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WA-RD 85.1

Evaluating the Effectiveness of Liquid Anti-Strip Additives in Asphalt Cement

Final Report

August 1985

Washington State Department of Transportation
Planning, Research and Public Transportation Division
In Cooperation With
United States Department of Transportation
Federal Highway Administration

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
WA-RD-85.1		

4. Title and Subtitle Evaluating the Effectiveness of Liquid Anti-Stripping Additives in Asphalt Cement	5. Report Date June 1986 6. Performing Organization Code
7. Author(s) David A. Malsch	8. Performing Organization Report No.
9. Performing Organization Name and Address Same as 12	10. Work Unit No. 11. Contract or Grant No. HR0613
12. Sponsoring Agency Name and Address Washington State Department of Transportation Transportation Building Olympia, WA 98504	13. Type of Report and Period Covered Final Report 14. Sponsoring Agency Code

15. Supplementary Notes

Research conducted in cooperation with U.S. Department of Transportation, Federal Highway Administration

16. Abstract

The use of liquid anti-stripping additives (LAA) for reducing moisture damage in asphalt concrete pavements was evaluated using pavement cores from pairs of projects with-and-without LAA. Results were analyzed using the wet/dry tensile strength ratio (TSR). In addition, a diazo-dye for determining the presence and concentration of polyamine LAA was attempted to access the value or the test as a field procedure.

The chemical additive detection tests proved inclusive, but the TSR analysis gave a positive indication of the need for the use of LAA in the wetter environments of the state. For drier environments and for low traffic volumes, use of LAA may not be cost effective. The indirect tensile test was judged to be a potentially valuable tool for statewide use in the evaluation of anti-stripping additive needs.

17. Key Words	18. Distribution Statement
Bituminous concrete, moisture damage, stripping, anti-stripping agents, indirect tension test, diazo-dye test, pavement life	No restrictions. This document is avalible to the public through the National Technical Information Service Springfield, VA 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
		79	

Form DOT F 1700.7 (8-69)
DOT FORM 341-004 6/72

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Research Report

EVALUATING THE EFFECTIVENESS OF LIQUID ANTI-STRIP ADDITIVES IN ASPHALT CEMENT

Proposed to

Washington State Department of Transportation

and

Federal Highway Administration, USDOT

by

David A. Malsch, P.E. *

Revised August, 1985

ACKNOWLEDGEMENTS

The research program which will be presented in this document was as the result of a desire for knowledge by the Washington State Department of Transportation. That desire was for an assessment of the applied technology in the area of some asphalt cement additives, additives which are designed to promote the adhesion or bond between the asphalt and the mineral aggregate of the asphalt concrete mix during moisture attack in pavements.

The pioneering work done by Dr. R. P. Lottman, University of Idaho, in his study of the mechanism of moisture damage in asphalt concrete pavements was a significant factor in the decision of where to pursue this research. The Department of Transportation utilizes the Lottman testing procedure to assess antistripping additive demand, yet there existed a desire on the part of the Department to maintain support of an in-state institution. The cooperative attitude existing in the engineering programs of Washington State University and the University of Idaho located just nine miles apart made it possible for Dr. Lottman to supervise the research and for the Department to achieve its goal. We appreciate and support that cooperation.

That research, and a FHWA fellowship, contributed to the possibility for the author, a career employee of WSDOT, to pursue advanced studies in Civil Engineering after more than twenty years since receiving his undergraduate degree. But more importantly, the author is indebted to his wife, Margaret, who believed that his ideas were sound, and who was again willing to tolerate a student-husband.

ABSTRACT: The use of liquid antistripping additives (LAA) for reducing the sensitivity of asphalt concrete pavements to moisture damage has grown steadily in Washington State for the past eight years. A factor in that increased usage has been the application of testing procedures involving accelerated conditioning and indirect tensile strength tests. The techniques were perfected by Dr. Robert P. Lottman under NCHRP Project 4-8(3) for predicting moisture induced damage to asphaltic concrete. This study applied the Lottman procedures to pavement cores from projects using related-aggregate sources. Projects were paired on a with-and-without LAA basis in three climatically diverse areas of Eastern Washington. Results are analyzed using the wet/dry tensile strength ratio (TSR). In addition, a diazo-dye test developed by the Carstab Corporation of Cincinnati, Ohio, for determining the presence and concentration of polyamine LAA was attempted to help evaluate the effectiveness and uniformity of application of LAA and to assess the value of the test as a field procedure. The chemical additive detection tests proved inconclusive, but the TSR analysis has yielded positive indication of treatment need in the wetter environments. For drier environments and for low traffic volumes, the use of LAA may not be cost effective. The indirect tensile test is seen as a potentially valuable tool for district-level labs using normally available equipment.

KEY WORDS: bituminous concrete, moisture damage, stripping, antistripping agents, asphalt cement adhesion, asphalt concrete failure, indirect tension test, pavement life, flexible pavements.

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CHAPTER 1

Introduction

The effectiveness of liquid anti-strip (antistripping) additives (LAA) in asphalt cement for reducing asphalt concrete stripping in the presence of moisture can be defined as how well they perform in reducing or preventing the loss of adhesion of the asphalt cement with the mineral aggregate. The subject has been the topic of ongoing discussions for many years. The proper selection and use of the liquid antistripping additives has shown that they can perform satisfactorily in laboratory specimens of asphalt concrete mixtures, and if they are effective, they should extend the service life in the field of pavements which might otherwise fail due to the effect of moisture-induced damage. Yet it is a fact that the field applications of LAA additives do not always seem to yield the satisfactory results demonstrated in laboratory tests.

Problems in Eastern Washington which tend to plague pavement designers include the following:

(1) Old decaying pavements which were produced using untreated asphalts with aggregates from sources which are now shown to be severely strippable by the newer, current tests.

(2) Old pavements now covered by seal coats or overlays and which seem to be stripping but apparently weren't when they were "bare".

(3) Recently laid pavements displaying serious stripping failures well before the passage of any reasonable pavement life.

This research was undertaken to determine the effectiveness of liquid antistripping additives used in Washington State by comparing field performance and laboratory testing. This comparison was designed to secure additional evidence supporting or rejecting the continued use of the commonly available liquid antistripping additives, and to provide a basis for recommendations on the use and selection of liquid antistripping additives for the near future.

The implementation of this research by the Washington State Department of Transportation (WSDOT) could include one or more of the following stratagies:

1) Continue using liquid antistripping additives if this research shows that they are effective.

2) Consider other procedures if the additives are not effective, either in terms of more favorable pavement serviceabilty or reduced economic impacts.

3) Changes in the initial testing and evaluation procedures for the proposed paving materials, and changes in the application and verification of the additive material in the field in order to assure uniformity in the final paving product.

4) Recommendations of new requirements for, or types of, additives needed to control the stripping of asphalt from mineral aggregate.

CHAPTER 2

SITE SELECTION

Climatic Considerations

For study purposes the State of Washington can be divided into two diverse climatic regions that are separated by the Cascade Range. The Western Washington region, west of the ridge, (Figure 1) is characterized by abundant rainfall and moderate air temperatures while the Eastern Washington region, east of the ridge, is considerably more arid and has wider temperature fluctuations. For example, rainfall in the mountains west of the ridge averages 190 to 203 cm (75 to 80 inches) per year compared to 20 to 25 cm (8 to 10 inches) per year east of the ridge, most of which occurs in the spring and fall.

There is a marked change descending easterly from the Cascade Crest in the 80+/- Km (50+/- miles) to the Columbia River to a much more severe climate than that of Western Washington. Along the Columbia River and to the east, air temperature extremes range from average lows of minus 18°C (0°F) in the winter to average highs of 38°C (100°F) in the summer. The hot, dry days of summer tend to produce high pavement temperatures which sometimes exceed 60° (140°F). Roadways in the southern part of the region tend to be and remain in a drier state than those of the central and northern parts of the region. On the eastern edge of the state, in the 80+/- km (50+/- miles) west of the Idaho border, the terrain begins to rise to meet the Idaho Bitterroot Ridge and the maximum

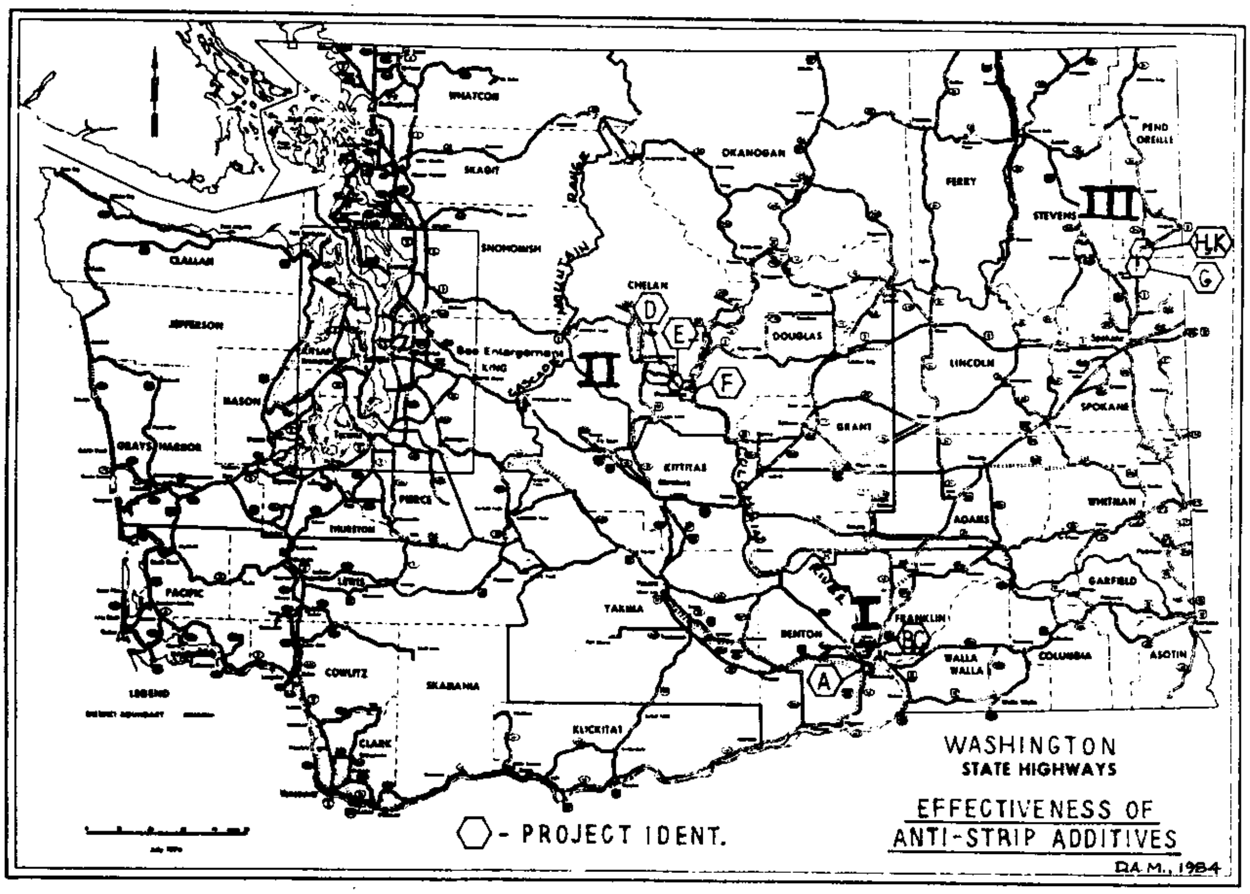

Figure 1--Vicinity map.

and minimum temperatures are somewhat cooler. Along with this decrease in temperature is a rise in the amount of precipitation to a range of 51 to 64 cm (20 to 25 inches) per year.

Limitation of research funding, time constraints and the totally different regional climatic conditions of Eastern and Western Washington influenced the decision to limit the study to the eastern region. Site selection analysis for this research identified three climatically diverse areas within the Eastern Washington region. The south-central area (Area I) includes Yakima, the Tri-Cities (Richland, Pasco and Kennewick) and Walla Walla. The north-central to east-central area (Area II) includes Okanogan, Wenatchee, Moses Lake and Ritzville. The third area (Area III) is the 80+/- km (50+/- mile) strip along the Idaho border lying generally north and south of Spokane.

During the winter temperatures of pavements at all sites usually fall below freezing, and in differing degrees, they will experience cyclic freezing and thawing (temperature swings through the freezing point) during the winter as well as the spring and fall seasons. In general, Area I is usually the warmest area and is followed respectively by the Area II and Area III as shown in Table 1 on page 10. Likewise, the Area I sites are the driest and the Area II and III sites tend to be progressively wetter.

Project Selection

In each of the three climatic areas, there were two or more projects of significant size which were paved from the same source of aggregates. At least one of the projects was paved using asphalt cement treated with a liquid antistripping additive, and at least one was paved without the additive. The need for antistripping additive was determined in WSDOT's State Materials Laboratory located at Tumwater, Washington, using the Lottman procedures.

After analysis of WSDOT paving summaries and project records, and after considering the availability of WSDOT support equipment needed to secure field samples, Projects A, B and C were selected from the Tri-Cities area (Area I). Then Projects D, E and F were selected from the Wenatchee area (Area II). Finally Projects G, H and K were selected from the area north of Spokane (Area III).

CHAPTER 3

FIELD CHARACTERIZATION

Much of the pertinent information on design and construction of the asphalt concrete pavements in Washington State was collected by reviewing WSDOT policies and records. The following is a summary of that review.

Additive Selection and Determination

In order to determine the dosage of liquid antistripping additive, WSDOT has used a modification of the Lottman procedure [1][1] for predicting moisture-induced damage to asphalt concrete. The test, identified as WSDOT Test Method No. 718 [2], is incorporated into the job mix design procedure (see APPENDIX A & B). A preliminary assessment of antistripping additive need is made at the pre-contract design phase. It is based on the proposed aggregate source and a "typical" brand of asphalt cement available in the project area.

Following award of a construction contract, samples of the actual aggregate to be utilized for the job, along with average stockpile gradations and the brands of asphalt cement and antistripping additive, are forwarded to WSDOT's Tumwater laboratory for a job mix design. Beginning with 1984 projects, WSDOT has somewhat modified the

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[1] The underscored numbers in the brackets refer to the references tabulated in the **REFERENCES** section.

Method No. 718 stripping test. The laboratory procedure now adopted and used includes cooling the preconditioned specimens to 25°C (77°F) and mechanical indirect tensile testing at a deformation rate of 5.08 cm (2.0 inches) per minute. The combination of the cooling and the tensile splitting is giving a more severe laboratory prediction of stripping, and it is resulting in somewhat higher dosing rates for the antistripping additive. The relationship between test speed and temperature is reported in an investigation by Lottman [3] in 1971 for the Idaho Department of Highways and the Federal Highway Administration (FHWA), and in another study by Maupin [4] in 1979.

Additive Application

The Department of Transportation requires certification of the liquid additive dosage from the asphalt cement producer as the means to monitor addition of the additive to the asphalt cement. Interviews with WSDOT project field personnel revealed a strong suspicion, based upon the presence (or absence) of a distinctive odor, that the additive may not have been present for some asphalt shipments or series of shipments to the asphalt concrete producer even though the certificates indicated otherwise. In other situations the presence of an additive was discernible even though certification of dosing was lacking. There was an expressed interest in having a test which could be run in the field (or in the District laboratory) to verify the presence and concentration of an additive.

The methods used to apply the liquid additive to the asphalt cement were also investigated. Field personnel had reported accounts from transport drivers of their having to bucket the additive material into their load by hand. Some terminals have made provisions for in-line blending of the additive while the asphalt cement is being loaded into the transport vehicle. Usually no WSDOT personnel are present while loading and blending the materials; shipments are made day or night (or both) depending on the demand and schedules.

Project Information

On-site inspections were made of each of the research project sites in order to note any local factors not evident from the record analysis and to photograph the projects. Table 1 is a summary of the project data.

Area I--Several factors of interest were gleaned from records for Projects A, B and C in Area I which may impact the test conclusions. Project A was paved in 1979. A mix design was prepared in advance and at that time the stripping potential was checked using a stripping test based on the Lottman procedures [1]. The results indicated only slight after 24 hours and no antistripping additive was used in the paving.

Projects B and C are from the same large construction project; the construction is not yet complete and, as noted in the project data summary, some portions of the new construction are not yet open to traffic. The asphalt concrete mix supplier was the same for Projects A, B and C. In mid-1982, a mix design was prepared by the WSDOT State Materials Laboratory at Tumwater using a particular asphalt cement and

Table 1--Project data summary.

Project I. D. Group	Year Paved	Site Information Length, miles (b)	Traffic ADT	Climatic Factors (e)	Aggr. Type	Mix Design Information Asphalt Cement Type	Anti-Stripping Type	Anti-Strip Amnt.	Construction Information Anti-Strip Appli.	Verif. Method	Mix Voids	Cur. Field Cond.
A.	1979	1.77 U	26,800	Sp: M, W-D Su: VH, D Fa: M, D-W Wi: C, D, Fr	Gravel	AR4000W (f)	...	...	...	...	6.8%	Good
B.	1983	(c) U	12,000		Gravel	AR4000W	Pavebond Special (g)	0.25%	R. T. (h)	Cert. (i)	8.6%	New
C.	1984	(d) U	6,600		Gravel	AR4000W	Pavebond Special	0.50%	R. T.	Cert.	9.6%	New
D.	1977	9.31 R	12,400	Sp: M, W, F/T Su: H, W-D Fa: M, D-W Wi: C-VC, Fr	Gravel	AR4000W	...	...	...	...	6.9%	Fair
E.	1975	3.22 S	13,700		Gravel	AR2000	...	...	...	...	7.1%	Failed
F.	1978	3.61 S	11,000		Gravel	AR4000W	Pavebond Special	0.75%	R. T.	Cert.	5.5%	Good
G.	1981	5.43 R	4,800	Sp: C, W, F/T Su: H, W-D Fa: C, W Wi: VC, Fr	Gravel	AR4000W	Pavebond AP Special	0.50%	R. T.	Cert.	7.9%	Good
H. (a)	1970	6.12 R	4,100		Gravel	85-100 Pen.	...	...	...	...	9.7%	Fair
K.											8.1%	Failed

FOOTNOTES:

a. Group H and K cores were from same project. Group H extracted from inside lanes; group K from outside lanes.
b. Key: Urban (U), Surburban (S), Rural (R).
c. Interchange crossroad - 0.75 miles represented. Use limited - mainline not yet open to traffic.
d. Interchange ramps. Not yet open to traffic.
e. Key: Season (Sp, Su, Fa, Wi); Temperature (Very Cold, Cold, Moderate, Hot, Very Hot);
Pavement State (Dry, Wet, Freeze/Thaw, FRozen).
f. Aged Residue (AR), viscosity graded asphalt cement.
g. "Pavebond" is a tradename of the Carstab Corp., Morton Thiokol, Inc., Cincinnati, OH.
h. Added at refinery terminal (See Text).
i. Certified by shipper.

antistripping additive selected by the contractor for the work represented by Project B, Project C and several other projects in the area. In early 1983 the asphalt concrete supplier elected to change to another asphalt cement source (along with its supplier-selected, but different, antistripping additive brand and type). For Project B and other projects paved in the Fall of 1983, a new mix design was issued by referencing from the 1982 mix design, but without a predictive test despite the change in the source of the asphalt cement and additive. As observed by Maupin [4] (and as examined under the heading **Discussion**) changing asphalt cements and/or antistripping additives using a source of known strippable aggregates can produce unpredictable results. Design recommendations should be supported by retesting the combination of aggregate, asphalt cement and antistripping additive prior to accepting the newly treated asphalt concrete material.

In November, 1983, another mix design, again referenced to the originally tested 1982 design, was issued for paving bridge decks within the overall construction project covering B and C. This design included the recommendation that the anti-stripping additive dosage be increased to 0.50 percent by weight of the asphalt cement. It was also suggested that a new tested mix design should be prepared in the spring of 1984. The recommended testing for a mix design apparently was not accomplished prior to starting paving on Project C in 1984. Ultimately, another mix design was issued in mid-April, 1984. This "mix design" was actually a confirmation of the estimated dosage; it was based upon tests of seven construction control samples of job-mixed asphalt concrete from the early 1984 construction representing 14×10^6 kg (15,400+ tons) of asphalt concrete mix.

Area II--Project D was paved in 1977 without antistripping additive. When the aggregates were originally tested for the mix design in 1977, the Modified Immersion-Compression Test (WSDOT Test Method No. 707 [5]), which is similar to ASTM Test for Effect of Water on Cohesion of Compacted Bituminous Mixtures (D 1075-81), did not disclose the critical need for the additive. In 1978, WSDOT applied the recently published Lottman field evaluation procedure [1] to assess what was observed in the field as a premature, between-the-wheeltracks strip raveling failure. The testing revealed severe strippability of the aggregates, a condition for which antistripping additive should have been specified.

The cores for Project E had been obtained in the fall of 1983 in preparation for a planned rehabilitation project. Project E was included because, although only two years older than Project D, it was in an advanced state of failure due to what appeared to be stripping. The cores were not needed for their original purpose so they were incorporated into the research to investigate and compare results with the adjoining Project D. Project E, like Project D, was paved from the same source of aggregates without antistripping additive based on the prediction of the Modified Immersion-Compression Test.

Project F, the originally selected companion to Project D, was paved in 1978 with liquid antistripping additive. Like Project D, the aggregates had been evaluated using the Modified Immersion-Compression Test and showed only a slight tendency for stripping. Because the aggregates came from the same source as for Project D the recommendation was made to add antistripping additive on the basis that more severe stripping was predicted by the Lottman procedure when it was used to analyze the premature failure within Project D. Additive dosage for this project was initially selected on a somewhat arbitrary basis at 1.0 percent by weight of asphalt cement. For reasons that are not clearly documented, that dosage rate was reduced to 0.75 percent before the wearing course lift (the layer from which sample cores for this research were obtained) had been paved. Based upon the test results which follow, the relatively high retained tensile strength ratios suggests that the dosage was quite adequate.

Area III--Project G was selected on the basis that it too, contained antistripping additive using an aggregate source in common with a project which had not been so treated. The companion untreated project, was a much older pavement and is the one represented by the data in groups H and K. That pavement connected to the northerly end of Project G; it has been rehabilitated by recycling after it was sampled.

The project represented by data groups H and K is interesting for several reasons. It is an isolated section of 4-lane rural expressway, it was the oldest project section considered for this research, and the outside lanes were in an advanced state of what appeared to be stripping failure while the inside (high speed) lanes were in significantly better condition as shown in Figure 2. The inside lanes were sampled separately from the outside lanes; this allowed a more in-depth study of the extent of the differential failure.

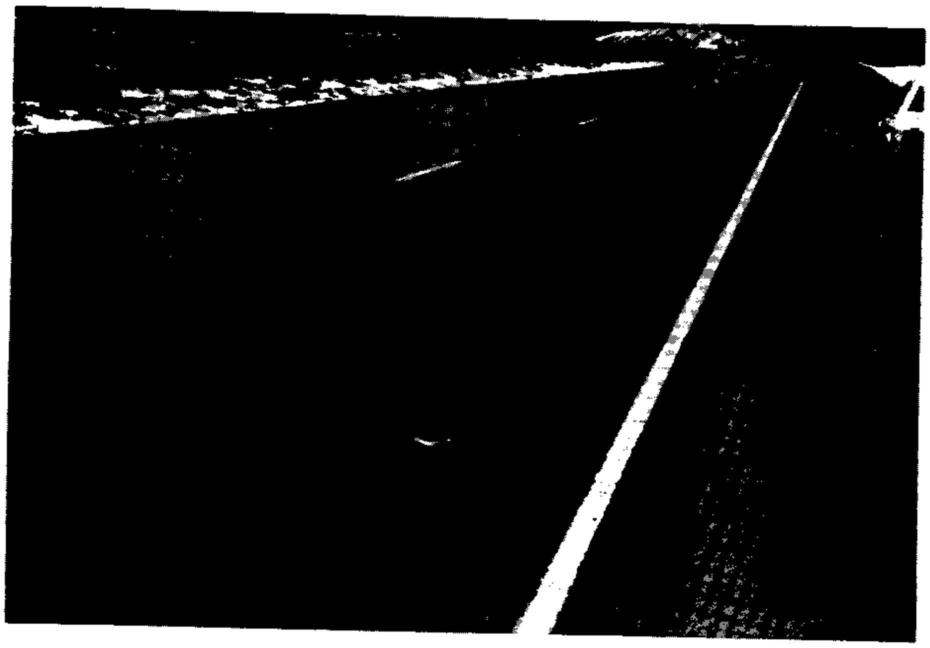

Figure 2--Advanced stripping failure.

(Sample Groups H and K)

Field Sampling

Core samples of the various pavements were secured with the cooperation of the three eastern Washington districts of WSDOT. Sampling activities for the projects were coordinated with the three District Materials Engineers; except as noted above, sampling was accomplished in the spring of 1984.

All field cores were removed using standard wet coring methods. The coring was in close conformity with the ASTM Method for Sampling Bituminous Paving Mixtures (D 979-83) and the companion WSDOT Test Method No. 712 [6]. But as mentioned under **ANALYSIS OF RESULTS**, at each coring site a different approach to "random" sampling was used. None were in very close conformity with ASTM Standard Practice for Random Sampling of Construction Materials (D 3665-82) or the correlative WSDOT Test Method No. 716 [7].

CHAPTER 4

LABORATORY CHARACTERIZATION

Background Information

Activities covered under this heading include the preparation of the cores to be tested and the tests that were conducted on those cores. While the Lottman's NCHRP field study [1] addressed itself to the bottom lift(s) of asphalt concrete pavements, the purpose of this study was to evaluate the antistripping additive effectiveness in the top (wearing) course. It was initially contemplated that both the tensile strength and modulus ratios would be required to characterize the pavement serviceability. The research goals were reviewed with Dr. Lottman and it was decided that a simplified approach which involved only tensile strength ratios (TSR's) and additive presence and strength would be adequate to address the study objectives. Additional testing would be assessed on a need basis to characterize problem areas.

Pavement Core Preparation

Field sample cores of asphalt concrete pavements were prepared for laboratory testing by separating the top lift (layer) of asphalt concrete pavement from other lifts in the core sample. Most of the cores were well bonded throughout and the top lift had to be removed by sawing after freezing the core. The cores were frozen to avoid rehealing any existing lost adhesion of the asphalt film. In a few cases, separation occurred at the lift interface without sawing.

In some cases, pavement failure caused by stripping had progressed to the point of disintegration of the AC mix at the interface of the top and lower lifts and only part of the top lift remained. This is one characteristic of some stripping failures and is illustrated in Figure 3, a photo of two cores taken from the Project K in Area III, north of Spokane. Both cores were removed from between the wheel paths at locations that were 1.6 Km (one mile) apart. Other cores in this set and others from Project E showed similar failings.

Indirect Tension Test

Cores were tested according to the procedure developed by Dr. Robert P. Lottman in the NCHRP Project 4-8(3) [1] to assess moisture-induced damage to asphaltic concrete.

Test Procedures--The same test procedures were followed for all core sets with the exception of the cores for Project C. Half the cores in each group were vacuum saturated with water under 66 cm (26 in.) mercury of vacuum drawdown and then cooled to 12.8°C (55°F) before mechanical testing. The other half of the cores were dried to a constant weight in a vacuum desiccator at approximately 10 cm (4 in.) of mercury drawdown) before testing in the dry state at 12.8°C (55°F). The vacuum drying of cores is not contained in the Lottman procedure; but after reanalyzing the procedure with Dr. Lottman, it was decided to employ the

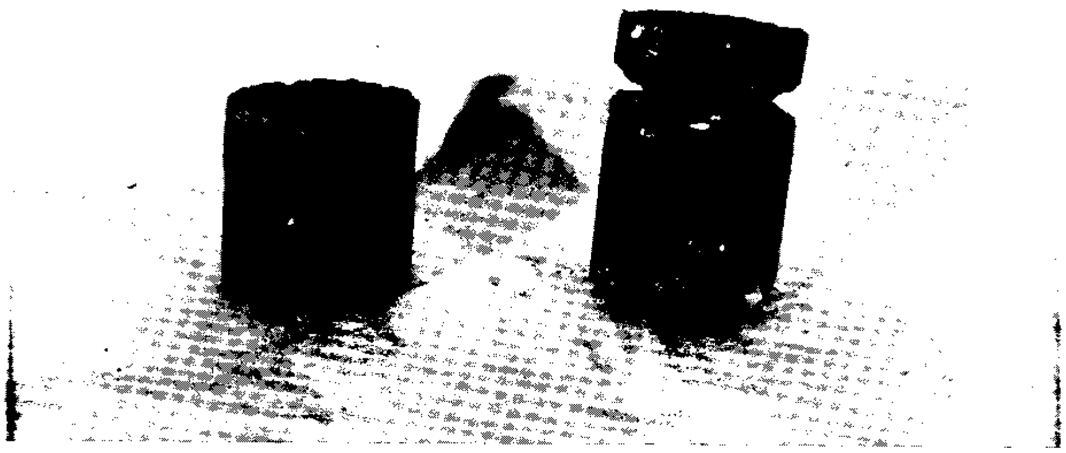

Figure 3--Cores from Group K.

techniques verified by Schmidt and Graf [8] for all specimens of this research. The process shortened drying times yet conserved the desired experimental accuracy. A more complete description of the test used in this research may be found at APPENDIX D.

After mechanical testing, the dry-state cores were used to determine the voids content of the represented pavements. WSDOT Test Method Nos. 704 [9] and 705 [10] were used. The methods are similar to ASTM Percent Air Voids in Compacted Dense and Open Bituminous Paving Mixtures (D 3203-83).

Project C was paved in the spring of 1984, and thus had not been subjected to natural or uncontrolled freeze/thaw preconditioning. The Project C sample set of six cores was divided into thirds. The cores were paired; one pair was vacuum dried and tested as dry specimens as explained above. The other two pairs were vacuum saturated with distilled water. One pair of these two pairs was cooled and tested as wet-saturated specimens as above and the remaining pair of the set was subjected to laboratory preconditioning. That procedure consisted of freezing after vacuum saturation and then water bath soaking at 60°C (140°F) for 24 hours before cooling and testing at 12.8°C (55°F).

Mechanical testing was conducted at 12.8°C (55°F) and consisted of loading to failure by line-loading the side of the cylindrical core. This was done at the controlled vertical deformation rate of 0.165 cm (0.065 in.) per min. in order to determine the diametral tensile strength of the core. The split (interior) faces of the cores were examined with the assistance of a 10-power stereozoom microscope. The amount of stripping was estimated by visual examination.

Test Results--The Diametral Tensile Strength (S_t) of the test specimens were used to calculate the index of the relative strengths. This index is called the Tensile Strength Ratio (TSR), defined as the ratio of S_{t-wet} (or preconditioned) to S_{t-dry}. Use of the dimensionless TSR index is considered a more reliable approach for assessing the relative moisture damage potential of samples compared to tensile strength values or stresses. Low ratios are associated with the mixture's inability to resist moisture effects [1]. Average TSR's for the various projects considered in this research were computed and do form the basis of the analyses. The results are presented in Table 2. It is interesting to note the variability and the relatively low stripping in some cases at about the same level of TSR. This implies that moisture damage in these projects is not only caused by adhesion loss (stripping) but also by cohesion loss (diffusion of water into the asphalt-fines binder).

Table 2-- Summary of physical testing.

Project I. D. Group	Pavement Age, years	Dry Unit Weight, pcf	Voids, percent	Tensile Strength Wet, psi	(Samples)	Tensile Strength Dry, psi	(Samples)	Tensile Strength Ratio	Wet Core Stripping Observed, percent	Footnote Reference for " * "
A.	5	145.4	7.5	127.4	(3)	162.3	(3)	0.79	0 - 1*	a
B.	< 1	141.9	8.6	95.3*	(2)	135.1	(3)	0.71	10 - 15	b
C.	< 1/2	142.3	8.4	70.0 68.1*	(2) (2)	96.0 96.0	(2) (2)	0.73 0.71	5 5	 c
D.	7	143.6	6.9	127.1	(3)	213.7	(3)	0.60	20*	d
E.	9	144.7 129.9	7.1 16.6*	138.5 58.1	(2) (1)	220.8 70.5	(2) (1)	0.63 0.83	15* 5	e f
F.	6	146.5	5.5	158.8	(3)	190.8	(3)	0.83	2	
G.	3	137.5	7.5	116.4	(4)	141.6	(4)	0.82	none	
H.*	14	138.6	9.6	148.1	(4)	176.4	(4)	0.84	1	g
K.	14	141.8	7.3	109.8	(4)	184.6	(4)	0.59	25 - 40*	h

FOOTNOTES:

a. One saturated specimen indicated 25 percent stripping.
b. One saturated specimen disqualified--failed prematurely at 34.2 psi. See text heading, Prediction Model
c. Accelerated conditioning by Lottman procedure.
d. Dry cores indicated 5 percent stripping.
e. Dry cores indicated 20-40 percent stripping.
f. One pair cores from experimental open-graded mix on a shoulder section.
g. Group H and K were from same project. Group H extracted from inside lanes; group K from outside lanes.
h. One saturated specimen indicated 1 percent stripping.

Additive Presence and Concentration Test

The additive presence and concentration test is a diazo dye-forming chemical test. The procedure was developed by the Carstab Corporation of Cincinnati, Ohio [11], as a means to confirm the presence of polyamine additives contained in asphalt cement and asphalt concrete and to measure the residual strength of the agent. It was postulated that the test could also be used be used to verify that the material had been properly and uniformly incorporated into the asphalt concrete when that material was produced by checking for the degree of variability of results. Concluding data was intended to assess the effectiveness of liquid antistripping additives used by WSDOT over the long term.

The WSDOT had previously conducted an evaluation of the polyamine additive detection procedure for asphalt cement on the major asphalt cements together with the brands and types of additives commonly available in Washington State [12]. They did not perform the additive concentration portion of the procedure nor did they conduct tests using asphalt concrete. The general conclusion of their testing was that the procedure did not produce consistent results for all anti-strip additives, but that it may be accurate for some types of those additives. Because of the inconsistency, the WSDOT chose not to implement the polyamine additive detection test as a routine procedure.

The WSDOT report did show that the procedure gave positive detection results when used with Carstab products. All of the additive-treated pavements selected for study in this research were identified by project records as containing the Carstab additives shown in Table 1. Thus, it was decided to evaluate the previously untried test for polyamine additives in asphalt concrete for this research and to include the additive concentration portion of the test to assess both residual strength and uniformity; then too, it should be possible to assess the effects of additive demand by the aggregate as well as by the asphalt.

Materials/Facilities for Tests--Asphalt concrete batches for additive concentration standards were prepared at the University of Idaho, Civil Engineering Laboratory. The antistripping additive presence and concentration tests were conducted in the Washington State University Environmental Engineering Laboratory. Test samples consisted of portions of cores that were retained from the saturated indirect tensile strength tests. All of the field asphalt concrete mixes represented by the cores had been designed for the addition of antistripping additive and project records verified the use of the additive in the mix. Lab mixed asphalt concrete, containing aggregate from the project source and asphalt cement from WSDOT was used for additive concentration standards. The antistripping additive was supplied by the Carstab Corporation.

Test Procedure and Results--The procedure for the detection and concentration tests for polyamine additives to be found in the asphalt concrete cores and standards is included for reference as APPENDIX E. Briefly stated, the test is identical for both core samples and for standards and consists of treating asphalt concrete mix with an asphalt solvent and an extraction solution. An emulsion forms which eventually tends to separate into aqueous and solvent phases. A portion of the aqueous phase is treated with strong sodium hydroxide solution. Butanol is added to form another layered solution. The bottom layer is maintained at a pH of 11.0 and the top layer maintained between 7.5 and 8.5. A detector solution is added, and if the polyamine additive is present, an intense reddish coloration appears in the top layer; absence of the polyamine additive should result in a light, yellowish coloring of the top layer. For concentration tests, a small amount of the top layer is diluted with more butanol and, after again adjusting the pH, a coloration measurement is attempted using visual color standards or a lab spectrophotometer.

Many problems were experienced in trying to get the emulsion to break so that the first of the aqueous solutions could be separated for use in the later stages of the test. Under Carstab's guidance various techniques were tried in order to induce separation. A few of the samples produced a positive indication of presence of the polyamine in cores and treated standard mixes, but not enough of them responded to develop reliable concentration data.

All facets of the test were reanalyzed with Carstab's help, the test apparatus was reassembled and again, reasonable results could not be obtained. For the purposes of this research, the antistrip concentration testing by the author is deemed inconclusive.

CHAPTER 5

ANALYSIS OF RESULTS

Background

Sampling Reliability--As previously stated, the three coring crews secured samples in their accustomed manner and as their schedules allowed. As a result, various methods were employed to select sampling locations. Upon analysis, none of the techniques meet the requirements for the probability methods specified in ASTM Standard D 3665-82, or the similiar WSDOT Test Method No. 716 [7]. While no sample sets can be classified as truly random, the coring location data has been analyzed and most can be characterized as representative of the field conditions and consistent with usual WSDOT practice.

Pavements--There were no control sections found amoung the Eastern Washington highways that represent the same mix and age including both additive-treated and non-treated asphalt cement. But with the exception of a short length of "open graded" mix placed in Project E (which is reported separately in Tables 1 and 2), all of the asphalt concrete represented herein meets the same "dense graded" specification. Furthermore, each area includes additive-treated and non-treated projects from a common aggregate source. Therefore the test findings will be evaluated on the basis of overall performance of a project (keeping in mind that results can be influenced by the different pavement ages).

The idea behind the analysis is to look at the data and see if additive-treated pavements perform better than non-treated pavements; to see if climatic differences are important; and also, to see if it is economically justified to continue to design pavements in Eastern Washington using liquid antistripping additive.

Analysis Methodology--The author's observations, supported by those of Lottman and others, suggest that traffic volumes, pavement age and moisture availability are the chief factors in determining the potential of stripping damage. For this reason the projects selected for this research represent a range of pavement ages, different climatical conditions and a mixture of different traffic classifications.

Lottman has proposed [13] that the pavement serviceability can be predicted by applying tensile strength ratios (TSR's) for various aged pavements with pavement thickness design procedures used by highway agencies. At a certain point in the pavement's life, an untreated strippable pavement will be considered to be at terminal serviceability. Based on predictive tests of the aggregate/ asphalt/ additive system, this same pavement should have had a longer life if an effective antistripping additive had been employed when it was constructed. Of the projects in Area I, Project B and C contained a liquid antistripping additive and were paved very recently. Project C had not been subjected to freezing temperatures in the field. This condition allowed application of controlled preconditioning, and a prediction of moisture-induced and freeze-thaw damage was possible. By comparing the data from Projects A (without antistrip) and B (with antistrip) with that of Project C (with antistrip) a relationship was sought to interrelate the data from one group to another. If the relationship is valid, it should then be possible to develop a prediction model which could be applied to data sets in other areas. By using the model, the likely point represented by the current test data could be identified on the pavement's life curve and future pavement performance could be assessed. A measure of effectiveness of the liquid antistripping additive could then be defined as the increase in the pavement's serviceability relative to the non-additive-treated pavement as a function of time.

Another approach to the analysis of effectiveness of liquid antistripping additives would be to compare the benefits versus the costs of applying the antistripping additives. Projects to be compared must have used the same aggregate source and the expected pavement life difference due to time factor must also be considered. A recent addition to state-of-the-art analysis is a computer-based procedure for micro computers known as ACMODAS (Asphalt Concrete Moisture Damage Analysis System) [14] to assess this parameter. The procedure was developed by Dr. Lottman and his staff and was investigated for this research.

While resource limitations restricted the size and number of the sample lots (as previously described under **Field Sampling**), analysis of the available data did produce an indication of the effects of climate. Analysis of the data suggests reasons for different degrees and rates of pavement degradation for projects with and without antistripping additive in different climatic areas of the eastern region of Washington. The measure of effectiveness of the additive could be thus be defined as the difference in serviceability of pavements as a function of climate.

Evaluation

Referring to Table 2, one sees that the TSR values shown for Project B (as well as for C) are low but acceptable. They approach the minimum of 0.70 suggested by Maupin [4]; that value is nearly the median of minimum values reported by Tunnicliff and Root in the NCHRP Project 10-17 [15]. A much lower value was first calculated for Project B. One core produced a value, S_{t-wet} = 34.2 psi which contrasted sharply with the average value for the other two cores in the set, S_{t-wet} = 95.3 psi (s = 2.4). If all three values are used, then S_{t-wet} = 74.93 psi (with s = 35.32) and yields a TSR of 0.55 instead of 0.71 as selected. Voids and densities were calculated and compared with other cores in the set, but no reason was found to explain why the one core of the wet set failed so low. Inclusion of the low value would have produced excessive deviation and for that reason the data for that core was excluded from the test results. Nevertheless, the hot mix for Project B appears susceptible to spotty moisture damage exceeding "allowable levels".

Pavement cores taken from Project C in the spring of 1984, were assumed to be equivalent to the zero-age pavement cores tested in the Lottman NCHRP study [1]. Based on Lottman's later work [13,16], the prediction ratio for the assumed "1.5-year field dry stage time" was taken as the TSR equals 1.00 for dry cores; the long-term prediction ratio for the wet condition extending beyond the 1.5-year field dry stage was represented by the TSR for the conditioned cores (0.71). A representation of the data is shown in Figure 4, along with the TSR's for the field-aged pavements; A, B, and D through K.

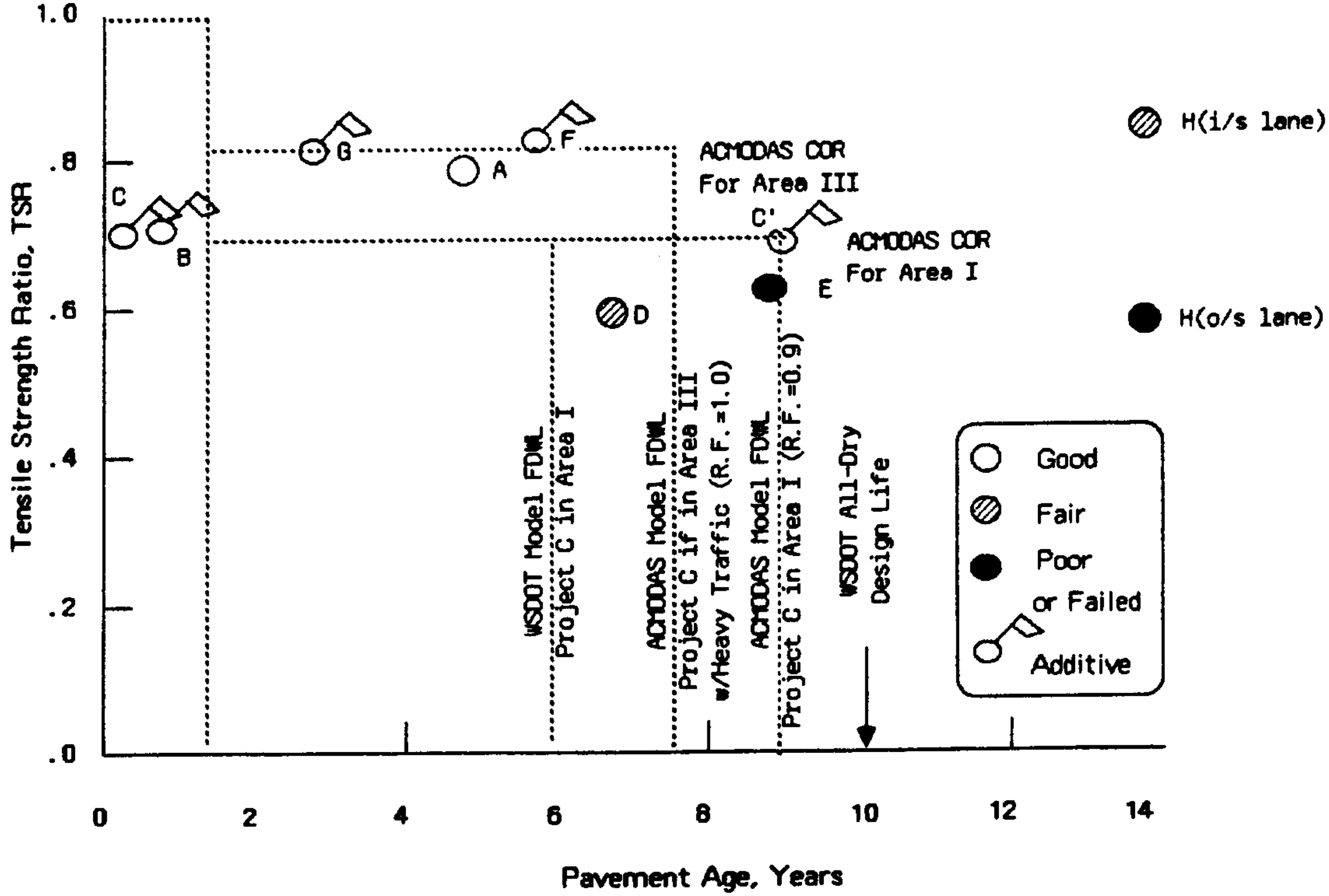

Fig. 4 — Two—step prediction model.

Prediction Model--Tensile strength ratios (TSR's) for Projects A, B and C in Area I were analyzed to try to establish a relationship between existing treated and non-treated pavements to assess remaining pavement life. Analysis of the available data suggests that it is too scattered and that an acceptable prediction model as initially contemplated for inter-group comparisons was not achieved. For instance, it is noted from Table 1 that the antistripping additive dose for Project B was 0.25 percent and the amount used for Project C was 0.50 percent. These doses were verified from the project records. Projects B and C were paved with a winter season between them, yet as seen from Table 2, the TSR's are virtually the same. The Project C dose was probably a somewhat better dosing rate in terms of additive demand as verified by the observed stripping. (As explained under **FIELD CHARACTERIZATION**, the difference in dosing rates was partly due to a preconstruction error since a mix design was not tested before paving began. The mix design used was taken from one issued six months before the 1984 paving began. The "mix design" that finally evolved was actually a confirmation report of testing of a much larger sample than would have been submitted to the WSDOT Materials Laboratory had everything gone, "by the book").

The Project B and C results were also open to question because the Project A (the control project) TSR ratios were so much higher than for B and C. The age of pavement A places it well beyond what is usually considered as the period of initial stiffening or strengthening in the early stages of pavement life [1,11].

The Project A ratios might be construed as equivalent to a tensile strength cut-off ratio (COR) [16] for asphalt concrete from the identified source. The COR is the minimum ratio of retained strength (wet versus dry strength) for acceptance of an asphalt concrete mix without inclusion of an antistripping additive. Aggregates for Project A, B and C were from the same source, but no antistripping additive was used in the Project A asphalt concrete.

At this point it was decided to compare tested results using the method suggested by Lottman [16] for predicting retained strength cut-off. In essence, an allowable reduction of all-dry field life is selected which results in a total field dry-wet life. The initial field dry stage time deducted and results in the field wet stage time. Using the proper strength parameters from pavement design theory, the TSR is calculated from the wet stage life; this value is the COR. For this research, there are insufficient data to precisely determine what effect the drier environment of Area I might have had on the aged pavement and on the new pavements, but there is a trend. There is some suggestion that a well-constructed pavement without treatment might perform just as well as an additive treated pavement of lesser quality in this type of environment.

The ACMODAS simplified two-step moisture stage was applied using the data for Project C as another type of a prediction model. This model is only useful to assess the serviceability the pavement which is being studied. The printout of the ACMODAS LCO prediction program for Pavement C is shown in Figure 5. ACMODAS LCO gives a prediction of the field dry plus wet life representing actual conditions and also the laboratory-predicted cut-off ratio (COR). Included in these computations are the user-selected regional index of 0.9 (representing a "moderate" freeze-

ACMODAS

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LCO ENTRY DATA

REGIONAL FACTOR	.9
FIELD ALL-DRY DESIGN LIFE (YEARS)	10.0
PERCENT ALLOWABLE REDUCTION OF FIELD ALL-DRY DESIGN LIFE	10.0
FIELD DRY STAGE TIME (YEARS)	1.5
DRY INDIRECT TENSILE STRENGTH AT 55°F (psi)	95.8
WET ACCEL COND INDIRECT TENSILE STRENGTH AT 55°F (psi)	68.1
DRY RESILIENT MODULUS AT 55°F (psi)	NO ENTRY
WET ACCEL COND RESILIENT MODULUS AT 55°F (psi)	NO ENTRY

LCO PREDICTIONS

FIELD DRY-WET LIFE (YEARS)	9.1
PERCENT CHANGE FROM FIELD ALL-DRY DESIGN LIFE	9.1 REDUCTION
TENSILE STRENGTH RATIO (BASED ON ENTRY)	0.71
APPROXIMATE MIN. TENSILE STRENGTH RATIO (TSR CUT-OFF) REQUIRED FOR PERCENT ALLOWABLE REDUCTION ENTRY	0.70

TSR CUT-OFF LESS THAN TSR ENTRY

Figure 5--Output from ACMODAS LCO for Project C

thaw climate with "moderate" traffic) and the user estimated 10-year field all-dry design life. The selected all-dry life is consistent with design procedures the Washington DOT [17]. The computed results were superimposed on Figure 4, the graph of the manually calculated ratio trends. Small differences in values between ACMODAS and manual methods are related to a feature of the program; it internally calculates with a reasonable modulus value (M_R) if the modulus values are not entered by the operator. Modulus value determinations were not conducted for this research as explained under **Lab Characterization.**

The predicted pavement life for Project C was also calculated using the WSDOT pavement design procedure including Miner's cumulative fatigue damage rule [18]. While ACMODAS is based on a "mechanistic" theory of elastic layer representation with relative fatigue life, the WSDOT procedure is an empirical one developed from laboratory testing which interrelates traffic load, the subgrade soil quality and the characteristics of the surfacing and paving courses. In the procedure the asphalt concrete is factored into an equivalent gravel thickness and calculations are based on the summation of equivalent gravel depths plus untreated surfacing depths in the cross section. Miner's cumulative fatigue rule is used to prorate the moisture stages, i. e. dry and accelerated moisture conditioned. Life reductions are made using a reduction of asphalt concrete gravel equivalency due to the loss of "cohesion" by moisture damage.

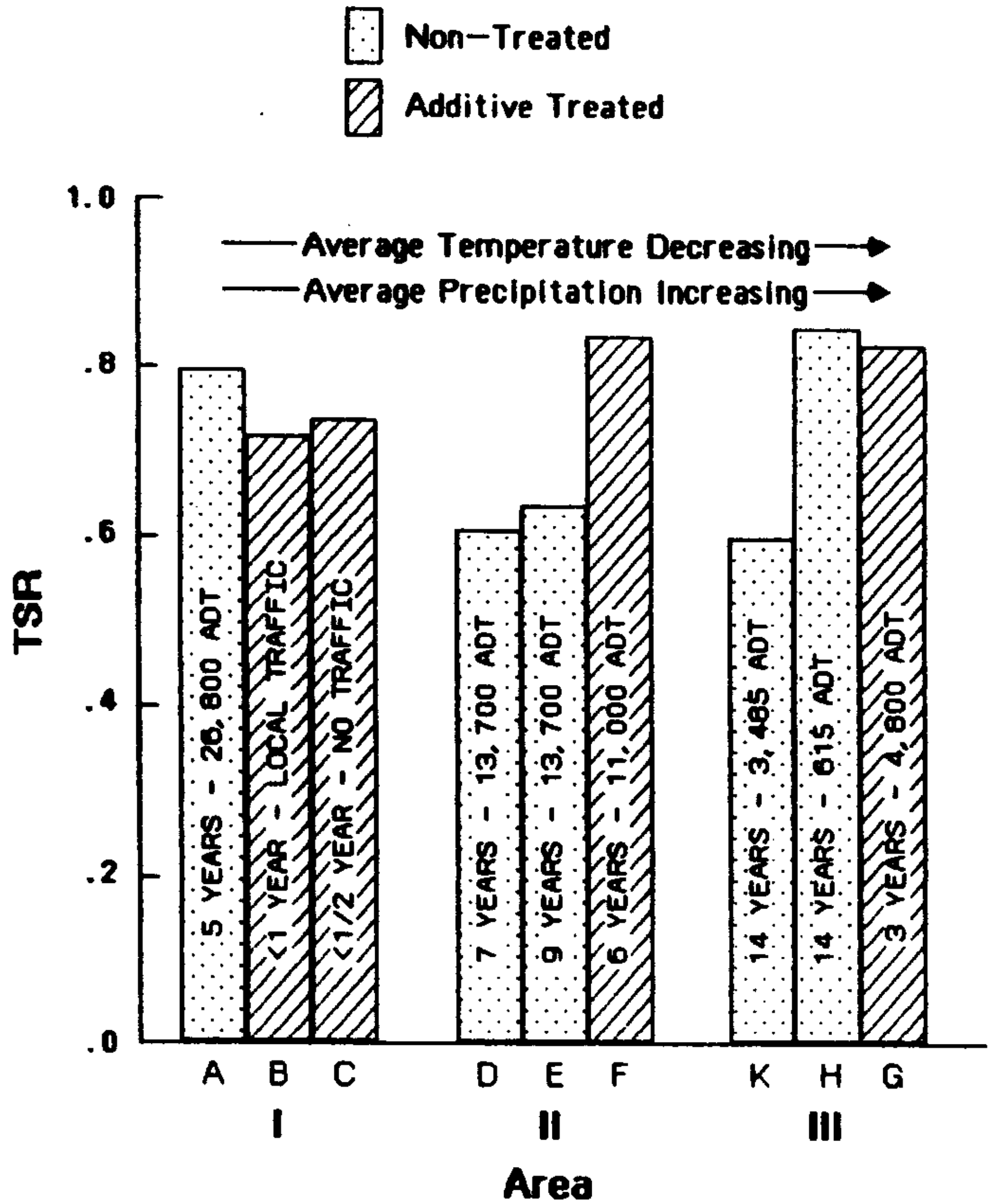
The calculation, which is included as Appendix F, utilizes data obtained from the project records. It was found that layer thicknesses of parts of the pavement structure are established by policy minimums and results in a total pavement structure which is considerably stronger than that required by the supporting subgrade soil strength. For that reason only that part of the traffic loading which was not resisted by the theoretical equivalent gravel section was used in the life prediction calculation. A separate calculation was made for each of the two roadway sections used in the project. The predicted life for Section C_1 is 6.2 years, that for Section C_2 is 9.3 years, compared to the ACMODAS LCO prediction of 9.1 years.

The predicted life for pavement C_2 compares quite favorably with the prediction from ACMODAS; the 6.2-year life predicted for the pavement C_1 is more conservative. The latter pavement has a thicker cross section of ACP due to higher projected traffic loadings. This fact alludes to a relatively greater potential loss of strength due to moisture damage. Other possibilities exist which could produce differences; prominent ones include the unknown effects of the absence of a definable regional factor in the WSDOT method and the effect of differences in the absolute strength of asphalt concrete when calculating the equivalent gravel depth. In addition, there is a tendency for emperical methods to have greater factors of safety to account for unexplainable differences in the base data from which those methods are developed. Based upon experience, the ACMODAS prediction is probably acceptable for the pavement in Project C.

Climatic Factor Analysis--Project sites were selected after analyzing differences in climatic factors to assess what effect temperature extremes or relative differences in precipitation might have on the performance of pavements with and without antistripping additives. This was the extension of the Lottman NCHRP study [1] which initially assumed that these environmental factors would tend to decrease the field time to reach predicted values. The study concluded that the effect could not be verified because of other variables such as different asphaltic concrete mixtures and traffic. With the exception of a short length of experimental "open-graded" mix placed on a shoulder in Project E which is reported separately in Table 2, all of the asphalt concrete mixtures represented by this study were manufactured using the same dense-graded specification. It is proposed that this commonality tends to reduce bias toward variations in asphaltic concrete mixtures for this research.

The Lottman NCHRP study proposed that heavy traffic appears to increase the rate of pavement damage more dramatically than extremes of precipitation and temperature. The graph, Figure 6, summarizes the data from Table 1 for the three significant climatic areas studied in the research. Data for the projects are arranged by "Area," in order of decreasing temperature and increasing precipitation; the traffic volumes associated with the various projects are also shown. The limited scope of this study did not allow for sufficient sampling to normalize all the effects of age and traffic volumes. And while a prediction model which utilized projects A, B and C as a measure for performance in other areas was not achieved, it is possible to make some observations from the graph. Those conclusions are as follows:

- Despite significant decreases in traffic volumes, there is a measurable decline in the untreated pavement serviceability in the cooler and wetter areas.



I. South Central Area (See site selection)

II. North to East Central Area

III. North to East Area

Figure 6 — Location Variables

- Considering only the additive-treated pavements in the cooler and wetter areas (II and III), Project G should be aged out of its dry stage and should tend to equalize itself with Project F. The TSR's for these pavements suggest no significant difference in serviceability despite significant differences in traffic volumes.
- When traffic volumes are relatively equal, additive-treated pavements performed better than non-treated pavements in Areas II and III. Non-treated, well-constructed pavements in Area I may perform equally as well as additive-treated pavements.
- Eighty-five percent of the total traffic volumes reported for the 4-lane project north of Spokane (Data Groups H and K) can be expected to use the outside lane represented by Group K cores [17]. Thus, the traffic volume for the inside (passing) lane is only approximately 600 vehicles per day. The TSR data for this lane (Group K) remains at a relatively high level for this old pavement and tends to indicate that a non-treated pavement could be used for low volume or low load situations. It should be expected to perform at a satisfactory level of serviceability for many years, even if the source of aggregates for the asphalt concrete was known to be somewhat strippable.

The micro-computer based asphalt concrete moisture damage analysis system (ACMODAS) [14] permits rapid determination of predicted pavement life in the dry plus wet environment based upon user selected variables. These variables include a "regional factor" to account for differences in traffic and/or climate (based on freezing degree days). Using the test data from indirect tension tests for Projects A and C, different values for field all-dry design life (FADDL), field dry-stage time (FDST) and the regional factor (RF) were used to calculate the field dry plus wet life (FDWL) using a ten percent allowable reduction of all-dry life (FADDL). Inspection of the results shown in Table 3 reveals only small gains or reductions in the FDWL from the FADDL for significant differences in the all-dry life and the dry-stage time.

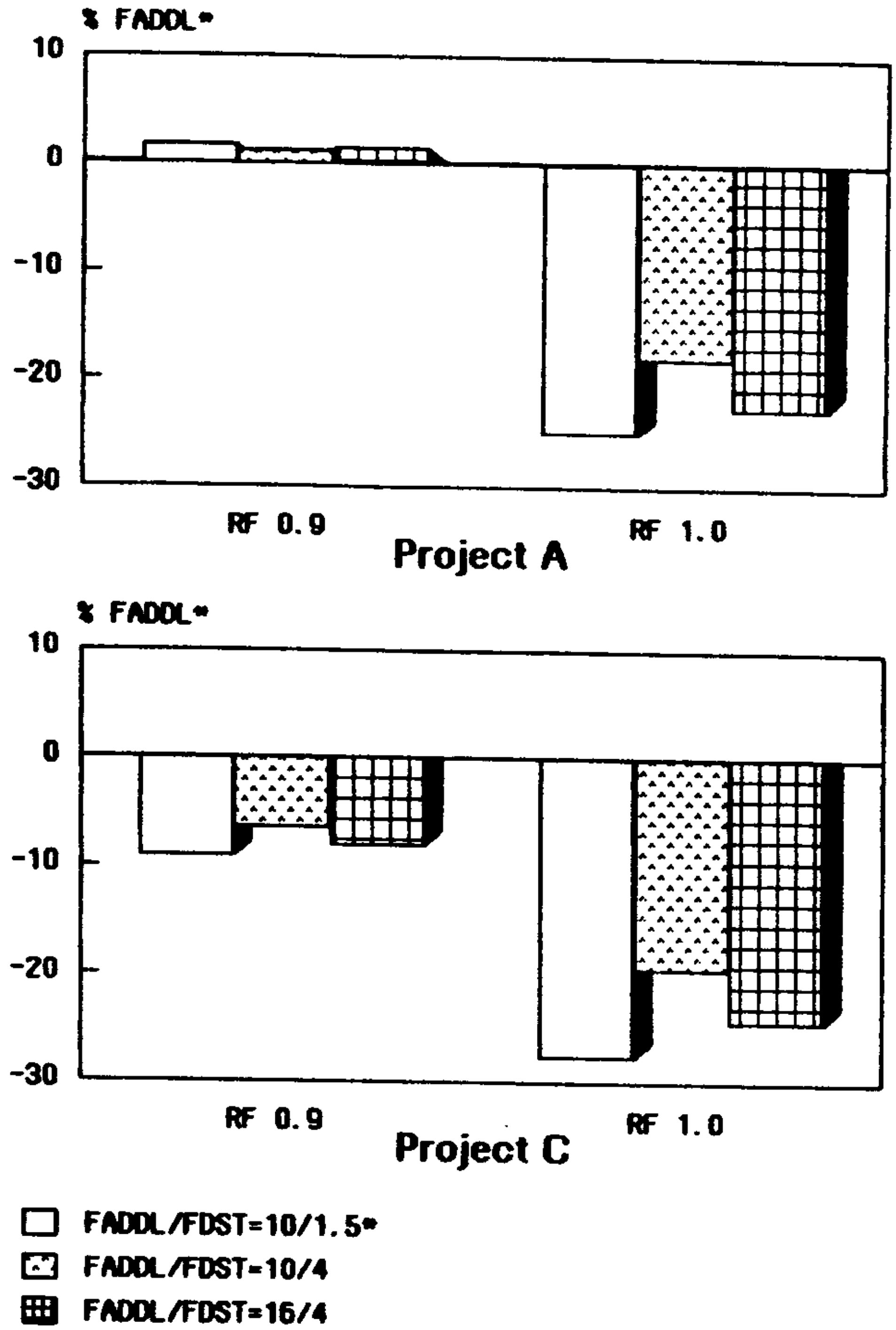
Such was not the case for changes in the regional factor which can represent differences in traffic or climate or both. By using the same data and recalculating using different regional factors, one can see a rather substantial reduction in predicted pavement life resulting from a change from a "moderate" to a "severe" climate for a given traffic level. The graph, Figure 7, vividly displays these effects.

Table 3--Effect of design life and climatic differences on predicted pavement life.

Project A							
FADDL	:	10	10	16	:	10	10	16
FDST	:	1.5	4.0	4.0	:	1.5	4.0	4.0
RF	:	0.9	0.9	0.9	:	1.0	1.0	1.0
FDWL	:	10.2	10.1	16.2	:	7.5	8.2	12.4
% Gain (Reduction)	:	1.7	1.2	1.5	:	(25.0)	(18.0)	(22.5)
TSR Cut-Off	:	.74	.72	.74	:	.85	.84	.85

Project C								
FADDL	:	10	10	16	:	10	10	16
FDST	:	1.5	4.0	4.0	:	1.5	4.0	4.0
RF	:	0.9	0.9	0.9	:	1.0	1.0	1.0
FDWL	:	9.1	9.4	14.7	:	7.3	8.1	12.1
% Gain (Reduction)	:	(9.1)	(6.4)	(8.0)	:	(27.4)	(19.4)	(24.2)
TSR Cut-Off	:	.70	.68	.69	:	.83	.80	.82

Key: FADDL - Field all-dry design life; FDST - Field dry stage time; RF - Regional Factor; FDWL - Field dry + wet life; TSR Cut-Off - Lab predicted minimum tensile strength ratio.



*See Table 3 for definition of terms.

Figure 7— Predicting pavement life.

CHAPTER 6

AGENCY QUESTIONNAIRE

In order to gain a current understanding of how others perceive the effectiveness of antistripping additive, a selected list of agencies has been contacted and asked to complete a questionnaire. These are agencies who have participated in NCHRP 4-8(3) [1], or in Dr. Lottman's subsequent research; the agencies also represent states identified by Tunnicliff and Root in NCHRP 10-17 [15] as users of "some, much or heavy" applications of antistripping additive(s).

The questionnaire was designed to assess additive types, pavement serviceability and additive system performance; it also inquired about desired requirements for future additives. Examples of the questionnaire used for the present survey will be found at APPENDIX G, and a summary of the responses is included herein as Table 4. The first part is an inquiry into the assessor's experience with the subject matter in order to qualify the responses of the second part with greater reliability. A study of the prediction of pavement distress was reported by Smith, et. al., in connection with NCHRP Project 9-4A [19]. In that study it was concluded that assessors be selected with: (a) a minimum of 15 years of experience in pavement engineering; (b) substantial experience observing the field performance of pavements; (c) familiarity with definitions of material properities; and (d) experience in construction over the past 10 years. This research had insufficient resources to mobilize the in-depth selection and training process used by Smith and

Table 4--Questionnaire data summary.

ITEM	A	B	C	D	E	F	G	H	I	J	K	L	M	Avg.
							AGENCY							
Qualifying Experience	28	30	20	5	8	30	29	20+	27	15	12	15	10	19.15
Verifiable Performance	y(1)	y	limit.	y(2)	limit.	?	y	y	n	y	(3)	(2)	y(4)	
Predominant Types														
Liquid A-S	many	y	y	y	y	y	rarely	y	many	y	y	y	y	
Lime (Slurry or Dry)	S&D	S	D	S&D	n	-	D(5)	S(6)	experm.	n	S&D	D	S	
Port. Cem. or flyash	PC	PC	-	-	-	-	-	experm.	-	-	flyash	PC	-	
Field service-life improved w/A-S Add.	y(7)	y	y(7)	?	y	y	y	y	y	y	y	too vari.	(8)	
System Performance			(N/R)				(9)					(N/R)		
Selection	3	3		3	3	3	5	4	3	4	4		2	3.2
Application	4	4		4	5	5	5	4	4	4	4		2	4.0
Dosing	3	4		3	4	3	5	3	4	3	3		2	3.2
Mixing	3	4		4	5	4	5	4	2	3	3		3	3.5
Testing	2	1		4	1	4	5	3	3	2	4		4	2.8
Verification	1	1		3	1	2	5	3	3	3	3		2	2.2

(1) Sketchy field data. (2) Lab evidence - lack evidence on field correlation. (3) Kept records. (4) Stripping failures assoc. w/LAA. (5) Dry with moisture added. (6) Marinated aggregate or slurry. (7) Usually. (8) Better with lime. (9) Rated for lime. (N/R) No response

his colleagues for their study. Instead, their criteria was incorporated into the questionnaire, and those who volunteered to respond to the current study were found to substantially meet the Smith criteria, i. e. their experience ranged from 5 years to 30 years, but 9 out of the 13 had 15 or more years for an average of 19.15 years.

The most significant facts emerging from the survey was that most felt they had verifiable data on anti-strip performance, and most felt that there was a measurable improvement in service life. Most had some to much experience with liquid antistripping additives, yet in some states there is significant use of lime (both as a slurry and as a powder) as well as some interest in the use of Portland cement powder. Dr. Lottman advises that lime is much more expensive than the liquids, and that its use predominates in areas having much worse of a stripping problem than do states in the Pacific Northwest. There is a basis to conclude that a test section (or several of them) should be established to evaluate performance and cost impacts along with liquid additive treatment(s) and no treatment using the same asphalt concrete mix from known sources of severely strippable aggregates.

Inspection of the data in Table 4 for liquid additive system performance (excluded are those who did not respond or show total bias toward non-liquid treatments) reveals that application and mixing performance is best; followed by selection, dosing and mixing; then testing and verification. Several respondents confirmed problems verifying and proportioning the material.

CHAPTER 7

DISCUSSION

Over-application of liquid antistripping additive is minimized by the cost of the additive and the way that the State pays for the product. Washington State DOT reimburses their contractors for the cost of the additive plus a handling charge for liquid antistripping additive which has been ordered into the asphalt cement and blended at the refinery terminal. The cost is reimbursed after the material is delivered to the asphalt concrete producer and after it is incorporated into the project paving. Thus, the project engineer can control the cost impact of using additives, and at the same time, he can avoid detrimental effects to the asphalt cement which over-application can bring about. Observed effects include alteration of the viscosity of the asphalt cement (sometimes outside specification limits) and a prolonged tenderness well after the pavement cools.

According to the study by Tunnicliff and Root [15], of the 43 agencies who reported they used procedures for determining the need for antistripping additive, only 18 agencies reported procedures to determine that the correct dosage is used on actual projects, but only one used the certification method. Certification, however, is the WSDOT specified method of assurance of conformity but it does not assure uniformity. An accurate assessment of the effectiveness of liquid antistripping additive must be based on correct application and use of the chemical in the field.

Tunnicliff and Root [15] reported two situations where in-line blending equipment was used to mix the additive with the asphalt cement; the systems were equipped with calibrated additive dispensers and could be checked regularly by inspectors. One area asphalt cement supplier who was visited by the author used in-line blending equipment for the first time for the 1984 paving season. It is not an automatically proportional metering system. The pump must be switched on manually and the operator must estimate how long to run the pump to deliver the product as uniformly as possible over 80 percent of the load (the desired standard). The possibility of over-application exists if the operator neglects to turn off the switch, but the risk of someone losing the tally while bucketing the material is minimized.

The WSDOT Materials Laboratory has begun investigating equipment to permit in-line blending at the asphalt plant where WSDOT inspectors could witness the application. This process would be similar to that which has been done routinely at plants which produce Portland cement concrete (with additives) for use on WSDOT projects. Until better tests can be developed to verify the presence and concentration of the additive, the calibrated and monitored in-line blending operation at the asphalt plant seems to be the most reliable approach to verification.

Taking the cue from those who claim they can detect certain additives by smell, perhaps a distinctive aromatic can be added by the additive makers if in-line blenders are not provided at the plant.

Some changes in the asphalt cement, aggregate, additive combination seem to be more important than others. The effect of a change in the source of asphalt cement may not be as critical as a change in the additive type. In a study reported by Maupin [4], the magnitude of TSR values for mixtures of different asphalt cements was similar at a 95 percent level of confidence. In contrast, it was found that there was a dramatic difference in performance for different types of antistripping additives. These findings are significant. After a proposed mix has been tested and a mix design recommended for a given aggregate source, the mix may not have to be retested each time a different asphalt cement is proposed for that source; but it must be retested each time a different additive is used.

Maupin's research shows that particular additives (brands and/or chemical types) may be required for particular aggregates. This theory was recently confirmed when a commercial asphalt concrete source east of Spokane was evaluated. For many years the gravel pit had been used as an approved aggregate source for the production of asphalt concrete. As testing improved, the need for moderate amounts of antistripping additive in the range of 0.25 to 0.50 percent was established. During the winter of 1983-1984 a routine retest of the pit material disclosed a sudden rise in antistripping additive demand to 1.50 (or 2.50) percent by weight of asphalt, depending on which of two sources of asphalt cement was being used. The WSDOT worked with the supplier to resolve the problem. As a result of the cooperative effort, the supplier selected another additive type by the same manufacturer. The effectiveness of the new additive again allowed reduced dosing in the 0.25 to 0.50 percent range instead of the former higher-cost 1.50/2.50 percent range. The WSDOT has suspended the use of the former additive with the designated asphalt cements and aggregates on all State contracts.

CHAPTER 8

CONCLUSIONS

For practical application in the field, future antistripping additives should be less sensitive to asphalt and aggregate changes. An ideal additive would be a generic material, but more importantly, it should be easily applied or incorporated at the asphalt concrete plant. It should give consistant improvements using existing technologies, and it should be cost effective. Certain conclusions were drawn from this research concerning the effectiveness of liquid antistripping additive (LAA).

- LAA appears to be satisfactory to meet the demands of locally available aggregates and asphalt cements, provided that proper care is given to their selection and application.
- Reliability and uniformity of application appear to be somewhat of a problem but it appears to be solvable.
- No overall, satisfactory test procedure for antistrip presence and concentration was discovered or tested which is applicable for field lab use.
- For some pavements (low volume and/or warm, dry environment) the use of the additive may be only marginally cost effective.
- LAA should be used for all pavements in the colder, wetter regions if the aggregate is at all strippable, and especially if one has existing, high traffic volumes.

- The indirect tensile strength test can be readily performed in a small, soils and materials laboratory equipped with a lower-range testing machine using the 51 mm/min (2 in./min) deformation rate at 25°C (77°F) [4,15]. The data can be quickly analyzed using now-available computer and/or numerical methods. The results can be utilized as a pavement management tool to monitor pavement distress, to verify that asphalt concrete production mixtures are performing as designed, and to provide reliable data on projected pavement life to resolve material quality disputes.

CHAPTER 9

RECOMMENDATIONS

This research indicates that the regional factor can be the significant influence in pavement design, but knowledge of the subject has not progressed much in the fifty years since asphalt stripping has been an issue. The research did indicate that climatical factors (especially precipitation combined with cooler temperatures) may play a larger role than was previously thought in the serviceability of anti-stripping additive treated versus non-treated asphaltic pavements. To validate this finding a larger, more-specific research project should be undertaken. Environmental monitoring equipment should be installed at each site in the study to effectively relate climate with performance. Sites should be choosen to normalize the traffic indices, and a section of non-additive-treated pavement should be included for control. Besides preparing lab-compacted specimens from the production mixes, cores should be taken for tensile split testing before the pavement freezes in the field. Additional cores should be taken on an annual basis for about five years, and then at ten years and at 15 years, if applicable. The results can be easily analyzed using ACMODAS [14].

While such a study might be undertaken using the diverse environments of both Western and Eastern Washington, a logical extension of the present study might first be in order to establish what are the regional design fectors for Eastern Washington. In addition, the literature abounds with case histories, and the questionnaires received from this study also suggest that hydrated lime as an aggregate additive or pretreatment should be evaluated to determine whether or not it will produce cost effective antistripping stratigies in asphalt concrete for Washington State.

If the practice continues to accept liquid antistripping additive as furnished from the refinery or terminal, other research is also indicated to develop a simple, but reliable field procedure to assess the presence and concentration of liquid antistripping additives.

During the past decade, with the evolution of the computer, there has been a significant shift away from full scale testing toward simulation of environmental and other effects. The simulation approach has applications, but the technologist must realize that the computer simulations are only as good as is the experience of those who program them. A more-basic "physical" research program should be undertaken to discover what environmental stripping mechanisms are at work and how to deal with them. The computer simulations could then be updated and refined with the experience gained from the physical environmental research.

Potentially strong resources and facilities do exist in the Pacific Northwest to undertake the suggested research. They are the research universities and their resident staffs. Institutions whose past accomplishments in asphalt technology are well documented in the literature include the University of Washington (UW), the University of Idaho (U of I), Oregon State University (OSU) and Washington State University (WSU). A large-scale research project should be implemented to investigate the combined effects of the environment using increased wheel loads and elevated tire pressures. An appropriate program could be undertaken as a collaborative effort by WSU and the U of I utilizing the Reidesel Test Track at WSU, augmented by the modern CE materials laboratories at the U of I. Renovating the test track by enclosing it and providing suitable atmospheric control equipment would create an ideal laboratory facility for full-scale testing. The facility would be similar to the FHWA Accelerated Loading Facility (ALF), but it would test the asphaltic pavements (with and without antistripping additives) under accelerated environmental conditions for the controlled study of an apparently widespread type of stripping damage found under this present research.

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APPENDIX

APPENDIX A

WASHINGTON STATE
DEPARTMENT OF TRANSPORTATION

JOB MIX DESIGN PROCEDURE

The Contractor should provide samples of aggregates from each stockpile he proposes to use along with the following data: average stockpile gradations of each stockpile, ratios of how he intends to combine stockpile aggregates, average gradation of this combined mix, brand of asphalt cement intended for use with the aggregate.

Upon receipt of the aggregate from the project office, the aggregate is dried and tested for sand equivalent and fracture count, then separated into specific sieve sizes for the specified class of ACP.

The aggregate is recombined to the Contractor's proposed final gradation and mixed with six varying percentages of asphalt cement (AR4000W) between 4.5 and 7.0 percent at one-half percent increments. The six briquets resulting from the compaction of these mixtures are tested for density, stability and cohesion. The results of these tests are used to aid in the determination of which percentage of the six most closely represents the criteria desired in an asphalt concrete pavement. The two briquets that are close to an ideal mixture ratio (2-4.5% voids and 35+ Stabilometer) are then further evaluated for rice density and Lottman stripping. If stripping is evident in this briquet, having no anti-strip optimum additive, then three new briquets are made at the optimum percentage of asphalt but with 1/4, 1/2, and 3/4 percent liquid anti-strip additive by weight of the asphalt cement. A visual evaluation of the Lottman stripping tests on these three briquets is used to determine the percentage of anti-strip necessary to prevent stripping.

The data as reported includes: average gradation; combining ratios, test results; percent of asphalt recommended; and percent of antistripping additive to be used.

APPENDIX B

WSDOT TEST METHOD NO. 718

METHOD OF TEST FOR STRIPPING OF ASPHALT CONCRETE

1. SCOPE

This test is used to determine the amount of stripping resulting from the action of water on laboratory-compacted asphalt concrete mixtures. Specimens shall be compacted as per WSDOT Method No. 702.

2. EQUIPMENT

(A) Water bath controlled at 60° ± 0.5°C (140° ± 1°F).

(B) Dessicator capable of holding a vacuum (26MM Hg) and large enough to accommodate test specimens and volume of water as described in the procedure.

(C) Perforated platform to hold test specimens off the bottom of the dessicator to assure complete saturation.

(D) Vacuum pump, vacuum system or water aspirator, for vacuum-saturation of specimens.

(E) A conventional air-bath freezer.

(F) Transfer dish—(approximate dimensions) 120mm dia. by 65mm ht. (4.7 in dia. x 2.6 in ht.) pyrex dish.

3. PROCEDURE

(A) Fill dessicator with distilled water, at room temperature, to a level at least 25.4mm (1 in.) above and 50.8mm (2 in.) below specimen level.

(B) Place specimens in dessicator for vacuum saturation.

(C) Apply a minimum vacuum of 26mm Hg to the dessicator for 30 minutes, during which time the dessicator should be agitated at 5 minute intervals.

(D) After 30 minutes of vacuum, remove specimen and place in plastic bag. Fold top of plastic bag over in such a way that the specimen is effectively sealed.

(E) Place specimen in a conventional air-bath freezer for 15 hours at temperature -6.7° ± 5.5°C (20° ± 10°F).

(F) After freezing for 15 hours, remove specimen from freezer, remove from plastic bag and place in a transfer dish. Place transfer dish containing the specimen in a 60° ± 0.5°C (140° ± 1°F) water bath for 24 hours.

(G) Remove from bath after 24 hours, break in half and inspect specimen for signs of stripping. Record condition of specimen as no stripping, slight, moderate or extreme stripping.

SLIGHT STRIPPING: The sample condition is solid to slightly soft with evidence of the asphalt cement beginning to withdraw from the edges and surfaces of the aggregates. After the sample has air-dried, the appearance remains black.

MODERATE STRIPPING: The sample condition is soft, easily broken in half, with partial to completely exposed aggregates. After the sample has air-dried, the appearance will be slightly gray.

EXTREME STRIPPING: The sample condition is soft to falling apart with the majority of coarse aggregate completely exposed and asphalt cement almost non-existent. After the sample has air-dried, the appearance is gray.

NO STRIPPING: The sample condition is solid with no evidence of asphalt cement withdrawing from the aggregate. After the sample has air-dried, the appearance is black.

APPENDIX C

CROSS REFERENCE TO WSDOT PROJECTS

REFERENCED IN TEXT

Project Ident. Group	Year Paved	State Route No.	State Contract No.	Section
A.	1979	14	1324	Kennewick I/C to West 4th Avenue
B.	1983	182	2236	Columbia River to Jct. SR 395 (Gr)
C.	1984	182	2495	" " " " " " (Pave)
D.	1977	2	0700	Peshastin to Monitor
E.	1975	2	9966	Monitor to Sunnyslope I/C
F.	1978	2	0839	Sunnyslope I/C to Columbia River
G.	1981	2	2122	Milan Rd. to Pend Oreille Co. Line
H.	1970	2	8602	Spokane Co. Line to Jct. SR 211
K.	"	"	"	" " " " " " "

APPENDIX D

LOTTMAN TEST PROCEDURE FOR INDIRECT TENSION TESTS

Step A. LABEL SPECIMENS--Choose an identification system for sample cores. The system should be designed to allow for the orderly cataloging, testing and tracking of specimens as well as facilitate the systematic analysis of data. The first character of the system utilized in this research identifies the various projects which were sampled, while the second orders the sample set within the project group. The letter "I" was avoided so as not to conflict with possible use of Roman numerals for other purposes; the letter "J" was reserved but not used. Use of a China marking pencil or a paint stick marker is recommended so as to preserve the sample's marks throughout the various soakings in water.

Step B. MEASUREMENTS--Measure each core by an appropriate standard method. Data is recorded using an form such as the one appended hereto (see TAB A). The thickness and diameter of each core sample was measured and recorded. The diameter of each core was checked for reasonably close conformity to 4 inches; that figure was used in the calculations.

Step C. WEIGH EACH CORE--The cores were separated into wet and dry test sets. The lowest numbered (or lettered) samples were for dry tests, the next series were for wet tests and the highest numbers represented those for freeze-thaw preconditioning. Specimens were allowed to dry to ambient air dryness before initial weighing. Then those for dry testing and those for freeze-thaw preconditioning were placed in heavy walled desiccating jars having lids fitted with sleeves for drawing and maintaining a partial vacuum.

Step D. DESICCATING CORES--The cores to be dried were subjected to a partial vacuum of 4 inches Hg at room temperature ambient and allowed to desiccate moisture from the permeable voids into the drying medium. Each day the samples were removed from the jars and weighed to the nearest 0.1 gm and then returned to the dessicators. The process continued until constant weight was achieved.

Step E. SATURATING CORES--While the dry cores were drying, those slated for wet testing were saturated with distilled water. A battery of five heavy walled bowls (like those used for maximum, or Rice density) were used to saturate each of five samples at room temperature under a vacuum of 26 inches. After 12 minutes the vacuum was removed and the cores were allowed to soak in the water for another 12 minutes at ambient indoor pressure and temperature. Following the soak, the cores were placed into separate canisters filled with distilled water and placed in the environmental chamber. The temperature in the chamber was set at 55°F. The cores remained in the chamber for 24 hours to ensure the entire specimen was at 55°F before mechanical testing. (The procedure for saturating accelerated conditioned cores is slightly different; see Step G.)

Step F. DIAMETRAL SPLIT TENSILE TESTING--A standard variable rate testing machine with a 0 - 30,000 lb low range dial was used for the required testing. The vertical rate was checked for each set of tests to ensure the required rate of 0.065 inches per minute.

SATURATED (WET) CORES--The wet cores remained in the environmental chamber until just before the test was started. The test core was removed from the can and promptly pressed on diametral sides of the core using aluminum pillow blocks (see photo, Figure D-1).

DRY CORES--After drying selected cores to constant weight, they are placed into the environmental chamber at 55°F for a minimum of six hours. Thereafter, these cores were pressed on their sides similar to their "wet" counterparts.

The tensile split test is run on the specimens as quickly as possible to ensure the temperature of the core remained as close to 55^{O}F as possible. The testing speed is very sensitive to the core temperature for proper stripping evaluation. Times in the testing machine to achieve failure were from one to two minutes.

Step G. ACCELERATED CONDITIONING--Some of the cores required an accelerated conditioned test. This procedure required that all the cores (wet and dry) be dried to a constant weight. The dry specimens went through the same procedure as the other dry cores; the wet (saturated) cores went through a somewhat different process.

The "wet" cores were dried to a constant weight and weighed. These specimens were then individually saturated using the procedure in Step E., above. After materation, the core was again weighed and the weight recorded. It was then wrapped in clear plastic wrap material and sealed by taping completely with masking tape. The wrapped core was then bagged in a plastic bag and placed in a freezer for 24 hours. After the 24 hours, the sample was completely unwrapped and placed into a canister partially filled with distilled water. The can was then placed into a 140^{O}F hot water bath for 24 hours. After the hot water bath, specimen was placed into the environmental chamber for 24 hours. Cores thus conditioned were then tested for diametral tensile strength as described above in Step F.

APPENDIX E

DETECTION PROCEDURE FOR POLYAMINE ADDITIVES IN ASPHALT CONCRETE [11]

(for siliceous aggregates)

Step A. Add 50 grams of loose mix asphalt concrete, 50 grams of asphalt solvent and 10 ml of extraction solution to a 250 ml separatory funnel.

B. Mix these ingredients thoroughly by gentle shaking for 5-7 minutes. Be sure to vent the separatory funnel occasionally. Allow the layers to separate for 20 - 30 minutes.

C. Drain the bottom layer (aqueous phase) from the funnel into a two-ounce glass bottle. Allow any aggregate particles removed during this step to settle to the bottom of the bottle.

D. Transfer 5.0 ml of the aqueous solution from Step C. into a second two-ounce glass bottle. Adjust the water solution in this second bottle to pH = 11 with 50 percent sodium hydroxide solution (a slight warming may occur). Add 5.0 ml n-butyl alcohol and mix for one minute.

Step E. Check the pH of the top and bottom layers resulting in Step D:

top layer	pH = 7.5-8.5
bottom layer	pH = 11

If the bottom layer is not at pH = 11, then adjust accordingly with additional amounts of 50 percent sodium hydroxide solution.

F. Add eight drops of detector solution to the Step E. solution. Mix this combination for one minute and then recheck the pH of each layer. Maintain the bottom layer at pH = 11. If polyamine additive is present, an intense reddish coloration will appear in the top layer. In the absence of polyamine additive the top layer will be light yellow in appearance.

G. Remove 2.0 ml of the top layer from Step F. and dilute with 2.0 ml of n-butyl alcohol. Adjust the pH to 10-11, using several drops of 50 percent sodium hydroxide solution. If the solution develops a hazy appearance, allow the sample to stand undisturbed for 20-30 minutes before comparing to the known standards. Visually compare the intensity of color to known standards within two hours in order to determine the amount of polyamine additive present in the asphalt concrete. The known standards should be generated with the specific aggregate, asphalt and additive used in the construction project.

APPENDIX F

PROJECT C - PAVEMENT LIFE PREDICTION

The following analysis is based on the WSDOT pavement design method together with Miner's cumulative fatigue damage rule. General comments on the application of this analysis can be found under the heading, **Prediction Model**. A 10-year field all-dry design life and a 1.5-year field dry-stage time has assumed.

Project Data

Roadway Section C_1 (Associated with core samples C-2 & C-8)

0.25 ft	(3.0 in.)	AC Pavement
0.35 ft	(4.2 in.)	AC Base
0.25 ft	(3.0 in.)	Crushed Surfacing
0.40 ft	(4.8 in.)	Ballast
1.25 ft	(15.0 in.)	Total Pavement Structure

Subgrade Soils Stabilometer, R = 58

Maximum Traffic Index, TI = 6.0

Roadway Section C_2 (Associated with core samples C-1 & C-7)

0.30 ft	(3.6 in.)	AC Pavement
0.25 ft	(3.0 in.)	Crushed Surfacing
0.70 ft	(8.4 in.)	Ballast
1.25 ft	(15.0 in.)	Total Pavement Structure

Subgrade Soils Stabilometer, R = 58

Maximum Traffic Index, TI = 5.6

<u>Laboratory Test Data</u> (Average of two core sets)

Dry Indirect Tensile Strength (I-T-S) @ 55°F, S_{t-dry} = 95.8 psi

Wet Accelerated Conditioned I-T-S @ 55°F, S_{t-wet} = 68.1 psi

Tensile Strength Ratio, TRS = 68.1 / 95.8 = 0.71

<u>Pavement Life Prediction - Roadway Section C_1</u>

1. Assuming the asphalt is "dry," the gravel equivalency for 7.2 in. of asphalt concrete is:

$$GE_{ac} = t_{ac}\ (\ 3 * t_{ac}\ )^{0.20} = 13.31 \text{ in.}$$

where: t_{ac} = 7.2 in. of undamaged asphalt concrete

Assuming 7.8 in. of crushed surfacing and ballast has a gravel equivalency of 7.80 in., then the total "dry" gravel equivalency is:

$$GE_{tot.,\ dry} = 13.31 \text{ in.} + 7.80 \text{ in.} = 21.11 \text{ in.}$$

The R-value of subgrade soil (as tested) is R = 58. This is compatible with the following design relationship:

From $GE_{tot.} = (\ TI + 0.1\ )(\ 5.9 - 0.047\ R\ ) - 9.0$,

$$TI_{dry} = \frac{GE_{tot.} + 9.0}{5.9 - 0.047\ R} - 0.1 = \frac{21.11 + 9.0}{5.9 - 0.047(58)} - 0.1 = 9.38$$

Note: $TI_{capacity}$ (9.38) is greater than TI_{design} (6.0). Therefore, only the R-value$_{req'd}$ will be used for life predictions (see text).

For TI_{des} = 6.0, the 10-year design, 5 kip Equivalent Wheel Loads (EWL) = antilog 6.0 = 1,000,000

From the design relationship above:

$$R_{req'd} = 125.53 - \frac{GE + 9.0}{0.047\ (\ TI + 0.1\ )} = 20.48 < 58, \text{ therefore OK}$$

2. Assume that the accelerated conditioned tensile strength ratio (TSR) is equal to the retained long-term asphalt concrete cohesion and that this retained cohesion is proportional to the added gravel equivalency for asphalt concrete above its actual thickness. Thus, the long-term wet gravel equivalency of asphalt concrete is:

$$GE_{ac,\ wet} = (\ 13.31 - 7.2\ )\ 0.71 + 7.2 = 11.54 \text{ in.}$$

The total "wet" gravel equivalency is:

$$GE_{tot.,\ wet} = 11.54 + 7.80 = 19.34 \text{ in.}$$

3. The basic (maximum allowable) traffic for the wet condition is calculated using the general relationship for TI; it is:

$$TI_{wet} = \frac{GE_{tot.,\ wet} + 9.0}{5.9 - 0.047\ R} = \frac{19.34 + 9.0}{5.9 - 0.047(20.48)} = 5.74$$

5 kip EWL = antilog 5.74 = 549,300

4. Assume that the pavement moisture conditions consist of 1.5 years dry followed by the remaining years in the wet condition, and that Miner's cumulative fatigue life rule applies as:

$$\frac{1.5 \text{ yrs dry } (100{,}000 \text{ EWL/yr})}{10 \text{ yrs } (100{,}000 \text{ EWL/yr})} + \frac{T_{wet}\ (100{,}000 \text{ EWL/yr})}{549{,}300 \text{ EWL}} = 1.0$$

$$\text{The wet life, } T_{wet} = (1.0 - 0.15)\ \frac{549{,}300}{100{,}000} = 4.67 \text{ years}$$

5. The total predicted life for Roadway Section C_1 equals:

1.5 yrs + 4.7 yrs = 6.2 years

<u>Pavement Life Predictions - Roadway Section C_2</u>

1. The gravel equivalency of 3.6 in. of "dry" asphalt concrete:

$$GE_{ac} = 3.6 (3 * 3.6)^{0.20} = 5.79 \text{ in.}$$

The total "dry" gravel equivalency is:

$$GE_{tot., dry} = 5.79 + 11.40 = 16.56 \text{ in.}$$

The Traffic Index and 5kEWL capacity of the dry pavement is:

$$TI_{dry} = \frac{17.19 + 9.0}{5.9 - 0.047(58)} = 8.25 > 5.6 \text{ [Use 5.6]}$$

Ten year design, 5kEWL = antilog 5.6 = 398,100

$$R_{req'd} = 125.53 - \frac{17.19 + 9.0}{0.047 (5.6 + 0.1)} = 27.77 < 58, \text{ OK}$$

2. The total "wet" gravel equivalency is:

$$GE_{ac, wet} = (5.79 - 3.6) 0.71 + 3.6 = 5.16 \text{ in.}$$

$$GE_{tot., wet} = 5.16 + 11.40 = 16.56 \text{ in.}$$

3. The allowable traffic for the wet condition is:

$$TI_{wet} = \frac{16.56 + 9.0}{5.9 - 0.047(27.77)} = 5.56; \qquad 5 \text{ kip } EWL_{wet} = 365,500$$

4. Applying Miner's cumulative fatigue rule:

$$\frac{1.5 (39,800)}{10 (39,800)} + \frac{T_{wet} (39,800)}{365,500} = 1, \qquad T_{wet} = 7.8 \text{ years}$$

5. The total predicted life for Roadway Section C_2 is:

$$1.5 + 7.8 = 9.3 \text{ years}$$

APPENDIX G

QUESTIONNAIRE SAMPLES

"Evaluating the Effectiveness of Liquid Antistripping Additive"

Responding Agency ______________________________

Person Completing) Name ______________________________
Form (Assessor))

Title ______________________________

Office Address ______________________________

____

____
City State Zip

Office Telephone (______)______________________

Qualifications of Assessor

Desired (not mandarory) Actual

1. Fifteen years of experience with pavement engineering. ______ years
2. Substantial experience observing field performance of pavements, including occurrence of stripping failures. ______ y,n
3. Familiarity with material property definitions and specification requirements. ______ y,n
4. Experiences with construction in the recent past. ______ y,n
5. Familiarity with pavement design procedures. ______ y,n

Comments on Qualifications of Assessor

____

1. Do have verifiable data on anti-strip performance?(y,n)____________

2. Predominant type used:

   Liquid:____________________________________ (type or brand)

   Lime Treatment:____________________________ (application form)

   Other:_____________________________________ (specify)

3. Field Serviceability Life of Pavements:

   Is it better with anti-strip?(y,n)_______

   Any identified problems?___________________________________

   ____

4. What is your definition of serviceability? (use back of sheet)

5. Overall system performance: (rate each area)

Parameter	(VG)	Rating	(P)
Selection	____,____,____,____,____		
Application	____,____,____,____,____		
Dosing	____,____,____,____,____		
Mixing	____,____,____,____,____		
Testing	____,____,____,____,____		
Verification	____,____,____,____,____		

6. Specific problems of non-performance: (indicate method)

   Unable to verify additive presence?__________________________

   Unable to verify right proportions?__________________________

   Unable to verify right material used?________________________

7. Recommendations: (use back of sheet or attach sheets)

   Future serviceability of anti-strip additives.

   Form, method and verification of application.

   Other system requirements. (Relate to No. 5, above)