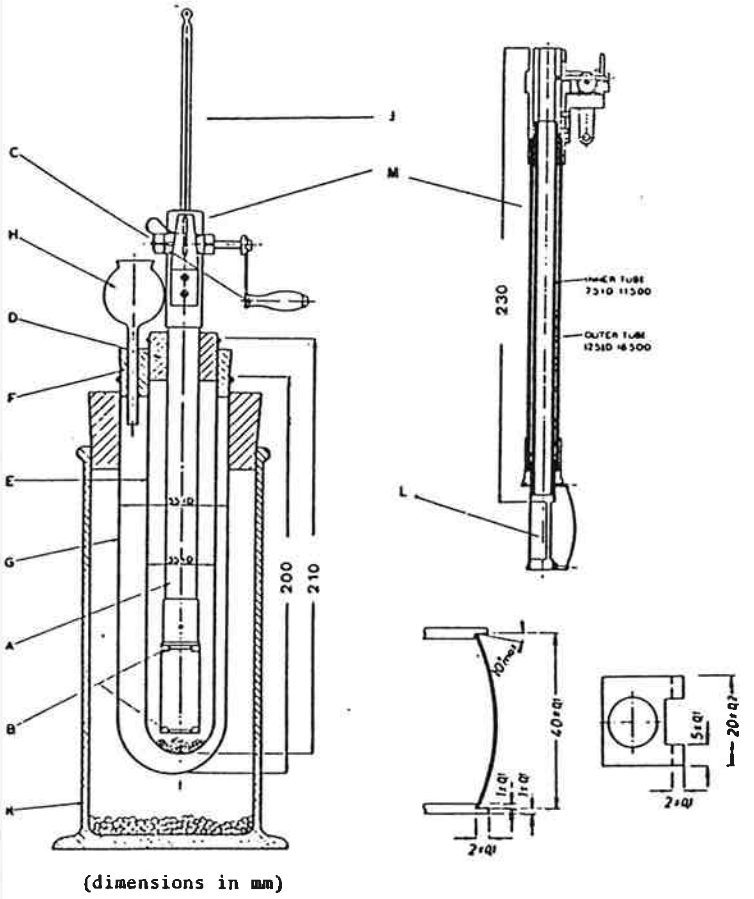
SAMPLE PREPARATION AND TESTING PROCEDURE OF FRAASS BRITTLE TEST

Figure 1 gives a general description of the component parts of the Fraass apparatus. The following describes the sample preparation procedure and test procedure. Throughout these procedures temperature and preparation times should be controlled very carefully.

A. Sample Preparation

The following are the steps used for the sample preparation:

1. Heat the asphalt cement until it has become fluid enough to pour from the can in which it is supplied. The temperature required to heat the asphalt cement to this consistency should correspond to a viscosity of 100 ± 10 Poises obtained from a plot of viscosity versus temperature, such as a BTDC. Do not heat the can for more than one hour.
2. Pour the asphalt on waxed paper and spread as thin as possible. Cool the asphalt at 10°C for one hour.
3. Place an amount of the asphalt corresponding to 0.40 ± 0.01 gr. in the solid state on a standard Fraass steel plaque (41 mm x 20 mm) of known tare weight. For each test, prepare at least eight plaques. This process is done at temperature below 10°C, such that the asphalt cement can be cut with a razor.



KEY

A:	Concentric cylinder
B:	Plaque hinges
C:	Rotating handle
D:	Supporter rubber bung. Seals chamber one.
E:	Test-tube. Air chamber one. Contains small quantity of calcium chloride to absorb moisture.
F:	Supported rubber bung
G:	Test tube containing acetone
H:	Funnel
J:	Mercury thermometer
K:	Glass cylinder. Air chamber two contains small quantity of calcium chloride to absorb moisture.
L:	Location of the thermometer bulb
M:	Bending apparatus

Figure A.1. Schematic of Fraass Apparatus (after Ref. 44).

4. After placing the sample on a steel plaque, heat the plaque on the baffle plate (Figure 2) cautiously. The temperature on the heating plate at the edge is the temperature used in step 1. Before heating, the level of the heating plate should be checked and maintained.
5. Place no more than three plaques around the edge (Figure A2) of the heating plate. Using a very thin needle, after the sample has softened, induce the sample towards the edge of the plaque by breaking the surface tension. This step should be done as soon as possible (within 3 minutes).
6. Cool the plaques at room temperature for about one hour then place in a refrigerator until executing the Fraass test.

B. Testing Procedure

The steps of testing are the following:

1. Check the bending apparatus and the distance between the plaque hinges. The distance should be 40 ± 0.1 mm as shown in Figure 1.
2. Place a small quantity of calcium chloride or anhydrone in the test tube E and the cylinder K (Figure 1).
3. Assemble the test tubes as shown in Figure A1, and fill the annular space between E and G to about half its height with acetone.
4. Attach a thermocouple behind the plaque and place the plaque between the clips of the bending apparatus, bending the plaque gently to do so, and mount the bending apparatus in the tube E.

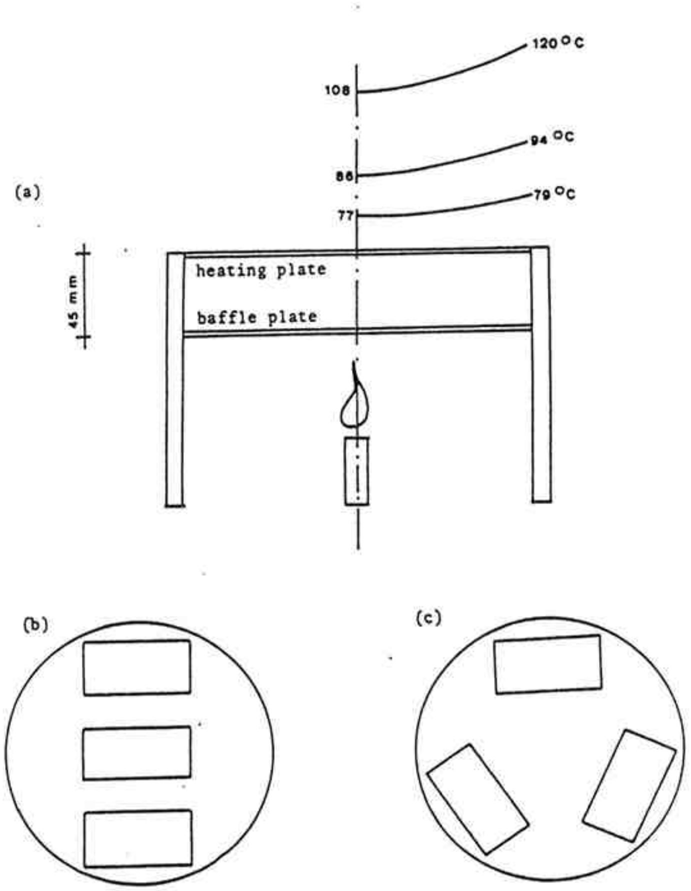


Figure A2. Schematic of Fraass Sample Preparation

(a) Temperature Gradients on Plate for Three Different Flame Intensities

(b) Plaques Positioned the Wrong Way

(c) Better Way of Placing Plaques During Preparation

5. Add solid carbon dioxide to the acetone at such a rate that the temperature falls at a rate of 1°C per min.
6. When the temperature reaches 0°C, start to turn the handle "C" at a rate of one revolution per second for 11 or 12 turns and then unwind the handle at the same rate.
7. Repeat step 6 until one or more cracks appear on the sample. Record the temperature at this point as the Fraass breaking point.