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PROJECT ADMINISTRATION DATA SHEET

[x] ORIGINAL [ ] REVISION NO. ______

ct No. E-20-659 — DATE 7/28/82

ct Director: Dr. Richard D. Barksdale — School/~~Lab~~ Civil Engineering

sor: Georgia Department of Transportation

Agreement: Task Order No. 1-2 (see below)

d Period: From 8/15/82 To 5/14/83 (Performance) -------- (Reports)

sor Amount: $28,022 — Contracted through:

Sharing: $5,462 (E-20-334) — GTRI/~~GIT~~

: Fatigue and Rutting Study of Lime Treated and Rubber Asphalt Concrete

MINISTRATIVE DATA — OCA Contact Faith G. Costello

ponsor Technical Contact:	2) Sponsor Admin/Contractual Matters:
outer Gulden, P.E., Chief	Hugh L. Tyner, Chief
avement and Physical Research Branch	Research & Development Bureau
a. Department of Transportation	Department of Transportation; State of GA.
ffice of Materials & Research	Office of Materials & Research
5 Kennedy Drive	15 Kennedy Drive
orest Park, GA 30050	Forest Park, GA 30050

ense Priority Rating: N/A — Security Classification: N/A

STRICTIONS

Attached Gov't Supplemental Information Sheet for Additional Requirements.

el: Foreign travel must have prior approval — Contact OCA in each case. Domestic travel requires sponsor approval where total will exceed ~~greater of $500 or 125% of approved~~ approved proposal budget category.

ipment: Title vests with Sponsor

MMENTS:

der Basic Cost Reimbursement (no fee) Research & Development Contract dated 3/1/81.

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5/SR851

SPONSORED PROJECT TERMINATION/CLOSEOUT SHEET

Date 7/15/88

roject No. E-20-659-R5972-0A0 School/~~Lab~~ CE

ncludes Subproject No.(s) N/A

roject Director(s) Richard Barksdale GTRC/~~GIT~~

ponsor Georgia Department of Transportation

:itle Fatigue and Rutting Study of Lime Treated and Rubber Asphalt Concrete

Effective Completion Date: 5/14/83 (Performance) 5/14/83 (Reports)

Grant/Contract Closeout Actions Remaining:

- [ ] None
- [XX] Final Invoice or Copy of Last Invoice Serving as Final w/certification
- [ ] Release and Assignment
- [ ] Final Report of Inventions and/or Subcontract: Patent and Subcontract Questionnaire sent to Project Director [ ]
- [ ] Govt. Property Inventory & Related Certificate
- [ ] Classified Material Certificate
- [ ] Other

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15 Kennedy Drive
Forest Park, Georgia 30050-2599

February 22, 1988

Mr. Brian Lindberg
Georgia Tech Research Corporation
Georgia Institute of Technology
Atlanta, GA 30332

Dear Mr. Lindberg:

Subject: GDOT Research Project No. 7603, "Fatigue and Rutting Study of Lime Treated and Rubber Asphalt Concrete"

This letter is sent at your request confirming the close out of the subject project. This close out confirms the acceptance of the subject final report dated November 1987 by Dr. Richard D. Barksdale. Our acceptance also confirms that no further reports are required to be delivered to the GDOT and that no further payments are required to be made to the Georgia Institute of Technology.

Sincerely,

Peter Malphurs
State Materials and Research Engineer

PM:SFV:cgm

Contract Research

GaDOT Research Project No. 7603
(Task Order No. 1-2)

FINAL REPORT

FATIGUE AND RUTTING STUDY OF LIME TREATED AND RUBBER ASPHALT CONCRETE

by

Richard D. Barksdale
Professor of Civil Engineering
School of Civil Engineering
Georgia Institute of Technology
Atlanta, Georgia 30332

Prepared for

Department of Transportation
State of Georgia

November, 1987

SCEGIT-83-112

ABSTRACT

Fatigue and rutting tests were performed on asphalt concrete surface E-mixes prepared using (1) no additive, (2) 0.5% AS-1, (3) 1% dry lime, (4) 2% dry lime, (5) 1% lime slurry, (6) 3% latex, (7) 20% granulated rubber, and (8) 30% Trinidad Lake asphalt; a recycle mix was also tested. The aggregate used in the mixes (with the exception of the recycle mix) was from Kennesaw quarry, which has a known history of stripping problems. The GHD boil best, tensile splitting, and Lottman rutting tests were also performed by the Georgia DOT on similar type mixes.

Due to the presence of a large but variable amount of weak aggregate, the main series of fatigue and rutting tests showed a large amount of scatter in test results. Supplementary fatigue tests were therefore performed on mixes having 1% dry lime, 2% dry lime, 1% lime slurry and 0.5% AS-1. The supplementary fatigue tests were performed on moderately preconditioned specimens.

The fatigue tests indicate that the recycle mix should perform quite well with respect to fatigue; very little weak aggregate was present in this mix. All three tests (fatigue, tensile splitting and Boil test) indicated the 2% dry lime mix to have the best performance of the additives. The supplementary beam fatigue tests indicated that a 1% dry lime E-mix should perform in fatigue better than a 0.5% Indulin AS-1 mix. The beam fatigue test was concluded to be a sensitive indicator of potential mix problems such as weak aggregate and the presence of uncoated biotite mica flakes. Finally, specimen stiffness in the tensile splitting test or Marshall test is probably a better indicator of potential fatigue behavior than the failure load.

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ACKNOWLEDGEMENTS

The author would like to express his sincere appreciation to the Office of Materials and Research, Georgia Department of Transportation, for their fine cooperation which helped to make possible the success of this research project. Acknowledgement is given to Mr. W. T. Stapler for his cooperation and general guidance in conducting this project and to Mr. Wouter Gulden who was contract manager. Special thanks is extended to Mr. Ronald Collins, Mr. Harold Linnenkohl, and Mr. Gene Slaughter for their continued assistance throughout the project, and in specific for performing the indirect tensile tests, boiling tests, and seiving the aggregates.

Special acknowledgement is given to Mr. Ken Thomas who performed the laboratory tests, and to Nick Syriopoulos for reducing the data. Finally, appropriate acknowledgement is given to Mrs. Vicki Clopton for carefully typing the manuscript.

CHAPTER I

INTRODUCTION

Fatigue cracking is probably the most important structural problem experienced by pavements in the United States. Most materials engineers now recognize this fact, and understand the importance of designing an asphalt concrete mix having a sufficient level of fatigue resistance [1]. The generally accepted method of evaluating the fatigue resistance of a mix is by applying in the laboratory a cyclic load to asphalt concrete specimens. A number of different tests have been used in fatigue testing [1]. The beam and diametral tests are, however, probably the most commonly used tests in the United States to evaluate fatigue life.

One of the most critical properties of an asphalt concrete mix with respect to durability and fatigue performance is the bond between the asphalt and aggregate. Stripping (loss of bond) at the asphalt-aggregate interface may occur with time as the asphalt concrete is subjected to both loading cycles and environmental weathering. Stripping is accompanied by a general loss of integrity of the mix including fatigue cracking, raveling and general disintegration of the asphalt concrete. Of importance is the fact that the stripping phenomena is associated with both repeated load cycles and also environmental weathering including the effects of moisture. Important stripping problems have been found to occur in some asphalt concrete mixes having the silicaceous aggregates found in the Piedmont Province of Georgia [20].

Attempts have been made for many years to improve the bond characteristics between the aggregate and asphalt [1]. As the price of petroleum products continue to increase, undoubtedly considerably more interest will be shown in the future in developing new techniques for altering the asphalt-aggregate bond to improve the performance of asphalt mixes. Systematic, scientific studies of the effect of additives on fatigue strength and rutting have, for the most part, been lacking in the past [3].

OBJECTIVE OF STUDY

The objective of this study is to determine in the laboratory the effects of adding selected additives on the fatigue and rutting characteristics of an asphalt concrete E mix. The fatigue and rutting properties are compared to those for a standard mix having 0.5% Indulin AS-1 anti-strip additive. The following additives were used in the study:

1. Dry lime
2. Lime slurry treated aggregate
3. Emulsified latex rubber
4. Granulated rubber
5. Trinidad Lake asphalt

In addition, a recycled asphalt concrete mix from I-85, and an asphalt concrete mix without additives were also evaluated.

RESEARCH PLAN

Fatigue and rutting tests were performed on asphalt concrete specimens prepared using the mixes and specimen conditioning summarized in Table 1. An asphalt concrete E-mix having AC-30 asphalt cement was used in most of the mixes studied. The exceptions were the recycled mix which used AC-10, and the granulated rubber mix which used AC-20 asphalt cement. The granite gneiss aggregate used throughout the study was from the Kennesaw

TABLE 1. SUMMARY OF TESTS PERFORMED ON ASPHALT CONCRETE E MIXES AT 7 PERCENT AIR VOIDS.

Additive/Mix	Properties @ 7% Air Voids				Tests Performed(1)			
	γ (pcf)	AC	Stability (lbs.)	Flow	Fatigue	Rutting	Indirect Tension	Boil
E-Plain AC-30	148.3	5.8	1040	10.7	MP	-	S	S
E-0.5% Indulin AS-1 AC-30	147.9	5.9	985	11.5	NP/LP	NP/LP	S	S
E-1% Dry Lime AC-30	148.2	5.7	1280	11.0	NP/LP/MP	NP/LP	S	S
E-2% Dry Lime AC-30	148.2	5.5	1320	11.7	NP/LP	NP	S	S
E-1% Lime Slurry AC-30	147.9	5.8	1200	11.8	NP/LP	NP	S	S
E-3% Latex 97% AC-30	147.9	5.8	1420	11.0	NP/LP	NP	S	S
E-20% Granulated Rubber 80% AC-20	142.5	6.8	1260	11.6	NP/LP	NP	S	S
E-I-85 Recycle Mix AC-10	146.6	5	~1400-1500	~ 6.5	NP/LP	NP	S	S
E-Trinidad Lake AC-30	148.5	6.1	1560	11.4	LP/MP	NP	S	S

1. Notation Used: NP = no preconditioning; LP = light preconditioning - just partial vacuum saturation/soak; MP = mild preconditioning - partial saturation, freeze, soak at 140°F (60°C); S = standard CDOT test.

Quarry of Vulcan Materials Company. This aggregate has a Group II classification and hence is silicaceous. It is known to have stripping problems, and has a mica content of about 20 percent.

The fatigue and rutting tests were performed on specimens in both the as-prepared condition and also on specimens preconditioned by partial vacuum saturation. All specimens were prepared at a 7 percent air voids content to permit easy penetration of water during preconditioning. In summary, preconditioning consisted of placing the specimens inside a vacuum chamber, and applying for 30 minutes a predetermined vacuum sufficient to cause 80 percent saturation. After vacuum saturation the specimens were allowed to soak for an additional 30 minutes. In a limited test series, fatigue tests were also performed on beams more severely preconditioned using vacuum saturation, freezing at 0^{o}F (-17.7^{o}C) and soaking at 120^{o}F (48.9^{o}C). Beams without any additives, with 1 percent dry lime and with 30 percent Trinidad Lake asphalt were tested in this manner.

The fatigue tests were performed on 3 in. by 3 in. by 20 in. (7.6 x 7.6 x 50.8 mm) long asphalt concrete beams placed on a rubber pad simulating base and subgrade support. The fatigue tests were carried out to failure at a temperature of 78 to 80^{o}F ($25.6 - 26.7^{o}$C) in an environmental chamber. Rutting tests were conducted on 4 in. diameter by 10 in. high (10.2 x 25.4 mm) specimens at 95^{o}F (35^{o}C). These tests were carried out to 100,000 load repetitions.

Indirect diametrical tension tests were also conducted on preconditioned specimens. The fatigue test results were compared with the results from the indirect tension test.

CHAPTER II

BACKGROUND

INTRODUCTION

The bonding of asphalt to aggregate and the resistance of the resulting bond to water is dependent upon a number of factors including: (1) properties of the aggregate including mineral composition, surface chemistry, aggregate shape, surface texture, dust coating and surface abrasion, (2) composition and viscosity of the asphalt, and (3) the resulting properties of the mixture including its permeability. The combined effect of debonding and fracture in the asphalt and aggregate for conventional mixes, as defined by a general fatigue failure, has been extensively studied for example by Barksdale [1], Monismith and Deacon [4], Deacon and Monismith [5], and Pell [6].

Unfortunately in the past the effect of adding additives to improve fatigue life has received relatively little attention. The important effect of water on debonding has been reasonably extensively studied for example by Rice [7], Majidzadeh and Brovold [8], Chehovits and Anderson [2], Lottman [9], Kennedy, et al. [10,11], and Maupin [12,13]. The indirect tensile strength and resilient modulus has frequently been used in these studies to evaluate stripping.

STRIPPING IN ASPHALT CONCRETE

Stripping, or debonding as it is sometimes called, is an adhesive failure at the interface between the aggregate and asphalt resulting in a physical separation. Silaceous aggregates are known to often have stripping problems [10,14].

Stripping problems are usually associated with moisture. In fact, water can displace the asphalt film at the asphalt-aggregate interface. If the source of moisture is removed, the debonded interfaces have even been observed to heal when not under load [14].

Silaceous aggregates love water, i.e., are hydrophilic, because of their acid surface chemistry. Carbonate rocks such as limestone usually do not have severe stripping problems. These rocks have a basic surface chemistry and hate water (i.e., are hydrophobic). Some stripping problems have, however, also been observed in limestone [15].
This should not be too surprising since stripping is simply a loss of bond, and can be caused by numerous factors.

The loss of bond can lead to general disintegration of the pavement including cracking, potholes and rutting [11,12,13,15]. If the pavement is subjected to light traffic loadings, however, deterioration of the pavement may not progress too far, even if stripping has occurred. The repeated tension (Fig. 1) caused by traffic loadings tends to overcome the interface adhesion. The resulting separations, which occur initially at localized points of stress concentration and poor adhesion, allow water to reach the interface which accelerates the stripping problem.

A dense graded asphalt concrete having a low permeability (low air voids) has a better resistance to stripping than a high permeability, open graded mix [16]. *To minimize stripping problems the aggregate should be completely coated with asphalt.* A completely coated particle makes it harder for water to penetrate to the asphalt-aggregate interface and cause stripping.

Low viscosity asphalt cements are better at coating the aggregate than high viscosity cements. On the other hand, the higher viscosity

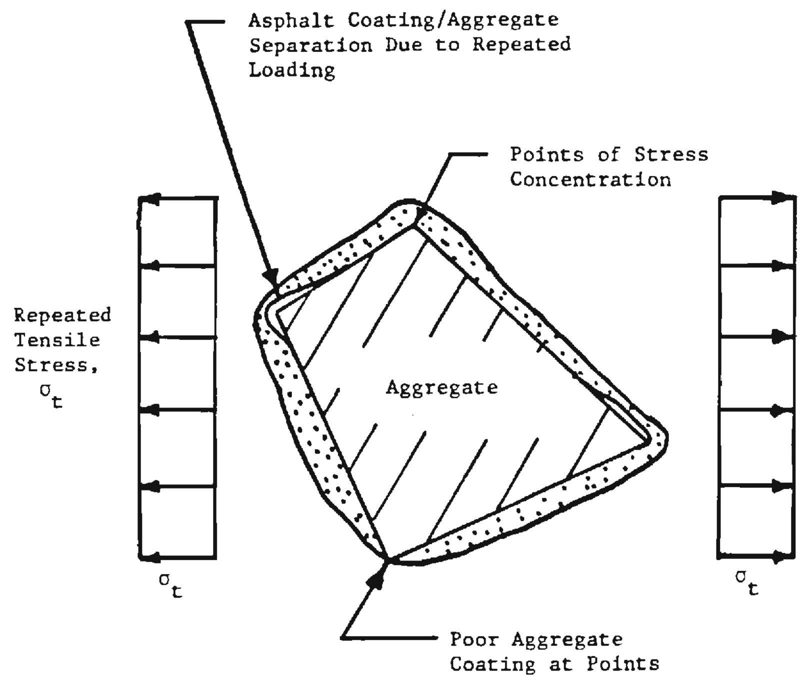

FIGURE 1. AGGREGATE-ASPHALT COATING SUBJECTED TO REPEATED LOADING.

asphalt cements, once on the aggregate, have a greater resistance to displacement by water than low viscosity asphalt cements. Rounded aggregates can be more easily uniformly coated than angular aggregates with sharp corners.

MECHANISMS OF STRIPPING

Taylor and Khosla [14] have recently presented an excellent state-of-the-art summary of stripping in asphalt concrete pavements. The following mechanisms of stripping which are briefly described were identified by them:

Detachment. *Presence of a thin layer of water or a dust coating on the aggregate surface can prevent proper bond of the asphalt to the surface.* Therefore the aggregate should be clean and the surface dry before mixing with asphalt.

Displacement. In this mechanism water penetrates to the aggregate surface displacing the asphalt film. The coating of asphalt film on the aggregate may be ruptured at sharp edges or due to a coating of dust present on the aggregate. Also, a small hole, due to poor coating, may exist in the asphalt film. Both these mechanisms allow water to penetrate to the aggregate-asphalt interface. Finally, an electrical charge on the aggregate surface and in the water can cause a build-up in a water layer at the interface.

Spontaneous Emulsification. The asphalt, in the presence of water, may emulsify at the aggregate-asphalt interface. Emulsification would of course be a form of stripping (loss of bond). Taylor and Khosla indicate emulsification may always be occurring, but at greatly varying rates.

Clays and some chemical additives can act as emulsifiers and greatly accelerate the emulsification process. Osmosis may also play a role in disrupting the asphalt film.

Pore Pressure. The postulation has been made that pore pressure may also cause or accelerate stripping. Assume water is present at the aggregate interface. Upon application of load a build-up in pore pressure could occur tending to cause further separation at the interface. Pore pressure might also affect the emulsification process.

Hydraulic Scouring. When the surface of a pavement is saturated, a moving wheel load forces water into the pavement ahead of the wheel. This causes a suction of water upward from behind the wheel. The hydraulic scouring action of the water helps to accelerate stripping and the formation of potholes in the surface.

SUMMARY

Many asphalt concrete mixes, especially those containing aggregates with a high silica content, are susceptible to stripping. Stripping (which is sometimes called debonding) is an adhesion failure at the interface between the aggregate and the asphalt cement. Numerous factors affect stripping including (1) the presence of moisture, (2) application of load repetitions, and (3) the mix design. To resist stripping the aggregate should be dry and clean of dust, clay, silt and other foreign substances. Also the aggregate should be completely coated. The mix should have a low permeability to water. Finally, the mix should be protected from water as much as possible.

A number of test methods have been proposed to evaluate the stripping problem in asphalt concrete mixes. The most promising tests appear to include the indirect tensile splitting test, boiling test, Texas pedestal test and the fatigue test. More laboratory and field work is required, however, to determine the most appropriate procedures for evaluting the stripping susceptibility of asphalt concrete mixes.

CHAPTER III

MATERIALS AND SPECIMEN PREPARATION

INTRODUCTION

Fatigue, rutting and indirect tension tests were performed on beams, cylinders and Marshall specimens, respectively. All asphalt concrete specimens tested were a Georgia Department of Transportation E surface mix. A separate mix design was prepared for each mix.

MATERIAL PROPERTIES

Marshall Mix Designs for each asphalt concrete mix studied in this investigation were performed by the Georgia Department of Transportation, Office of Materials and Research. A summary of the mix designs investigated are given in Tables 2 through 10. The primary variable in the mix designs was the additive. The materials and additive type including source is summarized in Table 11. Detailed properties are given in Tables 12 through 16. All mixes had almost the same aggregate gradation. The specimens tested had 7 percent air voids to allow water to penetrate the mix.

An AC-30 viscosity grade asphalt cement was used as the basic binder. Exceptions were the granulated rubber mix which used an AC-20, and the recycled mix which used an AC-10 for the new asphalt. The physical properties including source of these asphalt cements are summarized in Table 12.

Crushed granite gneiss aggregate obtained from Vulcan Materials Company's Kennesaw Quarry was used in all asphalt concrete mixes. The physical properties of the crushed granite-gneiss aggregate are summarized

DEPARTMENT OF TRANSPORTATION
STATE OF GEORGIA
OFFICE OF MATERIALS AND RESEARCH

ASPHALTIC CONCRETE DESIGN REPORT

TABLE 2

MIX: Surface E Mix
ADDITIVE: None
BINDER: AMOCO Oil Co.
AC: 30 **DENSITY:** 148.3 pcf for 7% Voids

GENERAL DATA – AGGREGATE

TYPE MATERIAL	SIZE	% USED	GROUP	SOURCE	LOCATION
COARSE AGGREGATE	7	38	II	Vulcan Materials Co.	Kennesaw, Ga.
FINE AGGREGATE 100<3/7"	810	47	II	Vulcan Materials Co.	Kennesaw, Ga.
	Washed				
	Screenings	15	II	Vulcan Materials Co.	Kennesaw, Ga.
MINERAL FILLER					

DESIGN MIX PROPERTIES (50 BLOW MIX)

ASPHALT CEMENT (%)	THEORETICAL SPECIFIC GRAVITY	ACTUAL SPECIFIC GRAVITY	VOIDS IN MIX (%)	MIX DENSITY (lbs ft)	VOIDS IN MINERAL AGGREGATE %	AGGREGATE VOIDS FILLED WITH A.C. (%)	STABILITY (lbs)	FLOW (1/100")
4.5	2.608	2.386	8.5	148.9	18.9	55.0	1550	9.1
5.0	2.587	2.418	6.5	150.9	18.2	64.3	1720	9.4
5.5	2.567	2.442	4.9	152.4	17.8	72.4	1880	10.9
6.0	2.547	2.457	3.5	153.3	17.8	80.3	1980	11.2
6.5	2.527	2.460	2.7	153.5	18.1	85.1	1830	13.4

COMPACTIVE EFFORT FOR 7% VOIDS

5.8	15 blows		7.9	146.9	21.1	62.6	950	11.7
5.8	20 blows		6.5	149.0	19.9	67.3	1060	10.5
5.8	25 blows		6.6	148.9	20.0	67.0	1150	9.9

VACUUM FOR 80% SATURATION (INCHES OF MERCURY)

5.8	19 blows		15 inches Hg	52% saturation	23" Hg
5.8	19 blows		20 inches Hg	68% saturation	
5.8	19 blows		25 inches Hg	92% saturation	

AGGREGATE GRADATIONS

Size	1	3/4	1/2	3/8	4	8	16	30	50	100	200	325
7		100	96	60	6							
810				100	84	67	57	43	28	16	9	7
Washed Scr.				100	99	83	62	41	20	7	2	1
Combined		100	98	85	57	45	36	26	16	9	5	3

SPECIMEN PREPARATION TEMPERATURES: The aggregate at 350°F is dry mixed to 320°F. Add the AC at 320°F and thoroughly mix. When the mix reaches 300°F place in mold and compact at 295°F.

DEPARTMENT OF TRANSPORTATION
STATE OF GEORGIA
OFFICE OF MATERIALS AND RESEARCH

ASPHALTIC CONCRETE DESIGN REPORT

TABLE 3
MIX: Surface E Mix
ADDITIVE: 0.5% AS-1 Indulin Anti-Strip
BINDER: AMOCO Oil Co.
AC: 30 DENSITY: 147.9 pcf for 7% Voids

GENERAL DATA – AGGREGATE

TYPE MATERIAL	SIZE	% USED	GROUP	SOURCE	LOCATION
COARSE AGGREGATE	7	38	II	Vulcan Materials Co.	Kennesaw, Ga.
FINE AGGREGATE	810	47	II	Vulcan Materials Co.	Kennesaw, Ga.
	Washed				
	Screenings	15	II	Vulcan Materials Co.	Kennesaw, Ga.
MINERAL FILLER	AS-1	0.5	-	Westvaco	Winder, Ga.

DESIGN MIX PROPERTIES (50 BLOW MIX)

ASPHALT CEMENT (%)	THEORETICAL SPECIFIC GRAVITY	ACTUAL SPECIFIC GRAVITY	VOIDS IN MIX (%)	MIX DENSITY (lbs/ft³)	VOIDS IN MINERAL AGGREGATE (%)	AGGREGATE VOIDS FILLED WITH A.C. (%)	STABILITY (lbs.)	FLOW (1/100")
4.5	2.607	2.402	7.9	149.9	18.3	56.8	1610	9.0
5.0	2.586	2.421	6.4	151.1	18.1	64.6	1760	9.3
5.5	2.566	2.438	5.0	152.1	18.0	72.2	1850	10.1
6.0	2.546	2.449	3.8	152.8	18.0	78.9	1940	10.9
6.5	2.527	2.454	2.9	153.1	18.3	84.2	1840	12.3

COMPACTIVE EFFORT FOR 7% VOIDS

5.9	15 blows		7.4	147.3	20.9	64.6	980	12.0
5.9	20 blows		6.1	149.4	19.8	69.2	1010	10.7
5.9	25 blows		5.5	150.3	19.3	71.5	1160	10.0

VACUUM FOR 80% SATURATION (INCHES OF MERCURY)

5.9	17 blows		15 inches Hg	59% saturation	22" Hg
5.9	17 blows		20 inches Hg	70% saturation	
5.9	17 blows		25 inches Hg	96% saturation	

AGGREGATE GRADATIONS

Size	1	3/4	1/2	3/8	4	8	16	30	50	100	200	325
7		100	96	60	6	3						
810				100	84	67	57	43	28	16	9	7
Washed Scr.				100	99	83	62	41	20	7	2	1
Combined		100	98	85	57	45	36	26	16	9	5	3

SPECIMEN PREPARATION TEMPERATURES: The aggregate at 350°F is dry mixed to 320°F. Add the AC at 320°F and thoroughly mix. When the mix reaches 300°F place in mold and compact at 295°F.

DEPARTMENT OF TRANSPORTATION
STATE OF GEORGIA
OFFICE OF MATERIALS AND RESEARCH

ASPHALTIC CONCRETE DESIGN REPORT

TABLE 4

MIX: Surface E Mix
ADDITIVE: 1% Dry Lime/99% Aggregate
BINDER: AMOCO Oil Co.
AC: 30 **DENSITY:** 148.2 pcf for 7% Voids

GENERAL DATA – AGGREGATE

TYPE MATERIAL	SIZE	% USED	GROUP	SOURCE	LOCATION
COARSE AGGREGATE	7	38	II	Vulcan Materials Co.	Kennesaw, Ga.
FINE AGGREGATE	810	47	II	Vulcan Materials Co.	Kennesaw, Ga.
	Washed				
	Screenings	14	II	Vulcan Materials Co.	Kennesaw, Ga.
MINERAL FILLER	Lime	1	–	Martin Marietta Cement Co.	Birmingham, Ala.

DESIGN MIX PROPERTIES (50 BLOW MIX)

ASPHALT CEMENT (%)	THEORETICAL SPECIFIC GRAVITY	ACTUAL SPECIFIC GRAVITY	VOIDS IN MIX (%)	MIX DENSITY (lbs ft³)	VOIDS IN MINERAL AGGREGATE (%)	AGGREGATE VOIDS FILLED WITH A.C. (%)	STABILITY (lbs.)	FLOW (1/100")
4.5	2.606	2.399	7.9	149.7	18.4	57.1	1740	9.8
5.0	2.586	2.425	6.2	151.3	17.9	65.4	1820	10.0
5.5	2.565	2.444	4.7	152.5	17.7	73.4	2070	11.3
6.0	2.546	2.455	3.6	153.2	17.8	79.8	2010	12.3
6.5	2.526	2.457	2.7	153.3	18.2	85.2	1770	14.6

COMPACTIVE EFFORT FOR 7% VOIDS

5.7	15 blows		8.0	146.8	21.0	61.9	1090	11.3
5.7	20 blows		7.2	148.0	20.3	64.5	1210	10.7
5.7	25 blows		6.7	148.9	19.8	66.2	1530	11.1

VACUUM FOR 80% SATURATION (INCHES OF MERCURY)

5.7	21 blows	15 inches	Hg		48% Saturation	23" Hg
5.7	21 blows	20 inches	Hg		68% Saturation	
5.7	21 blows	25 inches	Hg		87% Saturation	

AGGREGATE GRADATIONS

Size	1	3/4	1/2	3/8	4	8	16	30	50	100	200	325
7		100	96	60	6	3						
810				100	84	67	57	43	28	16	9	2
Washed Scr.				100	99	83	62	41	20	7	2	1
Combined		100	98	85	57	45	36	27	17	10	6	4

SPECIMEN PREPARATION TEMPERATURES: The aggregate is treated with 1% hydrated lime in dry form. The aggregate and dry lime at 350°F is mixed to 320°F. Add the AC at 320°F and thoroughly mix. When the mix reaches 300°F place in a mold and compact at 295°F.

DEPARTMENT OF TRANSPORTATION
STATE OF GEORGIA
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ASPHALTIC CONCRETE DESIGN REPORT

TABLE 5

MIX: Surface E Mix
ADDITIVE: 2% Dry Lime/98% Aggregate
BINDER: AMOCO Oil Co.
AC: 30 **DENSITY:** 148.2 pcf for 7% Voids

GENERAL DATA – AGGREGATE

TYPE MATERIAL	SIZE	% USED	GROUP	SOURCE	LOCATION
COARSE AGGREGATE	7	38	II	Vulcan Materials Co.	Kennesaw, Ga.
FINE AGGREGATE	810	47	II	Vulcan Materials Co.	Kennesaw, Ga.
	Washed				
	Screenings	13		Vulcan Materials Co.	Kennesaw, Ga.
MINERAL FILLER	Lime	2	II	Martin Marietta Cement Co.	Birmingham, ALA.

DESIGN MIX PROPERTIES (50 BLOW MIX)

ASPHALT CEMENT (%)	THEORETICAL SPECIFIC GRAVITY	ACTUAL SPECIFIC GRAVITY	VOIDS IN MIX (%)	MIX DENSITY (lbs/ft)	VOIDS IN MINERAL AGGREGATE (%)	AGGREGATE VOIDS FILLED WITH A.C. (%)	STABILITY (lbs)	FLOW (1/100")
4.5	2.595	2.396	7.7	149.5	18.1	57.5	1990	9.5
5.0	2.574	2.420	6.0	151.0	17.7	66.1	2130	10.2
5.5	2.554	2.436	4.6	152.0	17.6	73.9	2090	11.3
6.0	2.535	2.457	3.1	153.3	17.3	82.1	1930	12.5
6.5	2.515	2.468	1.9	154.0	17.4	89.1	1770	16.3

COMPACTIVE EFFORT FOR 7% VOIDS

5.5	15 blows		7.9	146.8	24.0	61.3	1140	11.9
5.5	20 blows		6.8	148.6	19.4	64.9	1360	11.6
5.5	25 blows		6.1	149.7	18.8	67.6	1550	11.5

VACUUM FOR 80% SATURATION (INCHES OF MERCURY)

5.5	20 blows	15 inches	Hg		55% Saturation	23" HG
5.5	20 blows	20 inches	Hg		74% Saturation	
5.5	20 blows	25 inches	Hg		82% Saturation	

AGGREGATE GRADATIONS

Size	1	3/4	1/2	3/8	4	8	16	30	50	100	200	325
7		100	96	60	6	3						
810				100	84	67	57	43	28	16	9	7
Washed Scr.				100	99	83	62	41	20	7	2	1
Combined		100	98	85	57	45	37	28	18	10	6	5

SPECIMEN PREPARATION TEMPERATURES: The aggregate is treated with 2% hydrated lime in dry form. The aggregate and dry lime at 350°F is dry mixed to 320°F. Add the AC at 320°F and thoroughly mix. When the mix reaches 300°F place in mold and compact it at 295°F.

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TABLE 6
MIX: Surface E Mix
ADDITIVE: 1% Lime Slurry/99% Aggregate
BINDER: AMOCO Oil Co.
AC: 30 DENSITY: 147.9 pcf for 7% Voids

GENERAL DATA – AGGREGATE

TYPE MATERIAL	SIZE	% USED	GROUP	SOURCE	LOCATION
COARSE AGGREGATE	7	38	II	Vulcan Materials Co.	Kennesaw, Ga.
FINE AGGREGATE	810	47	II	Vulcan Materials Co.	Kennesaw, Ga.
	Washed				
	Screenings	14	II	Vulcan Materials Co.	Kennesaw, Ga.
MINERAL FILLER	Lime	Slurry	-	Martin Marietta Cement Co.	Birmingham, Ala.

DESIGN MIX PROPERTIES (50 BLOW MIX)

ASPHALT CEMENT (%)	THEORETICAL SPECIFIC GRAVITY	ACTUAL SPECIFIC GRAVITY	VOIDS IN MIX (%)	MIX DENSITY (lbs ft³)	VOIDS IN MINERAL AGGREGATE (%)	AGGREGATE VOIDS FILLED WITH A C (%)	STABILITY (lbs)	FLOW (1/100")
4.5	2.600	2.391	8.0	149.2	18.4	56.7	1720	9.1
5.0	2.580	2.412	6.5	150.5	18.2	64.3	1770	10.0
5.5	2.560	2.429	5.1	151.6	18.0	71.7	2010	10.9
6.0	2.540	2.447	3.7	152.7	17.9	79.3	1900	12.2
6.5	2.521	2.462	2.4	153.6	17.8	86.5	1830	14.0

COMPACTIVE EFFORT FOR 7% VOIDS

5.8	15 blows		7.4	147.2	20.6	64.1	1040	11.9
5.8	20 blows		6.3	148.9	19.7	68.0	1340	11.5
5.8	25 blows		5.5	150.3	19.0	71.1	1290	10.7

VACUUM FOR 80% SATURATION (INCHES OF MERCURY)

5.8	17 blows		15 inches Hg	55% saturation	22" Hg
5.8	17 blows		20 inches Hg	73% saturation	
5.8	17 blows		25 inches Hg	91% saturation	

AGGREGATE GRADATIONS

Size	1	3/4	1/2	3/8	4	8	16	30	50	100	200	325
7		100	96	60	6							
810				100	84	67	57	43	28	16	9	7
Washed Scr.				100	99	83	62	41	20	7	2	1
Combined		100	98	85	57	45	36	27	17	10	6	4

SPECIMEN PREPARATION TEMPERATURES: The aggregate is treated with 1% hydrated lime in slurry form. Temperature: heat aggregate with lime to 350°F, then dry mix to 320°F. Add AC at 320°F and mix thoroughly. When the mix reaches 300°F place in mold and compact at 295°F.

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TABLE 7
MIX: Surface E Mix
ADDITIVE: 20% Granulated Rubber/80% AC-20
BINDER: AMOCO Oil Co.
AC: 20 DENSITY: 142.5 pcf for 7% Voids

GENERAL DATA – AGGREGATE

TYPE MATERIAL	SIZE	% USED	GROUP	SOURCE	LOCATION
COARSE AGGREGATE	7	38	II	Vulcan Materials Co.	Kennesaw, Ga.
FINE AGGREGATE	810	47	II	Vulcan Materials Co.	Kennesaw, Ga.
	Washed				
	Screenings	15	II	Vulcan Materials Co.	Kennesaw, Ga.
MINERAL FILLER					

DESIGN MIX PROPERTIES (50 BLOW MIX)

A.C. (%)	A.C. +R*	THEORETICAL SPECIFIC GRAVITY	ACTUAL SPECIFIC GRAVITY	VOIDS IN MIX (%)	MIX DENSITY (lbs ft³)	VOIDS IN MINERAL AGGREGATE %	AGGREGATE VOIDS FILLED WITH A.C. (%)	STABILITY (lbs)	FLOW (1/100")
5.6	7.0	2.500	2.333	6.7	145.6	19.4	65.5	1967	12.0
6.0	7.5	2.485	2.341	5.8	146.1	19.4	70.1	1633	12.2
6.4	8.0	2.471	2.343	5.2	146.2	19.7	73.6	1550	13.0
6.8	8.5	2.456	2.345	4.5	146.3	20.0	77.5	1625	14.2
7.2	9.0	2.442	2.341	4.1	146.1	20.4	79.9	1625	14.8

* AC + Rubber (%)

COMPACTIVE EFFORT FOR 7% VOIDS

6.8	8.5	15 blows		9.3	139.0	24.0	61.3	1023	10.6
6.8	8.5	20 blows		8.3	140.5	23.2	64.2	1243	11.5
6.8	8.5	25 blows		6.2	143.8	21.3	70.9	1247	11.6

VACUUM FOR 80% SATURATION (INCHES OF MERCURY)

6.8	8.5	24 blows		15 inches	Hg	67% Saturation	22" Hg
6.8	8.5	24 blows		20 inches	Hg	74% Saturation	
6.8	8.5	24 blows		25 inches	Hg	93% Saturation	

AGGREGATE GRADATIONS

Size	1	3/4	1/2	3/8	4	8	16	30	50	100	200	325
7		100	96	60	6							
810				100	84	67	57	43	28	16	9	7
Washed Scr.				100	99	83	62	41	20	7	2	1
Combined		100	98	85	57	45	36	26	16	9	5	3

SPECIMEN PREPARATION TEMPERATURES: The binder is AC-20 from AMOCO and rubber from Arizona Refining Co. Temperatures: aggregate 450°F dry mix to 400°F, then add AC at 400°F and mix thoroughly. When mix reaches 380°F transfer to mold and compact at 350°F.

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TABLE 8
MIX: Surface E Mix
ADDITIVE: 3% Latex 97% AC-30
BINDER:
AC: 30 **DENSITY:** 147.9 pcf for 7% Voids

GENERAL DATA – AGGREGATE

TYPE MATERIAL	SIZE	% USED	GROUP	SOURCE	LOCATION
COARSE AGGREGATE	7		II	Vulcan Materials Co.	Kennesaw, Ga.
FINE AGGREGATE	810		II	Vulcan Materials Co.	Kennesaw, Ga.
	Washed				
	Screenings		II	Vulcan Materials Co.	Kennesaw, Ga.
MINERAL FILLER					

DESIGN MIX PROPERTIES (50 BLOW MIX)

ASPHALT CEMENT (%)	THEORETICAL SPECIFIC GRAVITY	ACTUAL SPECIFIC GRAVITY	VOIDS IN MIX (%)	MIX DENSITY (lbs ft³)	VOIDS IN MINERAL AGGREGATE (%)	AGGREGATE VOIDS FILLED WITH A.C. (%)	STABILITY (lbs)	FLOW (1/100")
4.5	2.601	2.354	9.5	146.9	19.8	52.0	1320	9.4
5.0	2.580	2.396	7.1	149.5	18.8	62.2	1530	9.9
5.5	2.560	2.426	5.2	151.4	18.2	71.4	2130	10.4
6.0	2.540	2.441	3.9	152.3	18.2	78.6	1900	10.7
6.5	2.520	2.444	3.0	152.5	18.5	83.8	1900	12.3

COMPACTIVE EFFORT FOR 7% VOIDS

5.8	15 blows		10.0	143.0	23.0	56.5	800	13.2
5.8	20 blows		8.2	146.0	21.4	61.7	1410	11.5
5.8	25 blows		6.2	149.1	19.7	68.5	1440	10.7

VACUUM FOR 80% SATURATION (INCHES OF MERCURY)

5.8	23 blows		15 inches	Hg	77% Saturation	16" Hg
5.8	23 blows		20 inches	Hg	87% Saturation	
5.8	23 blows		25 inches	Hg	87% Saturation	

AGGREGATE GRADATIONS

Size	1	3/4	1/2	3/4	4	8	16	30	50	100	200	325
7		100	96	60	6	3						
810				100	84	67	57	43	28	16	9	7
Washed Scr.				100	99	83	62	41	20	7	2	1
Combined		100	98	85	57	45	36	26	16	9	5	3

SPECIMEN PREPARATION TEMPERATURES: The aggregate at 350°F is dry mixed to 320°F. Add the AC at 320°F and thoroughly mix. When the mix reaches 300°F place in mold and compact at 295°F.

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TABLE 9

Recycle E Mix

PROJECT NO. IR-85-2 (96) Ct. 2 (I-85)

COUNTY DeKalb & Gwinnett DATE 9/21/82

REPORT ON ASPHALTIC CONCRETE E

GENERAL DATA — AGGREGATE

TYPE MATERIAL	SIZE	% USED	GROUP	SOURCE	LOCATION
RECLAIMED MIX	Milled Material	40		I-85 Roadway	DeKalb & Gwinnett
AGGREGATE	7	35	IIB	Vulcan Materials Company	Norcross, GA
Washed Screenings		24.4	IIA	Vulcan Materials Company	Kennesaw, GA
MINERAL FILLER	Hydrated Lime	0.6		Martin Marietta Cement	Birmingham, AL

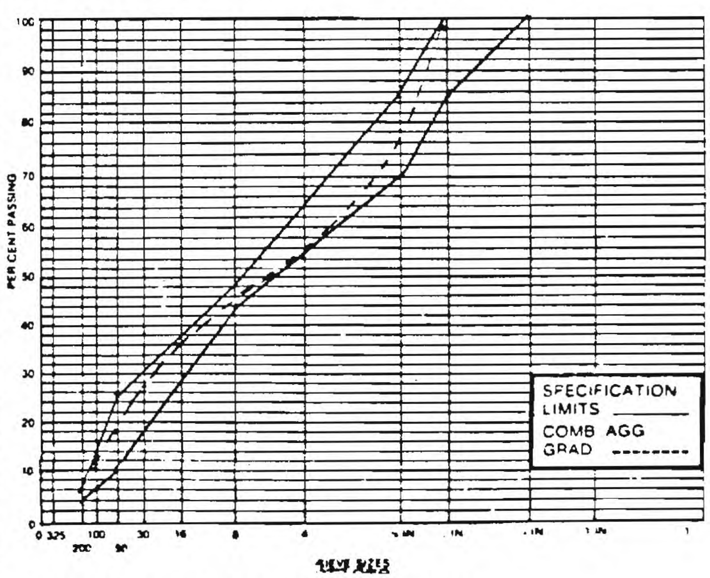
DESIGN MIX PROPERTIES

Total	ASPHALT CEMENT OLD	ASPHALT CEMENT NEW	THEORETICAL SPECIFIC GRAVITY	ACTUAL SPECIFIC GRAVITY	VOIDS IN MIX (%)	MIX DENSITY (lbs ft³)	VOIDS IN MINERAL AGGREGATE (%)	AGGREGATE VOIDS FILLED WITH A C (%)	STAB. (LBS.)	FLOW (.01")	ABSON PEN	ABSON VIC
5.25	2.0	3.25	2.496	2.365	5.2	147.6	17.3	69.9	1810	8.0		
5.5	2.0	3.5	2.484	2.385	4.0	148.8	16.9	76.3	2085	8.7	46	13048
5.75	2.0	3.75	2.475	2.386	3.6	148.9	17.0	78.8	1930	8.7		

RETAINED STABILITY WITHOUT ADDITIVE	%	TENSILE SPLIT: CONTROL 62.5 PSI, CURED 83.0 PSI	132.8 %
OPTIMUM ASPHALT CONTENT	5.4 %	THEORETICAL SPECIFIC GRAVITY	2.490

AGGREGATE GRADATIONS, SPECIFIC GRAVITIES AND COMBINED GRADATION CURVE

SIZE	Milled Material		7	Washed Screenings		Hydrated Lime	COMB GRAD
1 ½"							
1"							
¾"	100		100				100
½"	99		95				98
⅜"	93		42	100			77
#4	72		3	99			55
#8	58		2	84			45
#16	50			62			36
#30	42			41			27
#50	31			20			18
#100	20			7		100	10
#200	12			2		100	6
AC%							
APP							
BLK							
EFF							2.711



REMARKS:
This design was fabricated using AC-10 from AMOCO @ Savannah with 0.5% Pave Bond LP additive. Density for 7% voids is 146.6 pcf.

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TABLE 10
MIX: Surface E Mix
ADDITIVE: 30% Trinidad Asphalt & 70% AC-30
BINDER: AMOCO Oil Co.
AC: 30 **DENSITY:** 148.5 pcf for 7% Voids

GENERAL DATA – AGGREGATE

TYPE MATERIAL	SIZE	% USED	GROUP	SOURCE	LOCATION
COARSE AGGREGATE	7	38	II	Vulcan Materials Co.	Kennesaw, Ga.
FINE AGGREGATE	810	47	II	Vulcan Materials Co.	Kennesaw, Ga.
	Washed				
	Screenings	15	II	Vulcan Materials Co.	Kennesaw, Ga.
MINERAL FILLER					

DESIGN MIX PROPERTIES (50 BLOW MIX)

ASPHALT CEMENT (%)	THEORETICAL SPECIFIC GRAVITY	ACTUAL SPECIFIC GRAVITY	VOIDS IN MIX (%)	MIX DENSITY (lbs ft)	VOIDS IN MINERAL AGGREGATE %	AGGREGATE VOIDS FILLED WITH A C (%)	STABILITY (lbs)	FLOW (1/100")
4.5	2.606	2.356	9.6	147.0	19.8	51.5	1610	9.2
5.0	2.586	2.388	7.7	149.0	19.2	59.9	1740	10.0
5.5	2.565	2.417	5.8	150.8	18.6	68.8	1820	10.2
6.0	2.546	2.436	4.3	152.0	18.4	76.6	1970	10.3
6.5	2.526	2.446	3.2	152.6	18.5	82.7	2030	11.0

COMPACTIVE EFFORT FOR 7% VOIDS

6.1	15 blows		8.5	146.2	21.6	60.6	1210	12.4
6.1	20 blows		7.7	147.4	21.0	63.3	1410	11.7
6.1	25 blows		6.7	149.0	20.1	66.7	1640	11.3

VACUUM FOR 80% SATURATION (INCHES OF MERCURY)

6.1	23 blows		15 inches Hg	52% Saturation	23" Hg
6.1	23 blows		20 inches Hg	67% Saturation	
6.1	23 blows		25 inches Hg	88% Saturation	

AGGREGATE GRADATIONS

Size	1	3/4	1/2	3/8	4	8	16	30	50	100	200	325
7		100	96	60	6	3						
810				100	84	67	57	43	28	16	9	7
Washed Scr.				100	99	83	62	41	20	7	2	1
Combined		100	98	85	57	45	36	26	16	9	5	3

SPECIMEN PREPARATION TEMPERATURES: Place the ACin a pan and heat slowly adding the Trinidad asphalt while mechanically mixing. The aggregate at 350°F is dry mixed to 320°F. Add the AC at 320°F and thoroughly mix. When the mix reaches 300°F place in mold and compact at 295°F.

TABLE 11. SUMMARY OF MATERIALS AND ADDITIVES USED.

Product	Source	Specific Gravity	Properties
AC-30 AC-20 AC-10	AMOCO Oil Company P. O. Box 5077 Atlanta, Ga. 30302	1.041	Table 12
Aggregate	Vulcan Materials Co. Kennesaw, Ga.	2.777 (effective)	Table 13
Indulin AS-1	Westvaco Corp. Polychemicals Dept. P.O. Box 5207 N. Charleston, S.C. 29406	0.9895	Table 14
Lime	Martin Marietta Cement Co. Southern Division P.O. Box 182 Roberta, Alabama 35040	2.283 (approx. 38 pcf loose)	Table 15
Latex Ultrapave 65K	Textile Rubber & Chemical Co. 1300 Tiarco Dr.SW Dalton, Ga. 30720		Table 16
Granulated Rubber	Arizona Refining Corp. P.O. Box 1453 Phoenix, Arizona 85001 Ph: 1-800-528-5305		Table 17
Trinidad Asphalt		1.402	Table 12

TABLE 12. SUMMARY OF THE ASPHALT CEMENT PROPERTIES USED IN SURFACE E MIXES.

PHYSICAL PROPERTY	ASPHALT CEMENT(1)			TRINIDAD ASPHALT	
	AC-30	AC-20	AC-10(I-85)	100%	30% T./70% AC-20
KINEMATIC VISCOSITY					
140°F (Poises)	2786	1970	940		-
275°F (Centistokes)	561	400	280	31,520	687(4)
PENETRATION: 100 gm., 5 sec. (1/10 mm), 77°F	72	87	145	1.5	41
SPECIFIC GRAVITY	1.040	1.037	1.032	1.402	1.125
CLEVELAND FLASH POINT °F	477	495	480	387	547
DUCTILITY(2) 77°F (cm)	125	150+	150+	0.0(3) 3.75	32.5
THIN FILM RESIDUE	7627	5400	2300		

Notes: 1. Asphalt from AMOCO Oil Company, Savannah, Georgia.
2. Test performed on thin film residue.
3. Original ductility at 5 cm is 0.0 and at 0.25 cm is 3.75.
4. Residual viscosity at 275°F is 1,110; residual viscosity at 140°F is 15,850
5. Original ductility is 72.

TABLE 13. TYPICAL PROPERTIES OF KENNESAW GRANITE-GNEISS CRUSHED STONE.

Aggregate Description	Granite-Gneiss
Georgia DOT Class Aggregate	A
Group	Group II (silaceous)
Bulk Specific Gravity	2.77
Bulk Specific Gravity (SSD)	2.79
Apparent Specific Gravity	2.81
Sand Equivalent (graded aggregate base)	35
Absorption (%)	0.50
L.A. Abrasion (%)	37
Mag. Sulfate Soundness Loss (%)	1.8
Mineral Composition (%)(1)	A/B (2)
Quartz	15/20
Plagioclase	30/22
Microcline	5/8
Hornblend	30/25
Olivine	2/0
Biotite Mica	17/25
Trace Minerals	Present

1. Based on two petrographic studies made in 1974. Sample A was a Granite Diorite Gneiss and Sample B was a Biotite Hornblend Granite Gneiss.

2. The first number given is for sample A and the second number is for sample B.

TABLE 14. VISCOSITY REDUCTION AS A FUNCTION OF TREATMENT TIME FOR ANTI-STRIP ADDITIVES.

	CHEVRON U.S.A., INC. MOBILE, ALABAMA				HUNT OIL COMPANY TUSCALOOSA, ALABAMA			
TIME (HRS.)	0.5% INDULIN	% RELATIVE DROP	1.0% PAVEBOND	% RELATIVE DROP	0.5% INDULIN	% RELATIVE DROP	1% PAVEBOND	% RELATIVE DROP
0	1703	9.5	1576	16.2	1830	9.4	1658	17.9
2	1715	8.3	1595	14.8	1813	11.6	1622	20.9
4	1729	7.0	1634	12.2	1849	10.1	1681	18.3
24	1854	6.9	1727	13.3	1893	12.9	1767	18.7
48	2006	5.9	1928	9.6	2011	15.6	1908	19.9
72	2172	0.05	2124	2.2	2098	17.1	2036	19.6
96	2330	0.9	2317	0.6				

TABLE 15. PROPERTIES OF HYDRATED LIME - ASTM C-206[1]

Specific Gravity	2.283[2]
Ignition Loss (%)	25.18
Insoluble + SiO_2 (%)	0.90
Aluminum + Ferric Oxide (%)	0.45
Sulfur Trioxide (%)	0.18
Calcium Oxide (%)	71.55
Magnesium Oxide (%)	1.74
Non-Volatiles:	
CaO (%)	95.64
MgO (%)	2.32
Total -	97.96

Notes: 1. Source: Magnolia, Alabama.
2. Unit weight is approximately 38 pcf.

TABLE 16. SUMMARY OF LATEX ULTRAPAVE 65K PROPERTIES[1,2]

Monomer Ratio, Butadiene/Styrene	70/30
Solids Content, min.%	60
Solids Content, min. lbs./gal.	5.0
Coagulum on 80 mesh screen, max. %	0.1
Mooney Viscosity of Polymer (ML 4 @ 212°F) min.	100
pH of Latex	5.5
Surface Tension, dynes/cm	28-40
Brookfield Viscosity, cps	1500 max.
Storage Stability	Excellent
Mechanical Stability	Excellent

* For information purposes, average solids content runs 68% and 5.4 lbs./gal.

Notes: 1. Data obtained from manufacturer's literature.

2. Ultrapave 65K is a cationic (positively charged) styrene/butadiene latex for cationic asphalt emulsions.

in Table 13. This aggregate is highly silaceous (Ga. DOT Group II) having a silica content greater than 60 percent. The aggregate is also highly micaceous having a mica content of about 21 percent.

Properties of the special additives used in the mixes are given in Tables 14 through 16.

ADDITIVE MIXING PROCEDURES

Specific details of each mix are given in Table 2 through 10. Compacting temperatures used for each mix are also summarized in these tables. The mixing procedures used to incorporate the additive are as follows:

1. Anti-Strip Mix (Tables 3, 12 & 14). Heat AS-1 and asphalt to 320°F. Thoroughly blend the two together using a mechanical stirrer. Add the anti-strip chemical AS-1 at a level of 0.5% by weight of the asphalt cement. The aggregate is placed in an oven at approximately 350°F (177°C) for two and one-half hours before mixing. Dry mix to 320°F (160°C). The asphalt cement at 320°F (160°C) is added, and thoroughly hand mixed until the aggregate is completely coated. When the mix reaches 300°F (148.9°C), transfer it to the mold and compact at 295°F (146.1°C). A similar mixing procedure is also used for the mix without any additive (Table 2).

2. Lime Slurry (Tables 6,12 and 15). Mix 11 grams of hydrated lime with 33 ml of water. Place the properly graded aggregate into a plastic container. Add the lime slurry, place a lid on the container, and shake vigorously. Pour the contents of container into metal pans, and place them in an oven. Dry the aggregate slurry in an oven at 350°F (176.7°C) Until all the moisture has been driven off; frequently dry stir the contents during drying. The aggregate was treated with a slurry having a hydrated lime weight equal to 1 and 2 percent of the aggregate. The lime weight was counted as part of the total aggregate weight.

3. Dry Lime (Tables 4,5,12 & 15). Place the proper weight of dry lime and aggregate in a plastic container. Shake the contents vigorously. Pour the contents in an open pan and bring to the required temperature of

350°F (176.7°C). Both 1 and 2 percent dry lime was used in this study based on total weight of aggregate plus lime. The dry lime was counted as part of the total weight of aggregate.

4. Recycle (Tables 9 and 12). Place the recycled mix obtained from the plant in a pan and heat to 320°F (160°C). The recycle mix consisted of 40 percent milled material from I-85 and 60 percent new material (see Table 9). Both 0.6 percent hydrated lime was present in the aggregate and 0.5 percent Pavebond anti-strip agent was in the asphalt cement. The optimum asphalt content of 5.4 percent consisted of about 2 percent old asphalt and 3.4 percent new asphalt. An AC-10 from AMOCO Oil Co. in Savannah was used in the new portion of the mix.

5. Latex Emulsion (Tables 8,12 & 16). The latex emulsion has 70% residual rubber in it. An amount of emulsion was added required to give 3 percent residual latex rubber (or 4.29 percent liquid latex emulsion) based on the total weight of residual latex rubber and asphalt. The mixing procedure is as follows: Heat the AC-30 to 285 to 300°F (140.6 - 148.9°C). Using a mechanical stirrer, add a few drops of latex emulsion at a time. As the latex emulsion is added, steam is given off; wait for the steam to stop before adding more latex. Then add a few more drops of latex at a time until the required amount has been added.

6. Granulated Rubber Asphalt Blend (Tables 7 and 12). The procedure used for the rubber asphalt mix is as follows: (1) Heat the asphalt cement to 400°F (204.4°C), (2) Add rubber at 20% of the weight of asphalt cement, (3) Blend the rubber into the asphalt cement while maintaining a temperature of 400°F (204.4°C), (4) Blend for one hour, (5) Place the blended rubber and asphalt cement in a 400°F (204.4°C) oven and cure for one hour, (6) The mixing temperature for the rubber asphalt and aggregate is 400°F (204.4°C), (7) The compaction temperature is 350°F (176.7°C). A satisfactory mix could not be developed using AC-30; therefore an AC-20 was used in this mix.

7. Trinidad Asphalt Blend (Tables 10 and 12). Place the AC-30 asphalt cement in a pan and heat until it is liquid. Slowly add the Trinidad Lake asphalt while mechanically mixing. The binder consisted of 30% Trinidad and 70% AC-30 asphalt.

SPECIMEN PREPARATION

All aggregates were sieved and the resulting sizes stored in plastic bags. The aggregate for each specimen was then prepared by weighing the required material for each sieve size, and carefully blending the sizes together.

The asphalt concrete fatigue and rutting specimens were moulded using a large kneading type compactor. The kneading compactor produced laboratory specimens with orientation of aggregates similar to those developed in the field during the rolling operation. The load foot of the compactor was held at the proper temperature by an internal heating coil. Marshall specimens were prepared using a standard Marshall hammer. Mixing temperatures used are given in Tables 2 through 10.

Beam Fatigue Specimens. The beam fatigue specimens used in this study for the fatigue tests were 3 in. by 3 in. (76 by 76 mm) in cross-section and 20 in. (508 mm) long. After heating, the beam mould was placed in a Cox and Sons kneading compactor on a sliding rack. Since the loading foot of the compactor does not move laterally, the beam mould was moved manually in the sliding rack during the compaction operation. The aggregate, asphalt cement and additive were mixed together in preweighed amounts so that all aggregate particles were completely coated by the asphalt cement. The hot asphalt concrete mixture was then placed in the mould in three layers. Each layer was compacted by 3 to 4 passes of the compactor along the length of the beam. After all asphalt concrete was placed in the mould, a loading plate was positioned on top of the beam and loaded until a height of 3.0 in. (76 mm) was reached. This procedure also served to level the surface of the specimen.

The beam and mould were allowed to cool, and the mould was removed. After cooling, each specimen was measured and then stored on a surface ground steel plate. The specimens were stored on the flat surface so they would lie flat on the rubber pad used in the fatigue test. The use of the machined steel plates for storage of the beams was necessary to avoid inducing tensile strains in the beam before testing, and to give uniform subgrade support to the beam during the fatigue test. The specimen number, date of compaction and the future location of the loading foot were marked on each specimen.

Cylinder Rutting Specimens. The cylindrical shaped specimens used in the repeated load triaxial tests were 4 in. (102 mm) in diameter by 8 in. (203 mm) high. These specimens were compacted in a cylindrical steel mould using the kneading compactor. With the mould in place, the hot asphalt concrete mixture was spooned into the mould. As the mould was filled, the load foot was actuated downward. The foot pressed down on the material in the mould one time between adding each spoonful of mixture. Filling the mould and compacting the specimen required approximately five minutes and took approximately 60 spoonfuls of material. This kneading action compacted the specimen to within about 1/8 in. (3.2 mm) of the finished specimen height.

A circular piece of filter paper cut to fit the inside diameter of the mould was placed on top of the compacted specimen and a loading head was positioned on top of the filter paper. The entire mould assembly was immediately placed in a testing machine. A static load was placed on the specimen to level the top and to finish compacting it to the specified height of 8 in. (203 mm). The specimen was loaded on the top and bottom by two floating pistons to minimize end effects. After cooling under the static load, the specimen was extruded from the mould and then measured and weighed. A China marker was used to mark the sample number and circumferential lines 2 in. (51 mm) from the ends (102 mm apart); these marks were later used for positioning the LVDT clamps on the specimen.

Indirect Tension Test Specimens. Standard Marshall specimens were used in the indirect tension test. The specimen preparation procedures are described in detail elsewhere [18].

SPECIMEN PRECONDITIONING

Approximately one-half of the beam, rutting and tensile splitting test specimens were preconditioned to simulate environmental weathering. All preconditioned asphalt concrete specimens were vacuum saturated to 80 percent saturation. The predetermined vacuum levels needed to give 80 percent saturation for each mix are given in Tables 2 through 10. The required vacuum was applied for 30 minutes, then removed and the specimen allowed to soak for an additional 30 minutes before removing from the water.

Fatigue and Rutting Specimens. The partial vacuum saturation procedure just described was all the preconditioning used for most of the fatigue and rutting tests. A more severe level of preconditioning of the fatigue and rutting test specimens was considered at the beginning of the study. At that time it was felt that more severely preconditioned specimens utilizing Kennesaw aggregate would probably not hold together for the fatigue and rutting test.

Two untreated and two 1 percent lime slurry treated fatigue test specimens were subjected to a more severe preconditioning than just partial vacuum saturation. Following the 80 percent saturation preconditioning previously described, these specimens were subjected to 15 hours of freezing at $0^{o}F$ ($-17.8^{o}C$), and then 4 hours of soaking at $140^{o}F$ ($48.9^{o}C$). The specimens were then removed from the bath and covered with Saran wrap. After storing in air for 18 hours (overnight), the specimens were tested in the fatigue apparatus.

The fatigue and rutting specimens were partially vacuum saturated in a plexiglass chamber 25 in. (635 mm) long and 6 in. (152 mm) in diameter. This chamber was constructed with a shelf so the long asphalt concrete beams could be fully supported during preconditioning. The end of the chamber was sealed with an end plate having an "O" ring.

The specimens were first slid into the chamber. Water at $85^{o}F$ ($29.4^{o}C$) was added until it covered the specimen approximately 0.25 in. (6 mm). The specimen was then subjected to the required vacuum to cause 80 percent saturation.

After removing the beam fatigue specimens from the bath, they were immediately wrapped in Saran wrap and tested. The cylindrical shaped rutting test specimens were not wrapped in Saran wrap since they were enclosed in a rubber membrane.

Tensile Splitting Test Specimens. The Marshall specimens used in the indirect tension test were first subjected to 80 percent vacuum saturation as previously described. The specimens were then frozen at $0^{o}F$ ($-17.8^{o}C$) for 15 hours, followed by soaking at $140^{o}F$ ($60^{o}C$) for 24 hours. The specimens were then brought to $55^{o}F$ ($12.8^{o}C$) by soaking in a bath at $55^{o}F$

(12.8°C) for 3 hours. After this final soaking, they were immediately tested. The preconditioning and testing procedure is described in detail by Test Method GHD-66.

CHAPTER IV

EQUIPMENT AND TEST PROCEDURES

Introduction

Repeated load testing was used in this study to evaluate the rutting and fatigue characteristics of seven asphalt concrete E surface mixes. The fatigue test was performed by placing the beams on a rubber subgrade and applying a repeated load at the center. The fatigue test results were used to determine the relative fatigue life of the mixes studied. The repeated load triaxial test was conducted using a constant confining pressure σ_3 and repeated deviator stress $\sigma_1 - \sigma_3$. Fatigue and rutting test specimens were tested between approximately 7 and 21 days after preparation. The tensile splitting test specimens were tested immediately after preparation. The Georgia DOT tensile splitting test method is described in detail elsewhere [18]. Therefore, the indirect tension test equipment and test procedures are not described in this report.

To minimize the time required for specimens to reach the desired temperature, both the fatigue and rutting beam specimens were stored before testing in a large constant temperature chamber.

Electronic Test Equipment Calibration

All electronic instrumentation was carefully calibrated to ensure accurate test results. The linear variable differential transducers (LVDT's) used to measure deflections were all calibrated with a micrometer calibration device accurate to 0.0001 in. (0.0025 mm). When possible, the

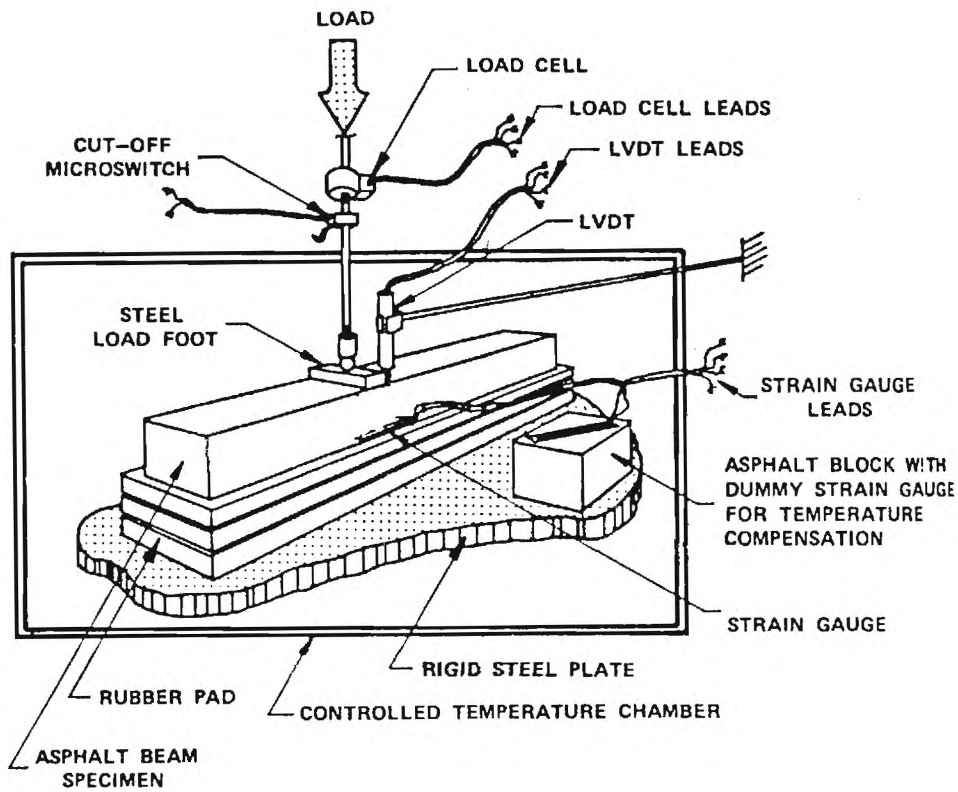
LVDT calibrations were checked before each test by using a steel measuring block having a thickness of 0.030 in. (0.76 mm). The load cells were statically calibrated by applying a load of known weight and recording the output from the load cell. The wire resistance strain gages were calibrated by connecting a shunt resistor of high precision in parallel across the strain gauge and measuring the recorder pen deflection. This process was repeated for several resistors and the corresponding pen deflections were recorded.

Fatigue Test

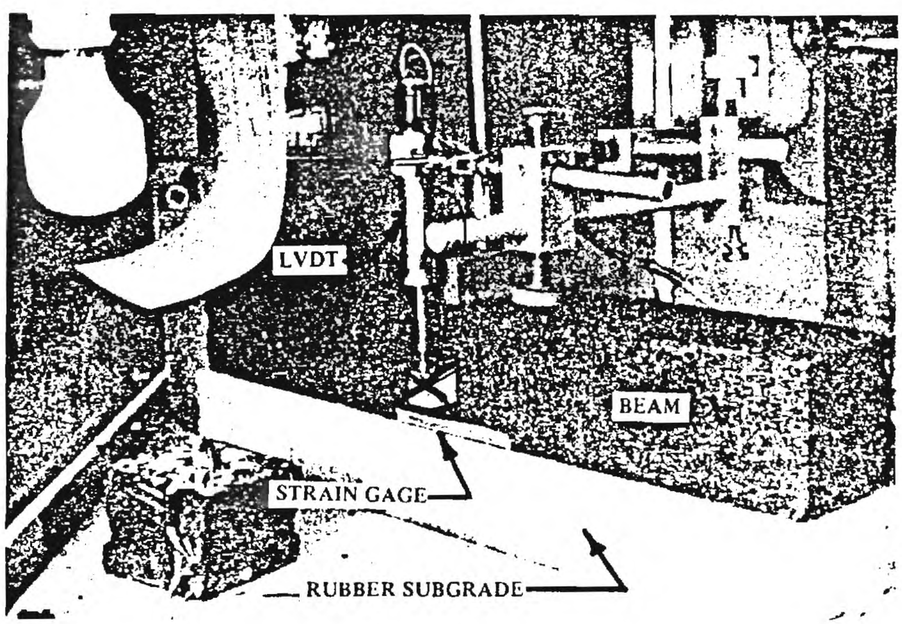
The fatigue test equipment (Fig. 2) consisted of load frame, a 4 in. (102 mm) thick rubber pad supporting the beam, and a pneumatic loading system. The fatigue specimen and rubber support were enclosed within an environmental chamber maintained at $80^{o}F \pm 1^{o}F$ ($27^{o}C$). The rubber pad had a modulus of subgrade reaction of 284 pci (7,861 gm/cc). The rubber pad was supported on the surface ground base of the load frame. The load was applied to the center of the beam using a rigid steel plate which was 1.5 in. (38 mm) wide, 3 in. (76 mm) long and 1 in. (25 mm) thick.

Dynamic strains measured using a wire resistance strain gage were used to calculate the dynamic bending modulus of the asphalt concrete. An epoxy encapsulated wire resistance strain gage (MM type EA-06-6-20CBW-120). was glued parallel to the long axis of the beam, 0.1 in. (2.5 mm) above the bottom. An encapsulated gage was used to prevent damage from moisture during preconditioning. The strain gage was located symmetrically below the center of the load. A temperature compensating strain gage was used to eliminate temperature effects.

The following procedure was used in attaching the gage to the beam:



(a) SCHEMATIC



(b) CLOSE-UP PHOTOGRAPH

FIGURE 2 . FATIGUE TEST APPARATUS.

1. A thin layer of Magnolia quick setting epoxy glue was applied to the beam over the area where the gage was to be placed.

2. After the first layer of epoxy dried, the encapsulated gage was positioned and covered with a thin coating of epoxy.

3. After drying, the epoxy was covered with a coat of Dow-Corning RTV.

No problems were encountered with the strain gage using the above procedure for moisture protection.

The deflection of the center of the beam was measured with a single d.c. LVDT and recorded on a Sanborn, two channel strip chart recorder. The constant load applied to the top of the beam was measured using a 2,500 lb. (11 kN) capacity load cell and monitored with an oscilloscope. The deflection under the load at the center of the beam and the radial tensile strain were measured at 100, 200, 500, 1,000 and approximately 1,500 repetitions. A repeated load of 140 lbs. (623 N) was used in all of the fatigue tests except two tests run early in the testing program.

Before each fatigue test was begun, the rubber pad was removed and the supporting surface beneath was thoroughly cleaned to insure continuous contact at the interface. After cleaning, the pad was replaced and an asphalt concrete beam specimen was carefully centered on the rubber pad. The strain gage leads were then soldered to the connections on the strain gage, and the system checked for continuity. The load foot was positioned inside the previously placed reference marks on top of the beam. The LVDT with the probe in place was positioned on the load foot and was vertically aligned.

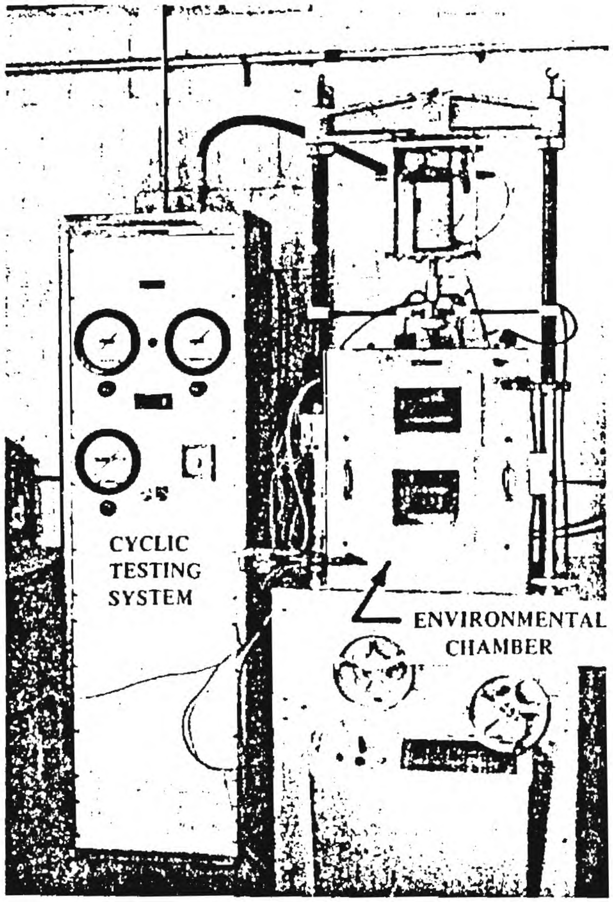
The dynamic bending modulus of the beam was determined after 1,000 load repetitions to allow the beam to stabilize. After 1,500 repetitions, the strain gage and epoxy glue were removed from the side of the beam to eliminate any strengthening effect. The beam was visually monitored for cracks during about the first 2,000 load repetitions. A cut-off switch located on the loading piston was used to automatically stop the test upon failure of the beam. Cracking of the beam initiated in the bottom in the vicinity of the load. This crack rapidly propagates upward, and upon failure the beam usually separated into two parts.

Repeated Load Triaxial Test

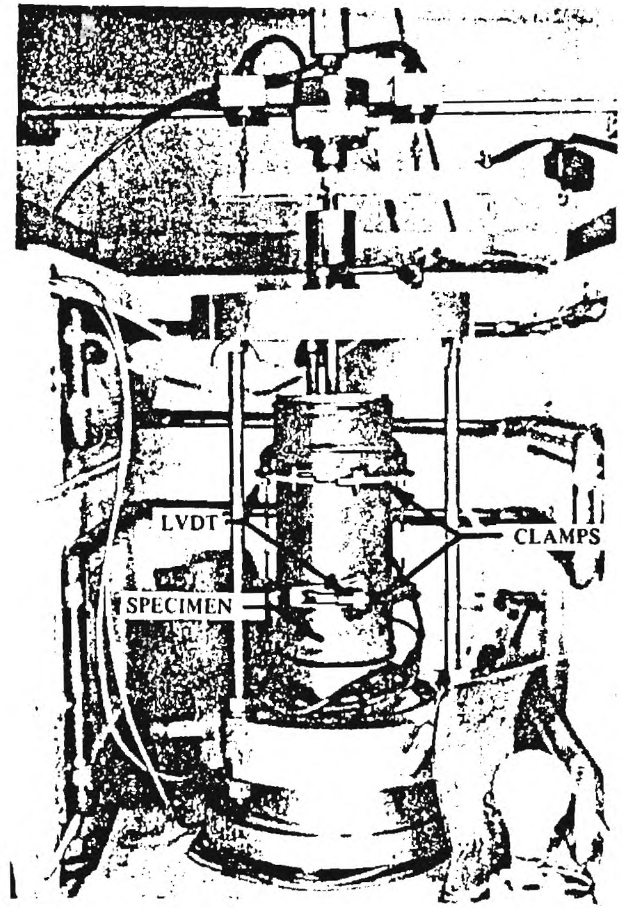
The repeated load triaxial test was used to compare rutting characteristics of the different asphalt concrete mixes. Tests were performed on specimens subjected to a constant confining pressure of 5 psi (34 kN/m^2) and a cyclic deviator stress of 25 psi (172 kN/m^2). All tests were performed at a temperature of $95^{o}F$ ($35^{o}C$). This is about the mean pavement temperature at which rutting in Georgia occurs.

Specimens 4 in. (102 mm) in diameter and 8 in. (203 mm) high were tested in a 6 in. (152 mm) diameter triaxial cell enclosed in a controlled environmental chamber (Fig. 3). Axial strain was measured by placing two clamps on the specimens as illustrated in Fig. 3. The movement between these clamps was measured using two small d.c. LVDT's. High output voltage LVDT's were used to permit measuring the small resilient specimen displacement. The two clamps were placed at a level one quarter of the distance in from each end of the specimen to minimize end effects. The outputs from the inside axial transducers were recorded on a Gould 2200-S two channel, strip chart recorder.

In addition to inside transducer axial displacement measurements, the total axial specimen deformation was measured by a pair of d.c. LVDT's



(a) GENERAL TESTING SYSTEM



(b) SPECIMEN AND LVDT CLAMPS

FIGURE 3. REPEATED LOAD TEST APPARATUS USED IN RUTTING TESTS.

which reacted against a Lucite plate attached to the loading piston outside the environmental chamber (Fig. 3b). The output from these transducers was recorded on a Mosley X-Y recorder. Load was measured by a 2,500 lb. (11 kN) capacity load cell and monitored with an oscilloscope.

The test procedure used for the repeated load triaxial test is summarized as follows: Each specimen was first carefully examined to assure that it was free from defects such as excessive voids due to the presence of large aggregates, and that both ends were flat and parallel. A rubber membrane was then placed around the sides of the specimen. The specimen was positioned on top of a bronze porous stone resting on the bottom loading platen of the triaxial cell. A thin Teflon pad was placed between the end of the specimen and the top platen. A rubber membrane was then pulled up over the top platen, and rubber O-rings were used to seal the membrane to the top and bottom platens.

The inside LVDT clamps were placed around the rubber membranes, and the LVDT probes were set at approximately a negative 3 volts output position. The clamps were placed on the specimen tightly enough to prevent slippage during the test, but not so tight as to exert excessive additional confining stress on the specimen. Once the LVDT's were in place and adjusted, the triaxial chamber was assembled. The environmental chamber was then placed around the cell, and the loading piston inspected to see that it was in alignment with the top platen on the specimen. The top cross-arm of the loading system was lowered so that a small seating load was applied to the specimen.

The pens on the recorder monitoring the LVDT deflections were centered and the test started. Specimen deformation was measured continuously for the first 10 repetitions, and then for a short time at approximately 100, 1,000, 10,000, 50,000 and 100,000 load repetitions. After 100,000 repetitions, the test was terminated. During the test the chamber temperature, cell pressure, pilot valve pressure, deviator stress and load pulse time were observed periodically to insure proper adjustment.

The resilient modulus and permanent (plastic) strain were calculated from the test results as a function of the number of load repetitions.

CHAPTER V

TEST RESULTS AND DISCUSSION

INTRODUCTION

The test results for the following series of tests for each additive tested are presented and discussed in this chapter:

1. Fatigue test results
2. Repeated load rutting tests
3. Static bending tests
4. Tensile splitting tests
5. Lottman rutting tests
6. Georgia DOT boiling test

Fatigue and rutting tests were performed on both as-compacted and preconditioned (artificially weathered) specimens. Preconditioning procedures used in the fatigue and rutting tests are described in Chapter III; the materials tested including stabilizing agent are also described in this chapter. Test procedures used in the repeated load fatigue and rutting tests are presented in Chapter IV.

TEST RESULTS

Fatigue and Rutting

The test results from the fatigue and repeated load rutting tests are presented in Tables 17, 18, and 19. The following results are included in these tables:

1. The number of repetitions to failure and the dynamic bending modulus of the beam fatigue specimens. The number of load

TABLE 17. SUMMARY OF REPEATED LOAD FATIGUE AND RUTTING TEST RESULTS - 0.5% INDULIN AS-1, 1% DRY LIME, AND 2% DRY LIME.

Test No.	Fatigue Test: Cyclic Load (lbs.)	Fatigue Test: N Repetitions to Failure N_f	Fatigue Test: Bending Modulus E_b (psi)	Static Bending: Def. (in.)	Static Bending: Fail. Load (lbs.)	Rutting Test: Resilient Modulus, E_R (psi) 1,000 Reps.	Rutting Test: Resilient Modulus, E_R (psi) 10,000 Reps.	Rutting Test: Resilient Modulus, E_R (psi) 100,000 Reps.	Rutting Test: Permanent Strain, E_p (3), (%) 1,000 Reps.	Rutting Test: Permanent Strain, E_p (3), (%) 10,000 Reps.	Rutting Test: Permanent Strain, E_p (3), (%) 100,000 Reps.
0.5% AS-1 Anti-Strip											
1A-1	140	55,125	137,208 (68,109)	0.028	325	186,570	396,960	913,700	0.478	0.536	~ 0.90 (2.150)
1A-2	140	32,765	80,311 (65,791)	0.023	337	139,630	143,440	1,334,000	0.845	1.028	~ 1.8 (1.814)
1A-3S	140	25,221	69,224 (57,447)	0.049	287	160,723	125,258	78,935	0.331	0.483	0.958 (0.765)
1A-4S	140	60,060	123,684 (75,706)	0.014	425	313,882	398,209	92,000	0.627 (0.420)	0.774 (0.517)	1.255 (0.941)
		43,293 all tests (42,640) soaked									
1% Dry Lime											
1B-1	140	14,482	91,972 (71,782)	0.024	325	197,630	211,750	226,100	0.943 (1.887)	1.095	1.151 (1.972)
1B-2	140	41,697	92,988 (52,900)	0.019	350	44,770	59,030	85,510	1.069	1.713	1.877 (2.323)
1B-3S	140	7,000	45,252 (16,072)	0.019	337	423,492	555,833	130,146	0.543 (.242)	0.620 (.341)	1.547 (0.900)
1B-4S	140	92,917	96,168 (119,125)	0.023	325	181,497	171,500	99,925	0.393 (0.320)	0.455 (0.340)	0.719 (0.560)
2% Dry Lime											
1C-1	140	72,444	137,867 (57,447)	0.020	350	494,070	523,140	177,870	0.477	0.536	0.601 (0.358)
1C-2	140	22,480	84,019 (98,004)	0.041	375	205,231	149,888	105,455	0.422 (0.201)	0.680 (0.298)	0.992 (0.401)
1C-3S	140	125,959	170,729 (98,004)	0.026	375						
1C-4S	140	33,598	118,547 (169,948)	0.026	350						

Notes:
1. The first dynamic bending modulus of the asphalt concrete beam was determined from the strain gage measurement; the second bending modulus, given in parentheses, was determined from the deflection measurement. Both moduli were determined after 1,000 repetitions.
2. Deflections were measured at a load of 100 lbs. (445 kN).
3. The number given not enclosed by parentheses is the permanent strain based on outside transducers; the number given enclosed by parentheses is based on the inside LVDT measurements.

TABLE 18. SUMMARY OF REPEATED LOAD FATIGUE AND RUTTING TEST RESULTS - 1% LIME SLURRY, I-85 RECYCLE AND 3% LATEX.

Test No.	Fatigue Test: Cyclic Load (lbs.)	Fatigue Test: N Repetitions to Failure N_f	Fatigue Test: Bending Modulus E_b (psi)	Static Bending(2): Def. (in.)	Static Bending(2): Fail. Load (lbs.)	Rutting Test: Resilient Modulus, E_R (psi): 1,000 Reps.	Resilient Modulus, E_R (psi): 10,000 Reps.	Resilient Modulus, E_R (psi): 100,000 Reps.	Permanent Strain, E_p(3), (%): 1,000 Reps.	Permanent Strain, E_p(3), (%): 10,000 Reps.	Permanent Strain, E_p(3), (%): 100,000 Reps.
1% Lime Slurry											
1D-1	140	35,856	77,444 (74,232)	0.026	287	266,040	342,051	156,941	0.328	0.386 (0.206)	0.594 (0.349)
1D-2	140	17,416	107,784 (71,783)	0.041	300	114,506	112,101	103,813	0.753 (0.259)	0.942 (0.332)	1.054 (0.401)
1D-3S	140	39,263	131,313 (119,125)	0.019	350						
1D-4S	140	26,349	146,168 (81,364)	0.029	287						
Recycle Mix From I-85											
2A-1	140	59,528	764,544 (107,920)	0.005	375	365,497	337,722	784,706	0.533 (0.172)	0.719 (0.238)	0.861 (0.277)
2A-2	140	800,000+ (not failed)	465,458 (173,715)	-	-	366,484	268,952	231,597	0.550 (0.341)	0.787 (0.459)	1.103 (0.624)
2A-3S	140	140,424	442,541 (386,701)	0.008	425						
2A-4S	140	416,319	287,198 (525,539)	0.016	275						
3% Latex											
3A-1	140	31,359	208,856 (140,254)	-	-	175,526	168,861	116,507	0.437 (0.242)	0.572 (0.309)	0.767 (0.470)
3A-2	140	34,866	160,454 (70,531)	0.012	450	148,222	162,683	134,747	0.495 (0.221)	0.690 (0.246)	0.767 (0.274)
3A-3S	140	2,640	77,844 (14,603)	0.063	275						
3A-4S	140	105,003	121,891 (114,477)	0.016	450						

Notes: 1. The first dynamic bending modulus of the asphalt concrete beam was determined from the strain gage measurement; the second bending modulus, given in parentheses, was determined from the deflection measurement. Both moduli were determined after 1,000 repetitions.

2. Deflections were measured at a load of 100 lbs. (445 kN).

3. The number given not enclosed by parentheses is the permanent strain based on outside transducers; the number given enclosed by parentheses is based on the inside LVDT measurements.

TABLE 19. SUMMARY OF REPEATED LOAD FATIGUE AND RUTTING TEST RESULTS - 20% GRANULATED RUBBER, 30% TRINIDAD LAKE ASPHALT.

Test No.	Fatigue Test			Static Bending (2)		Rutting Test					
	Cyclic Load (lbs.)	N Repetitions to Failure N_f	Bending Modulus E_b (psi)	Def. (in.)	Fail. Load (lbs.)	Resilient Modulus, E_R (psi)			Permanent Strain, ε_p (3), (%)		
						1,000 Reps.	10,000 Reps.	100,000 Reps.	1,000 Reps.	10,000 Reps.	100,000 Reps.
20% Granulated Rubber											
3B-1	140	29,380	173,649 (155,867)	0.15	337	141,915	119,107	104,219	0.359 (0.216)	0.555 (0.320)	0.801 (0.450)
3B-2	140	97,803	173,247 (124,011)	0.026	325	187,887	189,220	134,747	0.550 (0.135)	1.141 (0.384)	1.556 (0.701)
3B-3S	140	80,798	120,849 (66,938)	0.029	275	-	-	-	-	-	-
3B-4S	140	34,764	151,344 (166,284)	0.024	250	-	-	-	-	-	-
30% Trinidad Lake Asphalt											
4A-1	-	-	-	-	-	167,799	122,385	264,158	0.603 (0.255)	1.119 (0.285)	1.370 (0.491)
4A-2	-	-	-	-	-	165,714	156,023	129,515	0.302 (0.161)	0.379 (0.187)	0.538 (0.236)
4A-3S	140	49,900	69,558 (53,773)	0.041	275	-	-	-	-	-	-
4A-4S	140	19,693	93,048 (251,008)	0.029	375	-	-	-	-	-	-

Notes: 1. The first dynamic bending modulus of the asphalt concrete beam was determined from the strain gage measurement; the second bending modulus, given in parentheses, was determined from the deflection measurement. Both moduli were determined after 1,000 repetitions.

2. Deflections were measured at a load of 100 lbs. (445 kN).

3. The number given not enclosed by parentheses is the permanent strain based on outside transducers; the number given enclosed by parentheses is based on the inside LVDT measurements.

repetitions to failure indicates the resistance to fatigue of the specimens.

2. <u>The resilient modulus and permanent deformation at selected numbers of load repetitions</u>. The permanent strain, εp at 100,000 repetitions can as an engineering approximation be considered directly proportional to the amount of rutting which would occur in the field [19]. The resilient modulus is a dynamic compressive modulus of elasticity measured in the rutting test on a cylindrical specimen of asphalt concrete. The resilient modulus is typically about twice as large as the bending modulus. Probably the actual modulus of elasticity of the asphalt concrete is somewhere between the bending and resilient moduli values.

Figure 4 shows the relationship between the number of repetitions to failure and the measured bending modulus obtained from the fatigue test. A fair correlation is seen to exist between these two variables. A similar correlation has been found in other studies [19].

A condensed summary of the fatigue and rutting test results is given in Table 20. This table is suitable for easily comparing the beneficial effects on fatigue performance of the various additives. As discussed in the next chapter, the presence of weak aggregate was an important factor contributing to the relatively large variations in fatigue and rutting test results. Supplementary fatigue test results are given in Table 22.

Static Bending

The correlation observed between the dynamic bending modulus and fatigue life (Fig. 4) suggests a static bending test, which is an indication of static bending stiffness, might also be an indicator of fatigue life. To investigate this possibility a static bending test was performed on the larger half of the 20 in. (508 mm) long beam after it had been failed in the fatigue test. The beam was simply supported and had a clear span of 8 in. (203 mm). Load was applied at the center in 25 lb. (111 N) increments until failure. The center deflection of the beam was measured at loads of 50, 100 and 150 lbs. (222, 445, and 668 N).

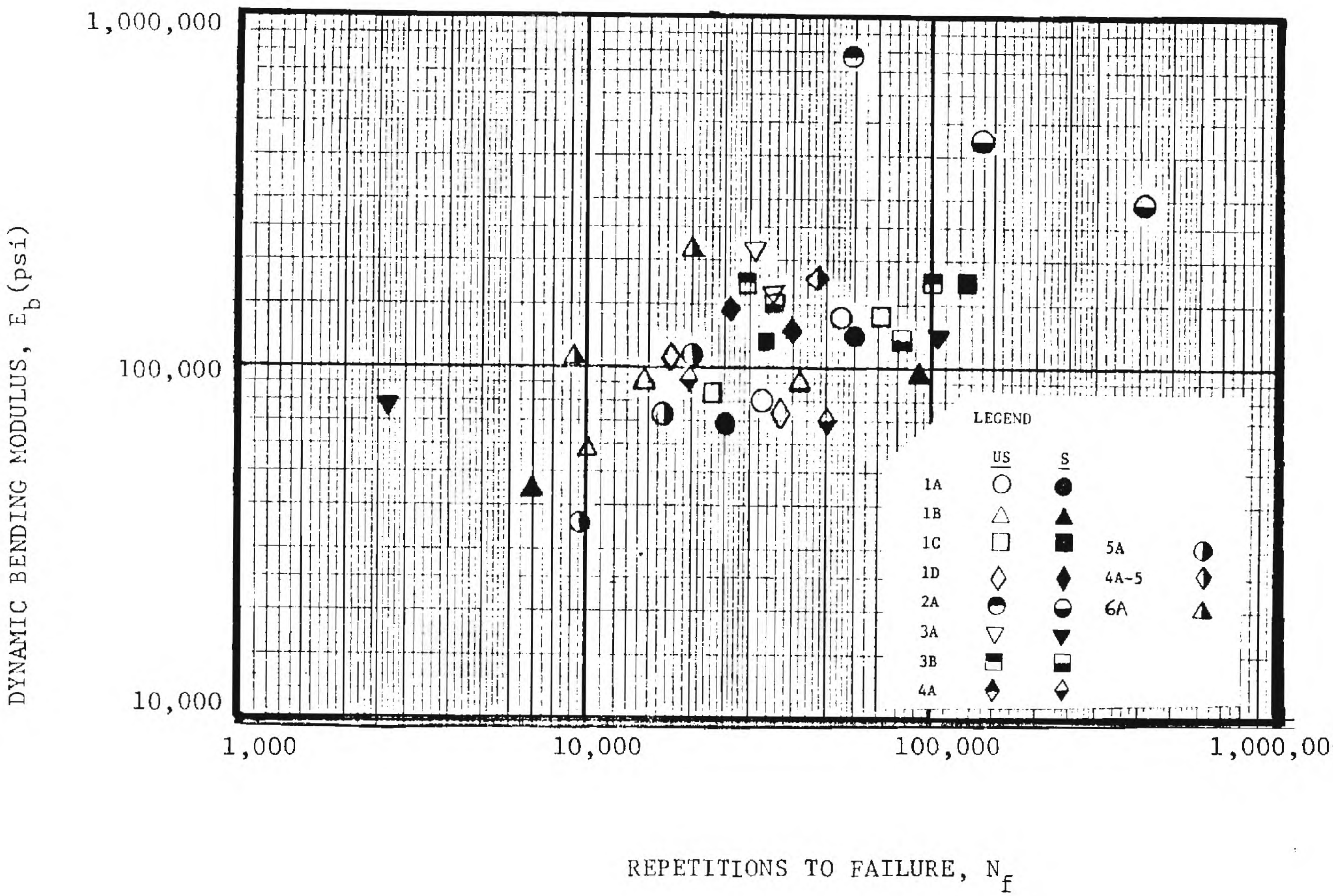


Figure 4. CORRELATION BETWEEN REPETITIONS TO FAILURE AND DYNAMIC BENDING MODULUS - FATIGUE TEST.

TABLE 20. COMPARISON OF AVERAGE TEST RESULTS.

Additive	Fatigue Life (reps.)		Bending Modulus (psi)		Rutting Permanent Strain[3] ε_p (%)	
	All Tests	Preconditioned	All Tests	Preconditioned	Dry	Precond.
Preconditioning: 30 min. vacuum to 80% saturation - 30 minute soak						
0.5% AS-1	43,300	42,600	102,000	96,000	[4] ~0.9	1.11
1% Dry Lime	39,000	50,000	73,200	70,800	[4] ~1.1	1.13
2% Dry Lime	63,600	79,800	127,800	144,600	0.80	-
1% Lime Slurry	29,700	32,800	115,700	138,800	0.82	-
Recycle	353,000	278,300	489,900	374,900	0.98	-
3% Latex	43,500	53,800	142,300	99,900	0.77	-
20% Granulated Rubber	60,700	57,800	154,700	136,000	1.18	-
30% Trinidad	-	34,800	-	81,300	0.95	-
Special Preconditioning						
No Additive	-	15,400	-	-	-	-
1% Dry Lime	-	14,600	-	-	-	-
30% Trinidad	-	47,100[1]	-	-	-	-

Notes: 1. Only one test was performed on the 30% Trinidad, Saturate-Freeze-Soak specimens.
2. 30 min. vacuum to 80% saturation; freeze at 0°F for 15 hrs.; soak at 140°F for 4 hours.
3. Permanent strain, ε_p, was determined after 100,000 load repetitions.
4. Neglecting one high value, results in line with average for preconditioned specimens.

The static beam test results are given in Tables 17-19. The correlation between fatigue life as measured in the fatigue test and static beam deflection at 50 lbs. (220 N) and 100 lbs. (445 N) load is shown in Figs. 5 and 6, respectively. At a load of 50 lbs. (220 N), a reasonably good correlation was found between static beam deflection and fatigue life (Fig. 5). At a load of 100 lbs. (445 N), however, the correlation between fatigue life and static beam deflection was relatively poor (Fig. 6).

Tensile Splitting, Lottman Rutting and Boil Test Results

The tensile splitting, Lottman Rutting and Georgia DOT Boiling Test results are summarized in Table 21. In this table the strength ratio in percent is the ratio of the splitting strength of preconditioned (weathered) specimens to dry (unweathered) specimens. A comparison of the strength ratios of different mixes thus gives an indication of the beneficial effect an additive may have on reducing stripping.

In the Lottman Rutting Test a larger number indicates a greater resistance to rutting. Boil test results are also given in Table 21 expressed as a percent of the aggregate surface area over which stripping did not occur (percent retained asphalt coating). A panel of three raters were used to estimate the retained coating for each additive. Another series of Boil tests (not reported here) indicated that only the specimen with 0.5 percent Indulin AS-1 had 95 percent or more retained coating; these results are in agreement with those shown in Table 21. In the series of Boil tests not reported, the results were recorded as either having a retained coating of more or less than 95 percent.

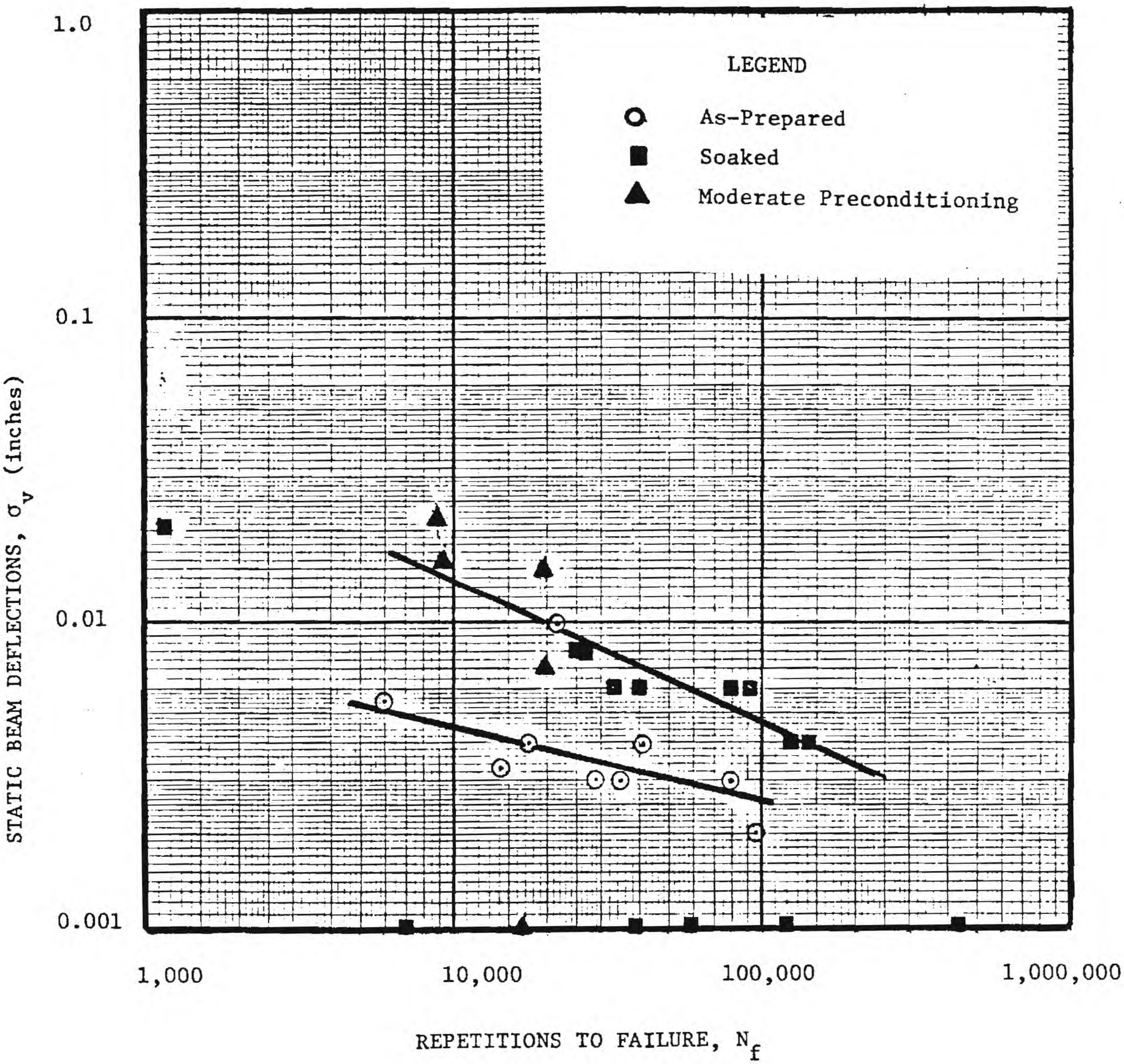


FIGURE 5. CORRELATION BETWEEN STATIC BEAM DEFLECTION AND REPETITIONS TO FAILURE - 50 LB. LOAD.

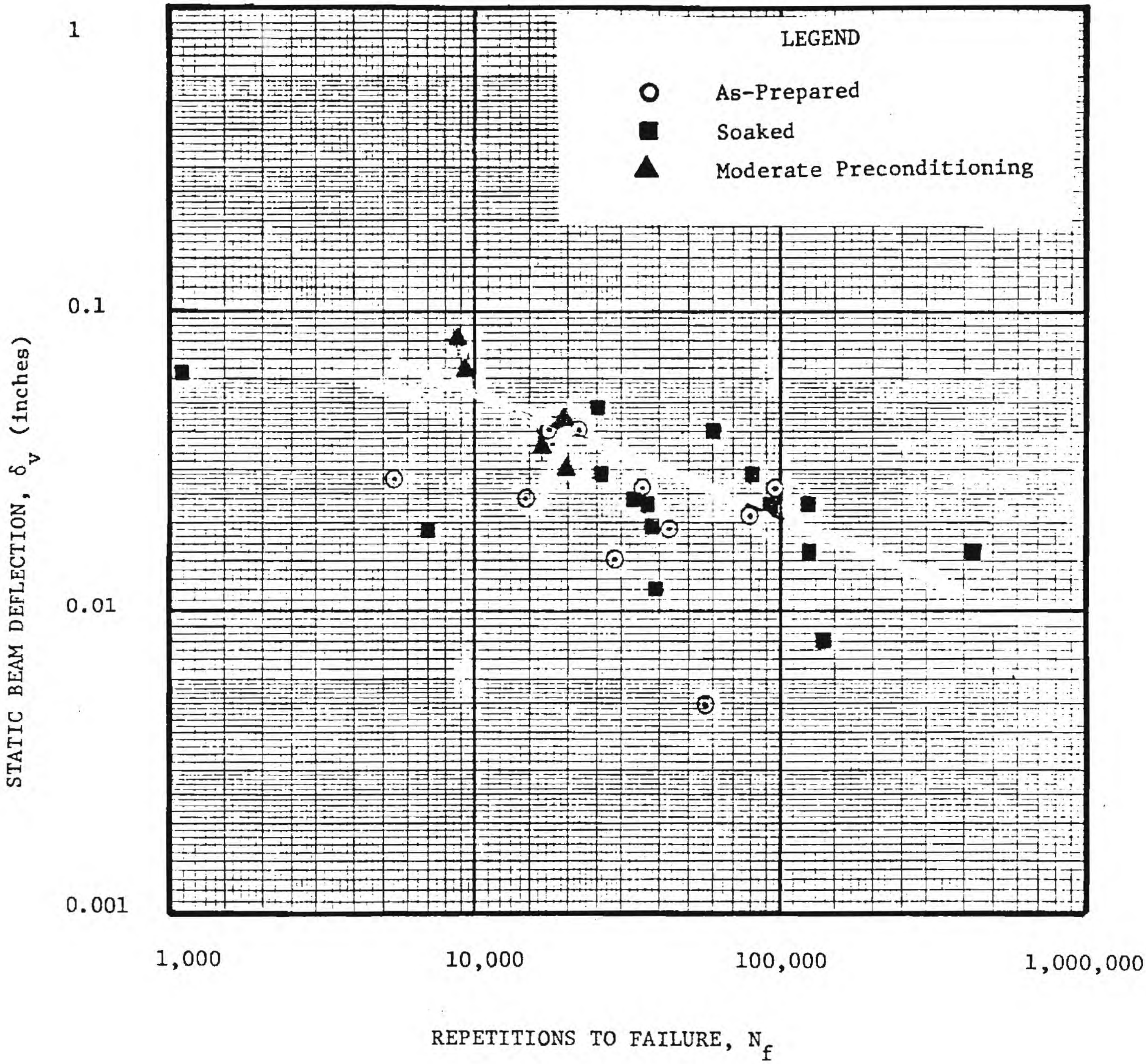


FIGURE 6. CORRELATION BETWEEN STATIC BEAM DEFLECTION AND REPETITIONS TO FAILURE - 100 LB. LOAD

TABLE 21. SUMMARY OF TENSILE SPLITTING, LOTTMAN RUTTING AND GHD BOIL TEST RESULTS.

MIX	SOURCE	TENSILE SPLITTING				MIX PROPERTIES			RUTTING	BOIL TEST
	Vulcan Materials Co. at Kennesaw, Ga.	AC Content (%)	Dry Strength (psi)	Wet Strength (psi)	Ratio Freeze-Thaw(psi) / Dry (psi)	Air Voids (%)	No. of Blows	Inches of Mercury	Dry 77°F at 0.10"/min. (psi)	Retained Coating (%)
E	No Additives - Plain	5.8	72.6	23.0	32	5.8	19	23	34.3	75
E	0.5% Indulin AS-1 Additive	5.9	59.7	54.3	91	6.2	17	22	27.7	95+
E	30% Lake Asphalt 70% AC-30	6.1	88.5	25.5	29	5.1	23	23	37.6	80
E	1% Lime Filler	5.7	95.6	83.8	88	5.2	21	23	35.5	85
E	2% Lime Filler	5.5	97.6	83.1	86	5.7	20	23	33.8	90
E	1% Lime Slurry	5.8	97.4	87.4	90	7.2	17	22	31.1	85
E	5% Latex 95% AC-30	5.8	109.8	26.2	24	6.0	23	16	20.2	80
E	20% Rubber 80% AC-20	8.5	77.0	25.3	33	6.6	24	22	42.6	90
E	I-85 Recycle Mix	5.5	125.5	114.0	90.8	5.0	16	20	62.7	-

CHAPTER VI

DISCUSSION

A condensed summary of the main fatigue and rutting test results is given in Table 20. The data presented in Table 20 are the average of all of the main series of fatigue and rutting test results. Primarily because of the presence of weak aggregates in the mixes, the scatter in test results is quite large. The supplementary fatigue test results are given in Table 22. The aggregates used in the supplementary tests in general were much better than the main fatigue tests. As a result the experimental findings for this series of tests are much more consistent.

Fatigue Tests

Type Test - Constant Air Voids or Asphalt Content. Fatigue life is significantly affected by both the amount of air voids and the asphalt content. Fatigue life increases with increasing asphalt content and decreasing air voids [19]. In comparing the fatigue life of mixes, usually either one or the other of these two variables are held constant.

All of the beam fatigue tests were performed on specimens prepared at 7 percent air voids. A constant air voids content of 7 percent was used to allow easy penetration of water into the specimen. An alternate would have been to use a constant asphalt content. Table 1 summarizes selected physical properties of each mix used in the fatigue tests at 7 percent air voids. These properties were interpolated from the mix design test results (Tables 3 through 10).

With the exception of the mix with 20 percent granulated rubber and the I-85 recycle mix, the asphalt content of all the mixes used in the fatigue tests were within ± 0.2 percent of the average value. This variation in asphalt content should be considered in interpreting the test results.

Stress Concentration. In a fatigue test the location of a pronounced point of stress concentration near the bottom tensile side of the specimen can have a significant negative effect on the fatigue life. Stress concentration can be caused by larger than average air voids, stripped aggregate, or broken or weak aggregates. Even fatigue test results on asphalt concrete specimens prepared using good aggregates (those that are hard and not susceptible to stripping) usually exhibit a relatively large amount of variation, at least partly due to the way voids are randomly formed in the mix during placement and compaction of the asphalt concrete.

The presence of two large aggregates together in the bottom of the beam near the center would, for example, result in a larger than average void, and hence lead to a premature fatigue failure of the specimen. The presence of a large amount of weak aggregate in the main series of tests undoubtedly played a major role in accounting for the scatter in test results.

Mica Content. The Kennesaw Quarry aggregate has a very high mica content. One Georgia DOT study, performed before the present study was begun, indicates the biotite mica content of the aggregate tested to be about 17 to 25 percent. A visual examination of the beam surfaces which failed in fatigue indicates the amount of visible mica on the failure surface varies depending upon the additive used. This finding was observed for both the main and the supplementary series of fatigue tests.

The variation of biotite mica on the failure surface with type additive used suggests that either (1) the degree of coating of mica with asphalt may depend on the type additive used, or (2) the mica content was variable between the aggregates used for the different additive mixes. Because of the high mica content, an absence of coating on the mica might result in a significantly reduced fatigue life.

Fatigue Test Variability. In the fatigue tests of this study the relatively large observed variability was very likely caused by a combination of the following factors: (1) presence of weak aggregate, (2) variable coating of the mica and/or aggregate, and (3) location of larger size aggregate which have undergone stripping with respect to the fatigue failure surface. The random location of air voids of varying size would also affect fatigue life.

In the main series of fatigue tests, the aggregates were quite bad with several tests having as much as 20 percent broken aggregate visible on the failure surface; a relatively large but variable amount of broken aggregate was present in most but not all of the tests. Observed stripping on the failure surface was also quite variable. In the main series of tests both weak aggregate and stripping (within a series of tests using the same additive) were considerably more variable than in the supplementary test series. All of these factors make interpolation of the fatigue test results from the main test series very difficult in terms of the effect of the additive on fatigue life.

Where weak aggregate or a large variation of aggregate quality exists, the use of at least 5 and preferably 10 beam specimens would be desirable. A statistical analysis could then be performed on the results.

Supplemental Fatigue Tests. A relatively large scatter in fatigue test results were observed in the main series of tests. Therefore, supplementary fatigue tests were performed on specimens having the following anti-strip additives: (1) 1 percent dry lime, (2) 2 percent dry lime, and (3) 0.5 percent AS-1.

The materials used (Kennesaw aggregates), gradations and other properties were the same as those used in the earlier tests and are described in Chapter III. The aggregates, however, were not obtained from the quarry at the same time as for the original series of tests; the amount of soft particles present was much less than for the first test series. Three replicate beam fatigue specimens each having anti-strip additive were prepared and tested.

To more nearly simulate field conditions, the moderate preconditioning sequence was followed described in the previous section. The fatigue test results from this supplemental series are presented in Table 22.

Aggregate stripping on the failed surface of all specimens tested in this series was less than about 5 percent. Also, with the exception of the 1 percent dry lime specimens, aggregate breakage was not a very serious problem. Therefore, it is felt that comparisons of fatigue performance of mixes having different additives are reasonably valid in the supplementary test series.

Although scatter did occur in the test results, clear trends were observed. The 2 percent dry lime specimens did significantly better than all the other three anti-strip additives, followed by the 1 percent lime slurry treated specimens. The 1 percent dry lime and the 0.5 percent AS-1 treated specimens performed relatively poorly compared to the 2 percent dry lime and 1 percent lime slurry specimens. On the average, the 1 percent dry lime specimens appeared to perform a little better than the AS-1 treated specimens, although important scatter in data were present.

Table 22. Supplementary Beam Fatigue Test Results - Moderate Preconditioning.

Treatment	Specimen	Bulk S.G.	Unit Weight (pcf)	Target Unit Weight (pcf)	Fatigue Life Reps. to Failure	Avg. Fatigue Life (Reps.)
1% Dry Lime	1	2.42	151	148	29,748	
	2	2.44	152	148	1,646	17,000
	3	2.38	149	148	19,542	
2% Dry Lime	1	2.38	149	148	340,700	
	2	2.39	149	148	836,487	622,000
	3	2.43	152	148	687,564	
1% Lime Slurry	1	2.38	148	148	170,563	
	2	2.35	147	148	90,599	124,000
	3	2.39	149	148	110,792	
AS-1	1	2.37	148	148	14,625	
	2	2.42	151	148	10,587	11,000
	3	2.39	149	148	7,243	

In the supplementary fatigue tests, a concentration of broken aggregates occurred in the 1 percent dry lime specimens. An aggregate breakage problem of this severity was not observed in the other mixes. Breakage of aggregate, it is felt, caused the 1 percent dry lime speicmens to appear weaker in fatigue, relative to mixes with other additives, than would be the case if similar strength aggregate were present in all the mixes. Nevertheless, the 1 percent dry lime mixes exhibited a better fatigue performance than the 0.5 percent AS-1 specimens. Also, the AS-1 specimens appeared to exhibit slightly more stripping than the 1 percent dry lime mixes. The original tests did not show any obvious difference between the AS-1 and 1 percent dry lime mixes. Weak aggregate on the failure planes, however, probably had an important effect on the observed fatigue life.

A relatively large difference in fatigue life was observed between the 1 percent dry lime and lime slurry treated specimens. As large of a difference in fatigue life would not be expected had aggregates of similar characteristics been located on the failure surfaces of the beams for the two type mixes.

A visual examination appears to indicate that less uncoated mica is evident on the failed surfaces of the 2 percent dry lime specimens than the others studied in the supplementary study. Also, the 1 percent lime slurry specimens appeared to have less uncoated mica than the 1 percent dry lime and the 0.5 percent AS-1 specimens. Coating of the mica with asphalt may prove to be an important factor in developing fatigue resistance of mixes. Also, different additives and levels of the additive used may result in different levels of coatings of the mica. These hypotheses of course need to be verified through further study.

Recycle Mix

The recycle mix demonstrated by far the best fatigue characteristics. Probably this was at least partly due to the mix having been subjected to two cycles of cooling and reheating. The other mixes were not subjected to any cooling-reheating cycles. Cooling and reheating significantly increases the stiffness of the mix, and as a result the fatigue life is increased other factors being equal. Further, less aggregate on the failure surface broke than for most of the other tests.

The recycle mix should exhibit good fatigue properties in the field. However, since the other mixes were not subjected to a similar cooling and reheating cycle, a direct comparison of the mixes might be misleading. In the future the recommendation is made that all fatigue specimens be heated to an elevated temperature and cooled before fatigue testing to cause some hardening of the asphalt cement to occur.

Preconditioning Effect on Fatigue

The modest preconditioning used as a standard testing procedure consisted of (1) 30 minutes of partial vacuum saturation at the vacuum level required to give 80 percent saturation (about 22 inches of mercury), followed by (2) a 30 minute soaking period (not under a vacuum). The mild preconditioning level selected for the main series of tests was probably not severe enough to show the full effects of stripping. A visual examination of the failed surface of fatigue test specimens substantiates this postulation; little stripping was observed on either the preconditioned or unconditioned specimens. A more severe level of preconditioning was not used for the main test series because it was felt before the study was begun that specimens prepared using Kennesaw aggregate might not hold together for fatigue testing. This decision was based on the results from preconditioned, indirect tension tests previously performed by the Georgia DOT.

Moderate Preconditioning. To further investigate the effect of a more severe level of preconditioning on fatigue life, a special limited series of fatigue tests were performed on more severely preconditioned specimens. The preconditioning consisted of (1) the standard, partial vacuum saturation procedure previously described, (2) 15 hours of freezing at $0^{o}F$ ($-17.8^{o}C$) followed by (3) four hours of soaking in a water bath at $140^{o}F$ ($60^{o}C$). All of the supplementary fatigue tests were also performed on specimens preconditioned in this manner.

In the future, the recommendation is made that specimens should also be first heated for about 24 hours at $140^{o}F$ ($60^{o}C$) before beginning the preconditioning sequence given above. Further, consideration should be given to using either a longer time in the water bath at $140^{o}F$ (8 to 24 hours) or else multiple cycles of freezing and soaking.

Rutting

Vertical permanent strain in the cylindrical asphalt concrete specimens tested is approximately proportional to permanent deformation. The rutting tests were carried out to 100,000 load repetitions. The observed relatively large variation in rutting in these tests would be due primarily to (1) difference in quality of aggregate between mixes, (2) differences in admixtures, (3) difference in asphalt content (void ratio was held constant), and (4) sample preparation technique.

The average permanent strain observed in the mixes at 100,000 load repetitions was about 0.94 percent with the variation between the extremes of the adjusted average data for unsoaked specimens being + 0.24 and

- 0.14 percent strain (refer to Table 20).

Considering the influence of aggregate variability and differences in asphalt content, it is felt that the additives studied probably do not have a very big effect on asphalt mix rutting. This data suggests that the 20 percent granulated rubber mix would very likely exhibit more rutting than the typical mix. The relatively high asphalt content in the 20 percent granulated rubber mix would account for the higher level of rutting. Further work using a consistent quality aggregate would be required to establish a more precise comparison between different admixtures.

Comparison of Fatigue and Tensile Splitting Test Behavior

Fatigue Performance. The best mixes for resisting fatigue, as defined by the repeated load fatigue test, in descending order of observed performance are approximately (1) the recycle mix (as indicated by the main test series) and the 2 percent dry lime, (2) the one percent lime slurry, (3) the 1 percent dry lime, and (4) the 0.5 percent Indulin AS-1. Also, the 20 percent granulated rubber specimens appeared to do well as indicated by the main test series. This good fatigue performance was probably due to the high asphalt content of the mix.

Because of the presence of weak aggregates in all the fatigue tests using the 1 percent dry lime, the exact ranking could not be determined; it more than likely would perform better had strong aggregate been present. Finally, specific rankings for the 3 percent latex and 30 percent Trinidad mixes could not be established because of the observed variability due primarily to the weak aggregate.

Where marginal granite-gneisses are encountered having high mica and quartz contents, the use of 2 percent dry lime appears most promising for improving overall fatigue resistance. Recycled mixes should also show good fatigue performance. More testing using specimens having similar levels of preconditioning are required to establish if the recycle mix would exhibit better performance than the 2 percent dry lime mix.

Tensile Splitting and Boil Test Behavior. The GHD Boil Test and tensile splitting test strength ratio indicates that the Indulin AS-1 treated mixes should have the most resistance to stripping, followed by the addition of lime (either dry lime or lime slurry). The Indulin AS-1 treated mix had both the highest tensile splitting test strength ratio and also the greatest retained coating on the aggregate in the boil test.

The GHD Tensile Splitting Test results indicate (in order of decreasing stripping resistance) that the 20 percent granulated rubber specimens without any additives, 30 percent Trinidad Lake asphalt and the 3 percent latex rubber all performed poorly.

Combined Results. Consider now the combined results of all three tests (i.e., fatigue, tensile splitting test and boil test). The 2 percent dry lime specimens were the only ones to exhibit excellent performance ratings from all three tests. In two out of the three tests the 0.5 percent Indulin AS-1 and 20 percent granulated rubber specimens also demonstrated excellent performance. The I-85 recycle mix is not included in this discussion.

In the fatigue tests the Indulin AS-1 treated specimens did not perform as the tensile splitting test and GHD Boil test indicated they should have. It should be remembered, however, that the fatigue test does not evaluate resistance to just stripping but the overall performance of the mix under simulated conditions of repeated flexing at a relatively low stress level.

Also, the influence of material variability including aggregate strength and susceptibility of the aggregates to stripping undoubtedly varied both within the individual series and from one series to another. Therefore, the observed relative performance of the different mixes at best are approximate. Further research is needed in both the laboratory and field to better define the actual behavior of mixes made using aggregates having properties similar to those from Kennesaw Quarry. Factors such as the presence of stress concentration points (larger voids and weak or stripped aggregate) will have a significant effect on fatigue performance. To simulate weathering and environmental effects, mixes prepared using marginal aggregates should be preconditioned.

In contrast to the fatigue test, the tensile splitting test applies a slow single load until failure. The load at failure is the only parameter measured that is used to evaluate the performance. This test is convenient for routine work, but does not faithfully duplicate fatigue type loading. A stiffness value (force divided by displacement) would probably be more appropriate than using just the failure load.

The boil type of test considers only stripping of the asphalt coating from the aggregate. It does appear to give a general indication of mix resistance to stripping, and hence may indicate general durability. The boil test does not indicate how the overall mix will perform with respect to fatigue.

From this discussion it follows that none of these tests can stand by themselves as a single method for evaluating asphalt concrete mixes made from marginal aggregates such as those found at Kennesaw Quarry. Certainly more research is needed to define the most appropriate test procedures.

SUMMARY

The fatigue, indirect tensile test, GHD Boil test and repeated load rutting tests were performed to evaluate the potential performance of selected E mixes. Varying levels of artificial weathering were used in these tests. All mixes (except the I-85 recycle mix) were prepared using Kennesaw aggregate which was known to have stripping problems.

With respect to potential fatigue/stripping performance, only the 2 percent dry lime mix received an excellent rating in all three tests. In two out of three tests the 0.5 percent Indulin AS-1 anti-strip and 20 percent granulated rubber specimens appeared to demonstrate good performance.

The presence of a variable but relatively large quantity of soft aggregate in the specimens tested in fatigue was an important factor in causing a significant scatter of results in the main series of fatigue tests (Tables 17 through 20). The Kennesaw aggregates tested in the main series of fatigue tests were considered at best marginal for use in an asphalt concrete mix because of the soft aggregate. The occurrence of weak aggregate was not nearly as great of a problem in the supplementary fatigue tests performed on moderately preconditioned specimens (Table 23).

The fatigue tests indicate that the I-85 recycle and the 2 percent dry lime mixes should demonstrate good fatigue characteristics. In the supplementary fatigue tests the E mix tested, when treated with 1 percent dry lime, did not perform as well as the 1 percent lime slurry mix. The 1 percent dry lime mix did perform better than the Indulin AS-1 treated mix in this test series. The 1 percent dry lime treated mix, however, had a substantial amount of weak aggregates on the failure surface. If the 1 percent dry lime mix had been composed of aggregates of comparable quality to the other mixes, undoubtedly better performance would have been observed.

Finally, an examination of the failure surfaces of the fatigue test specimens suggests that the degree of coating of biotite mica may possibly be related to fatigue life as well as the level of stripping.

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

The performance was studied of six asphalt concrete surface E mixes. These mixes were prepared using a crushed granite gneiss aggregate from the Kennesaw quarry. The granite gneiss had about 17 to 25 percent biotite mica and 15 to 20 percent quartz. The Kennesaw aggregate is known to be a marginal aggregate susceptible to stripping.

Additives used in the E mix were as follows:

1. Anti-Strip Indulin AS-1 (0.5 percent)
2. Dry lime (1 and 2 percent of aggregate)
3. Lime slurry (1 percent of aggregate)
4. Latex Emulsion (3 percent residual latex, based on binder)
5. Granulated Rubber (20 percent of binder)
6. Trinidad Lake Asphalt (30 percent of binder)

In addition, a recycle mix from I-85 was studied that used 60 percent new material and Norcross aggregate. Fatigue tests, rutting tests, the Georgia DOT Tensile splitting test and Georgia DOT boiling tests were all performed on the E mixes.

Main Test Series. In the main series of tests, fatigue and tensile splitting tests were performed on both "as prepared" specimens and specimens subjected to preconditioning. Rutting tests were performed on all E mixes without any preconditining. Rutting tests were also performed on preconditioned specimens having 0.5 percent AS-1 and 1 percent dry lime. A relatively mild level of preconditioning was used for most of the fatigue tests and, when used, on the rutting tests. The mild preconditioning consisted of vacuum saturating each specimen to 80 percent saturation, followed by a 30 minute soaking period.

A limited number of fatigue tests were also performed on selected specimens more severely preconditioned. The more severe level of preconditioning consisted of (1) the 80 percent partial vacuum saturation preconditioning just described, (2) freezing for 15 hours at 0°F (-17.8°C), and (3) soaking in a hot water bath for 4 hours at 140°F (60°C).

Tensile splitting test specimens were preconditioned by (1) 80 percent partial vacuum saturation, (2) freezing for 15 hours at 0°F (-17.8°C), (3) soaking in a hot water bath for 24 hours at 140°F (60°C).

Sample preparation and testing procedures were previously described in Chapter IV. The tensile splitting tests, Lottman rutting tests and GHD Boil tests were performed by the Georgia Department of Transportation.

Weak Aggregates. An examination of the failure surfaces of the beam fatigue test specimens indicated a large but variable number of broken aggregates in the main test series. In some specimens up to about 20 percent of the aggregate on the fatigue failure plane were observed to be broken. The presence of the weak aggregate was an important factor in the observed scatter in fatigue test results in the main test series. Because of the presence of the weak aggregates, the Kennesaw aggregate used in the fatigue tests in the main test series is considered by the author at best to be marginal for use in an asphalt concrete mix. Aggregate quality, of course, varies greatly within a quarry, and the typical Kennesaw aggregate would in general be expected to be of much higher quality.

Supplemental Fatigue Tests. To help to clarify the fatigue results from the main test series, additional fatigue tests were performed on 1 percent dry lime, 2 percent dry lime, 1 percent lime slurry and Indulin AS-1 treated mixes. The aggregate used was once again from the Kennesaw Quarry, but in general was of a much higher quality. Three replicate fatigue tests were performed on each mix.

CONCLUSIONS

1. The fatigue tests indicate that the recycle mix should perform quite well with respect to fatigue. Further, rutting should not be a problem in this mix.

   The recycle mix demonstrated the best fatigue characteristics of any mix tested in the main fatigue test series. The hardening that occurred during the two reheating cycles which the mix was subjected to is believed to account for at least some of its superior performance. Also, very little weak aggregate was present on the failure surface of this mix as compared to the other mixes tested in the main fatigue test series. Because of these factors, a direct comparison between the recycle and other mixes is not recommended.

2. All three type tests (fatigue, tensile splitting and Boil test) indicate that the addition of 2 percent dry lime is a promising additive for reducing stripping and increasing fatigue life; this is the only additive that received an excellent rating for all three tests. Also, only the recycle mix had a greater fatigue life than the 2 percent dry lime mix in the main test series; in the supplementary fatigue tests the 2 percent dry lime mix performed best. The 2 percent dry lime mix also exhibited a below-average tendency to rut.

3. The supplementary beam fatigue tests performed on moderately preconditioned specimens indicate the fatigue performance of the 1 percent dry lime mix to be better than the 0.5 percent Indulin treated specimens (16,980 repetitions to failure compared to 10,818 repetitions).

4. The tensile splitting and Boil test results both indicate that probably no significant difference should exist between the performance of E mixes stabilized with 1 percent dry lime compared with 1 percent lime slurry. This was also indicated in the main series of fatigue tests, but the scatter in test results was great due partly to the presence of weak aggregate.

In the supplementary fatigue tests, the 1 percent lime slurry mixes performed significantly better than the 1 percent dry lime mixes. A concentration of weak aggregate on the failure surface, however, occurred in the three mixes having 1 percent dry lime. Undoubtedly the performance of the dry lime mix would have been substantially better had aggregates of similar quality been present.

5. The addition of 0.5 percent Indulin AS-1 and 20 percent granulated rubber received an excellent rating in two of the three tests. Also, the 20 percent granulated rubber specimens demonstrated good fatigue properties in the main fatigue test series; only the recycle mix and 2 percent dry lime mix exhibited better fatigue characteristics than the granulated rubber mix in the main test series. The scatter of results in this test series was great, however, primarily due to the presence of weak aggregate.

6. The tensile splitting test indicated that the use of 0.5 percent Indulin AS-1, 1 percent lime slurry, 1 percent dry lime and 2 percent dry lime all had about the same beneficial effect in increasing the retained specimen strength. These additives gave the best performing mixes as defined by the tensile splitting test.

7. The Georgia DOT Boil Test indicated the Indulin AS-1 to have the greatest resistance to stripping (95 percent retained coating). The 20 percent granulated rubber and 2 percent dry lime demonstrated the next best performance (90 percent retained coating). A specimen without any additive showed the poorest performance with 75 percent retained coating.

8. The fatigue test is a sensitive indicator of potential mix problems caused by not only mix design variables but also variables such as weak aggregate and the amount of stripping which occurs on the failure plane. Points of stress concentration such as weak or stripped aggregate, uncoated mica, or larger than average air voids all appear to have an important effect on fatigue performance.

9. The failure surface of the best performing fatigue specimens visually appeared to have less uncoated biotite mica than the poorer performing mixes. This in general was true for both the supplementary and main series of fatigue tests. If this observation is indeed true, both the amount of mica present and the effectiveness of the additive to coat the mica may be important factors. The other explanation for the observed differences in mica on the failure surfaces would be that the mica content was varying. Such a variation does not seem likely for both the main and supplementary tests.

   A somewhat limited study indicates that both stripping and aggregate breakage occurrs on aggregates primarily composed of quartz.

10. The fatigue test, boil test and tensile splitting test tended to give somewhat different results. Some of the observed difference may have been due to differences of aggregate quality in the specimens. Aggregate samples were obtained from the quarry at different times for the different tests. Also, the tests in general are fundamentally different in concept. The boil test examines only aggregate coating. On the other hand, the fatigue test considers overall performance of the mix. The tensile splitting test is basically an attempt to simplify the fatigue type test to a more practical test for routine production testing.

    In the splitting tensile test a specimen is subjected to a single, gradually increasing load to failure. The failure load is then used to determine the ratio of strength of preconditioned to unconditioned specimens. In the fatigue test a load is repeatedly applied at a stress level considerably less than that required to cause failure under a single loading. Also, the specimen is supported on a simulated base/subgrade and is subjected to bending. Very likely the use of specimen stiffness (force divided by displacement) would be a better indicator of performance in the splitting test than strength. It is felt that the fatigue test more closely simulates conditions in the field than does the tensile test.

    Further, a study by Kennedy, et al. [10] indicated that the Texas boiling test (similar to the GHD Boil Test) correlated better with stripping than the Lottman Tensile Splitting Test. This is further evidence that the tensile splitting test may not be as representative as other test methods; other studies, however, have indicated the tensile splitting test to have reasonably good correlation with observed behavior.

RECOMMENDATIONS

1. Consideration should be given until further field and laboratory results are developed to using dry lime at higher levels than 1 percent in aggegates similar to those found at Kennesaw Quarry, known to have severe stripping problems. The addition of 2 percent dry lime was found to significantly increase performance by all three test methods.

2. To improve mix performance if the economics appears favorable, the potential use of 20 percent granulated rubber should be investigated further. The good fatigue performance was probably at least partly caused by the greater asphalt content used in the mix. This mix would, however, be very likely more susceptible to rutting.

3. In future fatigue test studies to evaluate the long-term durability of asphalt concrete mixes including stripping, the specimens should be subjected to moderate to severe preconditioning. Specimen preconditioning should include full

vacuum saturation, freezing and then soaking for perhaps as long as 8 to 24 hours in a 140°F hot water bath. A more time consuming alternate to more severe preconditioning would be to use multiple cycles of preconditioning. Also, before the above sequence of preconditioning is begun, specimens should be first heated for about 24 hours at 140° F.

Fatigue tests performed on specimens preconditioned in this way should probably give a reasonably realistic indication of field performance.

4. The Kennesaw aggregates tested in the main test series had excessive weak particles. Weak aggregate has also been observed in another mix (not from Kennesaw) in the past. The presence of a moderate level of weak aggreate particles might not show up in routine test results, but could have a significant effect on mix fatigue life. Therefore, the failure surface of all tested specimens should be examined and the level of weak aggregate reported. Also, precautions should be taken in the field to minimize this problem.

5. A small field project might be initiated to study (1) presence of weak aggregate, (2) asphalt coating on aggregates, and (3) the presence of uncoated mica in mixes.

6. For evaluating stripping/fatigue resistance of asphalt concrete mixes by the static tensile splitting test, consideration should be given to using stiffness (force divided by deflection) as a means for evaluating potential mix fatigue performance. Stiffness at a stress level below failure would probably correlate best with fatigue performance, as strongly indicated by the static beam bending tests. A cyclic diametral or beam fatigue test would probably be a better indicator of fatigue performance than the static tensile splitting test, but harder to perform.

   More research in both the laboratory and field is recommended for selecting the most appropriate testing procedure suitable for routine evaluation of stripping.

7. The GHD 56 Boiling test should continue to be performed as an aid in evaluating beneficial effects of additives added to reduce the stripping problem.

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APPENDIX A

TEXAS BOIL TEST

and

TEXAS PEDESTAL TEST

TEXAS FREEZE-THAW PEDESTAL TEST

The Texas Freeze-Thaw Pedestal Test (3), while empirical in many respects, has a fundamental basis being designed to maximize the effects of bond and to minimize the effects of the mechanical properties of the mixture, such as gradation, density, and aggregate interlock, by using a uniform aggregate size. Three categories of materials can be used in this test. The category is defined by aggregate type and size:

(1) natural aggregate -- interval between the No. 20 and 35 sieves,

(2) natural aggregate -- interval between the No. 40 and 80 sieves, or

(3) crushed aggregate -- interval between the No. 20 and 35 sieves.

To perform the Texas Freeze-Thaw Pedestal Test the proper amount of aggregate is weighed and mixed with asphalt. After determining the optimum asphalt content from Texas mixture design procedure (10), the initial specimen is prepared at the optimum asphalt content plus 2 percent and the asphalt percent is adjusted depending on the absorption characteristics of the mixture or individual aggregate being tested.

After initial mixing, the mixture is reheated and remixed two more times every hour, and then cooled to room temperature for at least 30 minutes. The mixture is then heated for an additional 20 minutes, placed in a cylindrical mold, and compacted at a constant load of 27.6 kN (6200 lb) for 20 minutes. This reheating and remixing procedure was designed to produce an asphalt with a viscosity similar to that of an aged asphalt after 5 years of field service. Each briquet specimen is cylindrical with a diameter of 41.33 mm (1.627 in) and a height of 19.05 mm (0.750 in). The briquet is extracted from the mold and allowed to cool, the height is measured, and the briquet is cured at ambient temperature for 3 days before being subjected to freeze-thaw cycling. The specimen is then placed on a stress pedestal in a jar covered with 12.7 mm (0.5 in) of distilled water, placed in a temperature-controlled room, and subjected to thermal cycling, which consists of 12 hours at -12°C (10°F) followed by 12 hours at 49°C (120°F). At the end of each cycle the surface of the specimen is inspected to determine if the briquet has failed by cracking. The number of freeze-thaw cycles required to induce cracking in the briquet is the test result which is then used as a measure of water susceptibility.

Most aggregate mixtures consist of materials from several sources that are blended naturally or by the contractor to satisfy a grading requirement. These individual components may vary in size, shape, surface texture, and chemical composition. Thus, it is desirable to first evaluate the mixture and then if stripping is detected it may also be desirable to evaluate individual components. To evaluate the mixture, the individual components should be included in the mixture in proportion to their weight. Another approach would be to include the components in proportion to their surface area since stripping is a surface phenomenon. However, until additional work is conducted relative to the importance of surface area, the current recommendation is that the components be proportioned by weight.

TEXAS BOILING TEST

The Texas Boiling Test is a synthesis of several boiling tests used in several state agencies (4). In the test a visual rating is made of the extent of stripping of asphalt cement from the aggregate after a sample has been subjected to the action of water at elevated temperatures for a specified time. The rating is made after the mixture has been cooled in the beaker and then poured on a clean surface.

To perform the test the asphalt cement is heated at 103°C (325°F) for 24 to 26 hours. The unwashed individual aggregate of 100 or 300 g of mixture is also heated at 103°C (325°F) for 1 to 1-1/2 hours. At the appropriate time, asphalt is added to aggregate and mixed manually on a hot table. The mixture is allowed to cool at room temperature for 2 hours.

A 1,000 ml beaker is filled approximately half full .th distilled water and heated to boiling. The mixture is dumped in the beaker and boiled for 10 minutes. Any floating asphalt cement is skimmed away from the surface of the water. After boiling the beaker is removed from the heat and cooled to room temperature, the water is then poured off and the mixture is emptied onto a paper towel.

The degree of stripping is visually rated by a panel of three graders. Each observation should be matched with a rating performed at the end of the boiling period. The mixture should also be examined on the next day after drying.

When dried out some mixtures show evidence of stripping of the fines that is not apparent when the mixture is still wet.

Most aggregate mixtures consist of materials from several sources that are blended naturally or by the contractor to satisfy a grading requirement. Because these individual components vary in size, shape, surface texture, and chemical composition, the mixture is evaluated using the fraction called for in the job mix formula used for construction of the pavement. In order to have the same asphalt film thickness on the individual aggregate, the standard procedure suggests that the aggregate-asphalt mixture contain the design optimum asphalt content determined according to method Tex-204-F (10) and that the percent asphalt be increased or decreased by 1 percent depending on the characteristics of the individual aggregate.

APPENDIX B

SELECT

GEORGIA DEPARTMENT OF TRANSPORTATION

RESEARCH RESULTS

TENSILE SPLIT RESULTS

Date on Design	Mix Type	Quarry Location	No Lime No Cement 0.5% Liquid			1% Lime 0.5% Liquid			1% Cement 0.5% Liquid			No additive 1% Lime			No additive 1% Cement		
			PSI ÷ PSI		% RTD	PSI ÷ PSI		% RTD	PSI ÷ PSI		% RTD	PSI ÷ PSI		% RTD	PSI ÷ PSI		% RTD
10-29-82	Base	Fla. Rock Rome	73.4	73.1	100%	83.1	80.2	104%	87.5	79.3	110%	86.7	89.8	97%	80.6	80.1	101%
11-1-82	B	Fla. Rock Rome	72.4	88.2	82%	76.3	88.7	86%	75.8	80.4	94%	75.5	85.3	89%	75.7	83.8	90%
8-12-82	H	Stoneman Rossville	39.4	51.9	76%	61.3	75.0	82%	63.6	72.2	88%	37.8	54.7	69%	40.9	54.5	75%
8-17-82	G	Chatsworth	42.8	73.1	59%	52.3	81.2	64%	49.0	60.9	80%	24.5	57.7	42%	46.7	61.5	76%
8-27-82	E	Colwell Ellijay	60.3	74.4	81%	63.6	87.4	73%	65.9	72.1	91%	47.0	78.2	60%	63.0	76.4	82%
8-27-82	Base	Stoneman Rossville	53.0	75.6	70%	67.7	76.1	89%	56.7	70.6	80%	52.9	79.4	67%	51.9	72.4	72%
11-13-82	H, I	Consolidated Douglasville	88.2	83.8	105%	97.6	82.2	119%	94.5	83.1	114%	103.2	88.8	116%	99.9	81.4	123%
11-13-82	G	Fla. Rock Rome	69.7	86.2	81%	75.3	87.4	86%	94.2	75.8	124%	72.4	87.9	82%	84.8	77.3	110%
	H	Chatsworth										71.2	73.6	97	77.5	72.6	107

DEPARTMENT OF TRANSPORTATION
STATE OF GEORGIA

INTERDEPARTMENT CORRESPONDENCE

FILE

OFFICE Materials & Research
Forest Park, Georgia

DATE November 1, 1982

FROM Tom Stapler, P.E., State Materials and Research Engineer

TO Thomas D. Moreland, P.E., Commissioner

SUBJECT MOISTURE DAMAGE IN ASPHALTIC CONCRETE PAVEMENTS THAT UTILIZE DENSE GRADED SURFACE MIXES

Attached is a report prepared by our Bituminous Construction Section pertaining to moisture damage to mixes under dense graded surfaces.

This study is based on approximately 300 cores cut from pavements through Georgia late in 1981 and early 1982.

This comprehensive study points out numerous ideas that are basic to constructing and maintaining our transportation system, many of which we are very familiar with and tend to sometimes overlook.

In addition it brings out somewhat less familiar information; for instance, the relationship between moisture damage determined by Lottman's Tensile Split and by visual inspection. It compares recently constructed mixes to not so recent construction, aggregate sources related to poor performance and others related to good performance, etc.

TS/RC/tjp

Attachment

cc: Hal Rives
John Hassell
Alva Byrom
Alan Childers
Stanley Lord
Tony Dowd
Jim McGee

MOISTURE DAMAGE
IN
ASPHALTIC CONCRETE PAVEMENTS
WHICH UTILIZE DENSE-GRADED SURFACES

BY

GEORGIA DEPARTMENT OF TRANSPORTATION
OFFICE OF MATERIALS AND RESEARCH

OCTOBER, 1982

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LIST OF FIGURES

MOISTURE DAMAGE
IN
ASPHALTIC CONCRETE PAVEMENTS
WHICH UTILIZE DENSE-GRADED SURFACES

General

This study was performed during early 1982 to determine the extent of moisture damage (stripping) to asphalt mixes in Georgia pavements that utilize dense-graded mixes as surfaces. In this study 4" diameter full depth cores were cut from each of the randomly selected state routes in Figure I. Each construction lift in each core was evaluated by the procedures in Appendix A. Basically, this procedure consisted of an initial visual inspection, followed by diametral tensile strength testing, and then a visual inspection of the inside of each core after it was forced apart in tension.

The degree of stripping for mixes in this study is included in "Appendix B". A rating of # 1 indicates slight stripping, # 2 indicates considerable stripping, and # 3 indicates severe stripping. There are very few rated # 3 because this rate generally represents cores that were literally falling apart.

Generally, it is difficult to observe stripping in freshly cut cores because the cut surface always looks better than the inside of the core after it has been forced apart in tension. A moisture damage core separates predominantly at the asphalt/aggregate interface while an undamaged core separates through asphalt film to leave the aggregate visually coated.

Some construction lifts were thick enough to allow inspection and testing of the top and bottom of the same lift. Frequently the lower portion was found to have more moisture damage and was weaker than the upper portion. The lower portion was usually more open visually and likely allowed water to enter and leave more readily. This finding vividly points out the need for positive drainage, good sealing procedures, and closely controlled voids throughout the construction lifts, especially if a lift is thick.

The Office of Materials and Research conducted investigations in 1979 and in 1981 into stripping where open-graded mixes were utilized as surface courses. These studies are on file in the Office of Materials and Research.

Recent Changes in Asphalt Mixes

In recent years Georgia has encountered sufficient instances of severe stripping to realize a need for improved procedures. This present study verifies and better documents the problem. Numerous precautions have been implemented to improve resistance to moisture damage in Georgia mixes.

1) During 1979 Georgia adopted a method for testing retained stability (GHD - 66 in Appendix A) based on work by Dr. Robert P. Lottman (1, 2, and 3) with diametral tensile splitting and moisture damage to Asphaltic Concrete mixes. This method of test has proven more pertinent than Georgia's Immersion Compression Retained Stability Test Method (GHD - 53).

2) During late 1979 Georgia began requiring 0.5% anti-stripping additive in all dense-graded asphalt concrete mixes and 1.0% in open graded mixes in an effort to eliminate the occurrence of severe stripping. Previously, additive had been used only with aggregate sources known to be susceptible to stripping, and then only the amount of additive necessary to pass the Boil Test (GHD - 56) was used.

3) During 1981 the amount of minus # 200 in Georgia's Specifications was reduced to insure a thicker film of higher grade binder. Excessive dust had "contaminated" the asphalt cement and reduced its ability to coat and adhere to the aggregate, especially if the aggregate was dusty or damp. Reducing the minus # 200 should help the stripping problem.

4) During 1981 the use of washed gradations on all mixes and mix ingredients was implemented to insure adherence to the reduced minus 200 specifications.

5) The use of hydrated lime has recently been adopted to increase the retained stability of mixes which increases resistance to moisture damage. Lime should also provide additional benefits such as reducing the rate of asphalt cement oxidation, pushing, shoving and rutting.

6) A transition from viscosity grade AC-20 to AC-30 has been adopted to toughen the film without sacrificing a significant increase in laydown temperature. Resistance to rutting, a greater retention of the asphalt film on the aggregate, and use of a little more AC is expected. These changes should result in an increased resistance to moisture damage.

7) Adoption of the use of maximum specific gravity (AASHTO: T-209) of mixtures, and effective specific gravity of aggregate provides a more appropriate theoretical by allowing for the intrusion of varying fractions of the asphalt cement into the aggregate. This provides greater precision for controlling compaction and air voids. In addition more emphasis is being placed on the void content and void variability on pavements being constructed.

8) Recent reduction of the amount of sand allowed in mixes reduces surface area, and in turn, increases film thickness. It also generally increases the amount of AC that is practical to use. The increase in asphalt cement should also increase film thickness to improve resistance to moisture damage. In addition, mix tenderness and a reduction in further immediate consolidation under traffic should be realized from this change.

Discussion of Study

Moisture damage could be seen on the drill cut surface of the cores in Figure II prior to testing. Drill cut surfaces almost always look good and seldom indicate severe stripping, therefore, the materials in Figure II should be considered highly susceptible to stripping, especially those near the top of the page.

Figure III was prepared to provide a general overview of stripping for each mix type in pavements of various ages. Many observations can be drawn from the data in Figure III.

1) Construction lifts placed prior to 1965 almost always had some stripping. Lifts placed after 1965 have gradually increased in resistance to moisture damage as can be seen in the following table which is derived from Figure III.

Time Frame	Total # of Tests	Numbers of Tests Per Rate				% of Tests Per Rate			
		0	1	2	3	0	1	2	3
Before 1965	21	1	9	11	-	4	43	53	-
1965 to 1973	23	9	11	3	-	39	48	13	-
1974 to 1977	75	51	13	11	-	68	17	15	-
1978 to 1981	82	43	26	16	2	52	32	20	2

2) "E" mixes placed after 1974 and base mixes placed from 1974 to 1977 have very good stripping rates. There were 86 tests total; of which 50 were rated "0", 24 were rated # 1, 11 were rated # 2 and none rated # 3; which is 58%, 28%, 13% and 0%, respectively. Strengths of these mixes were superior with most falling above 100 PSI with some above 200 PSI.

3) "G" and "H" mixes have strength generally below 100 PSI. Base mixes placed since 1978 had a relatively high frequency of low strengths and # 2 and # 3 rates. Also G and H mixes placed prior to 1965 were poor.

4) Generally, the lowest strength in Figure III for each time period improved from before 1965 through 1973 and then gradually dropped through 1981. These values are about 35 PSI, 70 PSI, 45 PSI, 30 PSI, respectively, beginning with the right side of the page and moving to the left. This trend can also be seen in the Table in # 1 above.

5) There is a very strong correlation between visual stripping of the inside surface of cores broken in tension and the diametral tensile strength in PSI (Review Figure III).

Figure IV was prepared from the data in Figure III, primarily to point out that since 1974 Georgia has placed its best moisture resistant mixes, its strongest mixes; and during this same period it has placed some of its worst moisture resisting and lowest strength mixes. In an effort to better understand this, Figure V and VI were prepared. Figure V contains the "Excellent Group" and Figure VI contains the "Poor Group". According to the two figures, most of both groups are on roadways in District # 4, however a few were from other districts. Some comments on these two Figures are given below:

1) In both groups lower portions of construction lifts are weaker than top portions.

2) The "Excellent Group" contains aggregates from Stockbridge, Warrenton, Barin, Auburn Alabama, Arlington, and Palmer Station.

3) The "Poor Group" contains aggregate from Elberton, Ruby, Mountain View, Kennesaw, Camak, Fortson, and Palmer Station.

4) For comparison with # 2 and # 3 above, note that aggregates in Figure II are from Dan, Postell, Ruby, Camak, Palmer Station, Kennesaw, Mountain View, and Yatesville, It is interesting to note that five of the aggregates in Figure II are from sources in the Poor Group and only one in the Excellent Group. Palmer Station is in all three groups.

5) Other similarities and differences may be found if an in-depth comparison were made of pavements in the above three groups.

Figure VII relates to mixes containing limestone aggregates in District # 6. These mixes are generally weak by Diametral Tensile Split Strength (PSI), probably because they contain excessive dust and are deficient in the sizes that act as abrasives (generally minus # 16 + # 50) to prevent movement of larger particles in the mixes. The dust and asphalt cement act together to approach saturation of the mixture when too little asphalt is present. This produces a poor quality binder less capable of resisting moisture damage and rutting. Marshall flow values are generally high in these mixes and the mixes are susceptible to both stripping and rutting.

Observation Through Stereomicroscope

Cores from a few of the pavements were observed through a stereoscopic microscope with varying powers from about 8 to 80. There were numerous observations worthy of note and attempts were made to photograph some of them. However, the photographs and slides did not show the necessary detail since they were not stereoscopic. In the absence of photographs, some of the observations are listed below:

1) By visual inspection some cores appeared to be grey in color; but through the microscope, the asphalt had a very shiny black appearance, which may have been influenced by the heat from the microscope's light. The grey appearance came from the very fine, uncoated particles adjacent to the asphalt. (One specific sample came from a "B modified" mix under an open-graded mix).

2) In some cores, a rock could be removed and a dull "mat" appearance would remain as if the rock had been coated with dust which prevented proper contact of asphalt and aggregate. Observing the removed rock, small specks of well attached asphalt could be seen on an otherwise relatively clean aggregate surface.

3) Observing the aggregate in some cores, the surface appeared to be fractured, or more nearly "shattered". Some fraction of the asphalt had penetrated at times as far as 1/4 inches into the aggregate surface. On the same core, rocks observed on the cut surface had a defininte color distinction between the penetrated and non-penetrated area. The penetrated area was coffee stain in color. The shattered appearance may be related to rollers, temperature when rolled, lift thickness, brittle aggregate, etc.

4) In some cores the fine particles appeared to have very sharp "spike-like" points, and generally were clear to milky in color. This material did not appear to have firmly attached asphalt

cement; as if there may have been incompatibility involved.

5) In some cases the AC and dust was pretty well mixed to produce a poor quality binder. A knife blade was used to move the binder about. The binder had a dirty, greasy, dull, and weak appearance.

6) It was also noted that sand asphalt mixes or mixes containing sand generally had a strong looking bond of asphalt to the sand. This bond was probably better in general than for most quarry aggregate, whether coarse or fine. Although all sand asphalt mixes in Appendix B were very old, they generally were very strong.

Recommendations

Many measures that should be mentioned to improve the resistance of asphalt mixes to moisture damage have been implemented, as previously mentioned in the Section on Recent Changes in Asphalt Mixes. However, there are some other steps that should be considered and are hereby recommended.

1) Make every effort to insure positive drainage and good sealing of Georgia's pavements to prevent intrusion by moisture. This includes limiting the water during construction of bases, subbases, subgrades and embankments to, or near, optimum. An asphalt mix should not be placed on wet (above optimum) material. In existing pavements careful attention should be placed on how water can be moved away from the pavement before it has time to enter. There exists places where water flows or stands along the edge of the pavement rather than being caused to flow into a ditch. The flowline of ditches should be much lower than the subgrade or embankment surface to insure that the pavement remains drained; in super-elevation the flowline should be below the lower edge of the subgrade or embankment surface.

2) Perform an in-depth comparison of the Groups in Figures V, VI, and VII to search for other causes of stripping and to find ways to more frequently encounter the quality in the "Excellent Group" of Figure V.

3) The stereomicroscope should be used to randomly monitor for ways to improve pavements. This tool should be useful for comparing the effects of different compaction equipment on aggregates and pavements. Such aspects as amplitude, frequency, weight, tire air pressure, etc. should be studied.

4) The stereomicroscope should be used to randomly monitor current and future design mixes, roadway cores, plant mixes, and aggregates for excessive dust coating on aggregates, excessive dust, shattered aggregate surfaces, etc. This device should be beneficial in many types of problem solving.

5) To insure that asphalt cement contents are based on correct compaction and void data, an aggressive comparison program should be used to continuously monitor nuclear gauges by core results.

6) An investigation should be made to determine if surface G an H mixes are being placed too thin for the condition of the pavement. At times high points on the old pavement may prevent proper densification of new pavement.

7) Efforts should be made to determine why base mixes placed on roads since 1978 are frequently very weak and visually stripped.

8) To insure use of correct compaction and void data in decision making, maximum specific gravity should be determined using mix produced through the plant rather than values determined months earlier based on laboratory mixed material shown on designs. Maximum gravity tests should be determined at the start of construction and as needed thereafter, especially where the amount of asphalt, source of asphalt or aggregate blend is changed. Where the amount of asphalt has been changed, and data is needed immediately, the effective gravity of aggregate in the mix may be used along with the current amount of asphalt to calculate a new theoretical. When this is used, it should soon be followed up with a maximum gravity test for verification.

9) As much asphalt cement should be put into mixes as practical for the roadway involved; however, with the reduction of dust caution should be exercised to avoid causing a skid problem. Where a mix will not receive as much asphalt cement as it should, an effort should be made to determine why. Such effort may include observations through the stereomicroscope, additional maximum gravity testing, nuclear gauge correction factor check with conventional methods, etc.

10) When investigating a pavement for stripping, the procedure in Appenix A is recommended. Observing cut surfaces of cores is not considered reliable.

11) Further study of recommendation numbers 2, 6, and 7 may best be performed through a research project. Also in such a project, the stereomicroscope should be used to evaluate types of compaction equipment for use on different aggregate types to determine if the mixes are being harmed in certain cases.

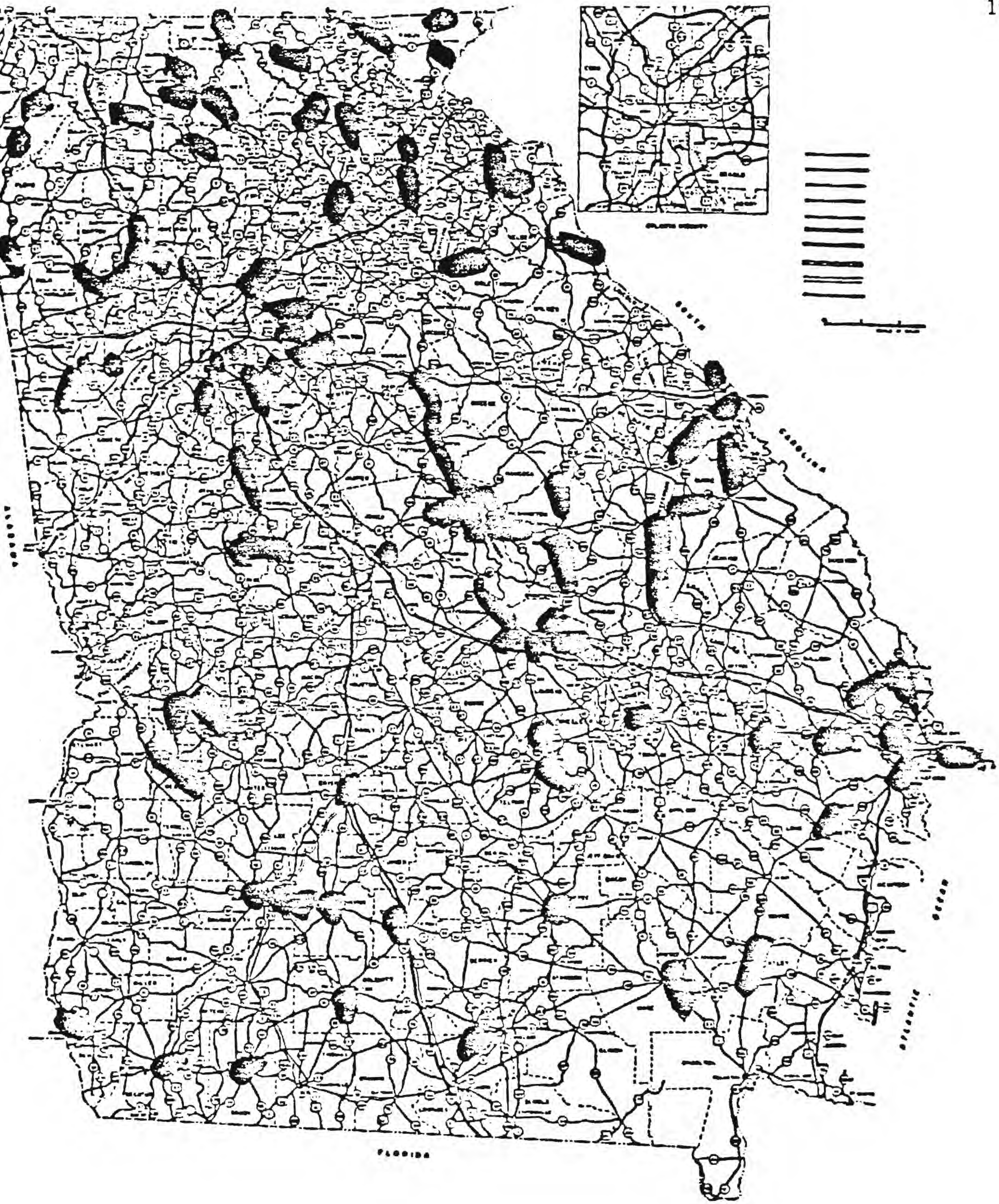

Figure I

Randomly selected state routes from which full depth cores were cut are designated on this map.

PRESENT MOISTURE DAMAGE

(Severity & Frequency of Stripping Noted)

AGGREGATE SOURCE	STRIPPING RATING		
	1 (slight)	2 (considerable)	3 (severe)
Martin-Marietta of Dan	6 Cores	2 Cores	2 Cores
Southern Aggregate at Postell	1 Core	1 Core	1 Core
Martin-Marietta at Ruby	10 Cores	4 Cores	---
Martin-Marietta at Camak	6 Cores	1 Core	---
Florida Rock at Palmer Station	3 Cores	---	---
Martin-Marietta at Dan/Camak	1 Core	---	---
Vulcan Materials at Kennesaw	1 Core	---	---
Vulcan Materials at Mt. View	1 Core	---	---
Yatesville	1 Core	---	---
	(maybe no stripping)	(pitted surface)	Core break with hand or can't get core out

Figure II

Moisture damage could be seen on the drill cut surfaces of the above cores prior to testing. The drill cut surface almost always looks good and seldom indicates stripping; therefore, these materials should be considered highly susceptible to stripping, especially those near the top of this page.

Age - Mix Type - Stripping

Tensile Strength & Visual Rating

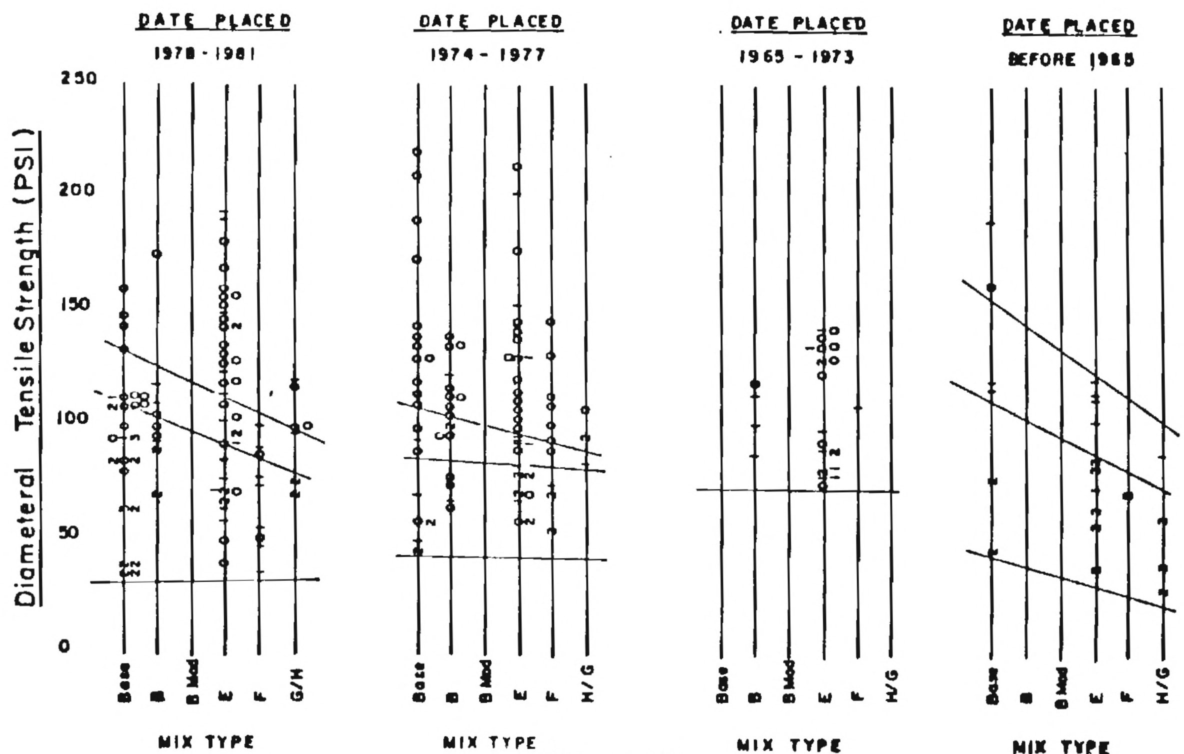

Figure III.

This figure was prepared to provide a general overview of stripping for each mix type in pavements of various ages. The stripping rate (0, 1, 2 and 3) is plotted for each mix type in each time period; and, the tensile strength is on the scale at the left of this page for comparison. Observations drawn from this figure are on Page 4.

Strength Change With Pavement Age

(This Sketch is Based on Figure III)

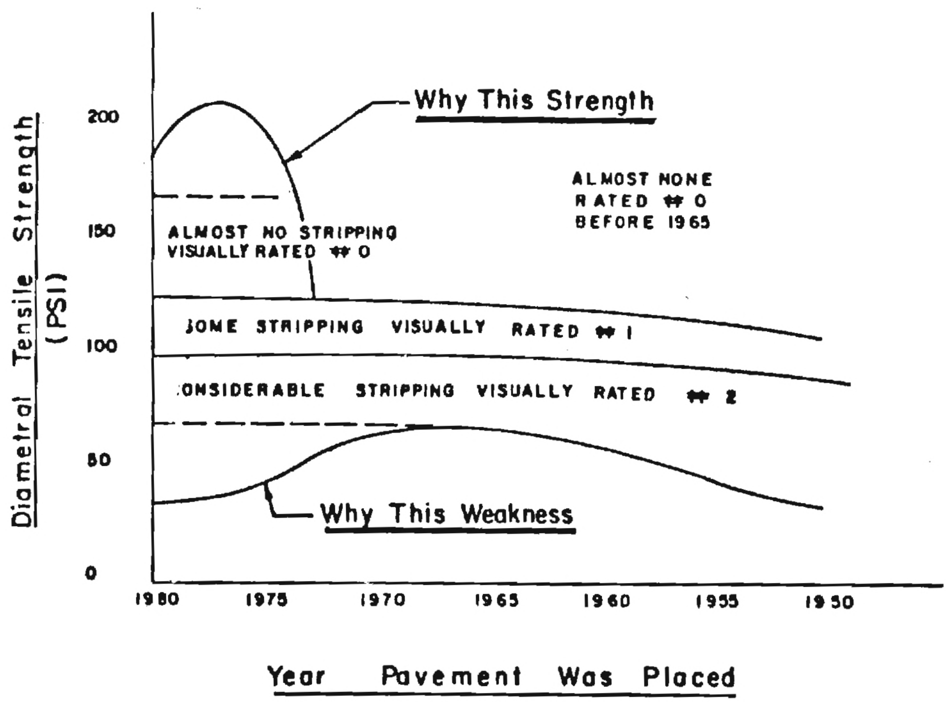

Figure IV

Since 1974 Georgia has placed its best moisture resisting and strongest mixes; and, during the same period it has placed some of its worst moisture resisting and weakest mixes. In an effort to better understand this, Figures V and VI were prepared.

Excellent Group of Pavements Placed Since 1974

Strong and Visually Almost No Stripping

Core #	Source	Sand	Route	PSI	Mix Type	Thickness (in.)	County
7 - 30	Stockbridge	-	SR 42	182	E	1 3/4"	Henry
5 - 3	Warrenton	x	SR 21	191	E	1 1/2"	Effingham
5 - 34	?	- ?	US 1 (SR 4)	217	E	1 1/8"	Ware
4 - 14	Barin	x	US 82	177	E	2 1/2"	Dougherty
4 - 17	Barin	x	"	204	Base	5" (Top)	"
"	Barin	x	"	224	"	5" (Bottom,	"
4 - 18	Barin	x	"	192	"	4 1/2" (Top)	"
"	Barin	x	"	137	Base	4 1/2" (Bottom)	"
4 - 22	Barin/Stockbridge	x	Albany By Pass	106	"	5 1/4" (Top)	"
"	"	x	"	173	"	5 1/4" (Bottom)	"
4 - 31	Auburn AL/Arlington	x	US 84	204	E	1 1/2"	Seminole
4 - 39	Barin/Limerock	-	"	191	Base	2 1/2"	Decatur
4 - 45	Palmer Station	x	319 By Pass	175	B	2 1/2	Colquitt

Figure V

The above mixes placed since 1974 represent the high strength mixes in Figures III and IV that were not stripped. Although these are performing very well it is noted that the bottom of the lifts are weaker than the top for some of the thicker mixes.

POOR PERFORMANCE GROUP OF PAVEMENTS
Placed Since 1970
(Rated # 2 Visually and Weaker Than 70 PSI)

Core #	Source	Sand	Route	PSI	Type	Thickness (in.)	County
5 - 11	Elberton	-	US 80 (SR-26)	72	F	1 1/4	Hart
1 - 16	Elberton	-	SR 72	55	F	1 1/8	Elbert
2 - 42	Ruby	x	SR 15	59	E	1 1/8	Washington
7 - 4	Mtn. View	-	SR 85	71	E	1 7/8	Clayton/Fay.
7 - 35	Kennesaw	-	US 41	71	E	1 1/4	Cobb
1 - 13	Camak	-	SR 8 (US-29)	72	H	1 1/4	Effingham
4 - 8	Ruby	x	US 82	37	A	2 1/2	Worth
"	Ruby	x	"	45	A	5 1/4 (Top)	"
4 - 8	Ruby	x	"	33	A	5 1/4 (Bottom)	"
4 - 9	Ruby	x	US 82	70	E	1 1/2	"
4 - 12	Ruby	x	US 82	41	Base	5" (Top)	"
4 - 12	Ruby	x	"	31	Base	5" (Bottom)	"
4 - 23	Columbus (Fortson)	x	US 280	46	Base	5" (Top)	Crisp
4 - 23	"	x	"	61	Base	5" (Bottom)	"
- 48	Palmer Station	x	SR 125	92	Base	4 1/2" (Top)	Lowndes
4 - 48	"	x	"	69	Base	4 1/2 (Bottom)	"
4 - 50	Ruby/Camak	-	US 441	71	B	2 5/8"	Coffee
4 - 51	"	-	"	116	Base	4 1/8" (Top)	Coffee
"	"	-	"	110	Base	4 1/8" (Middle)	"
"	"	-	"	65	Base	4 1/8 (Bottom)	"
4 - 52	"	-	"	67	Base	4 5/8"	"

Figure VI

The above mixes placed since 1974 represent the low strength mixes in Figures III and IV that were stripped. The lower portions of the thicker mixes were also weaker than the top. Many of these aggregates also were in Figure II.

PAVEMENTS CONTAINING LIMESTONE

FROM DISTRICT # 6

Source	Route	County	Core #	Mix Type	Strength (PSI)	Visual Rate
Rome	US - 41	Bartow	6 - 1	B Mod.	43	0
"	"	"	6 - 1	F	124	0
"	"	"	6 - 1	Base	95	0
"	"	"	6 - 2	B Mod.	38	0
"	"	"	6 - 2	F	94	0
"	"	"	6 - 2	Base	52	0
"	"	"	6 - 3	B Mod.	58	0
"	"	"	6 - 3	F	49	0
"	"	"	6 - .3	Base	62	0
Rossville	Old 27	Chatooga	6 - 13	F	54	0
"	SR - 151	Walker	6 - 15	E	54	0
"	"	"	6 - 15	E	35	1
"	"	"	6 - 16	E	59	0
Dalton	SR - 136	Gordon	6 - 17	E	53	0

Figure VII

Construction lifts containing limestone from District # 6 are generally much weaker than mixes from throughout Georgia even though the visual stripping of these cores were very good. Previous inspections of pavements containing this limestone have reflected severe stripping in mixes on high traffic volume roads. The limestone aggregates in District # 6 have excessive dust and are deficient in the sizes between the # 16 and # 50 sieves. Based on limited laboratory work removal of the excessive dust and using 1% hydrated lime frequently increases the retained tensile strength (PSI) significantly.

REFERENCES

1. Lottman, R.P., "Predicting Moisture-Induced Damage to Asphaltic Concrete," NCHRP Report 192 (1978).

2. Lottman, R.P., "Predicting Moisture-Induced Damage to Asphaltic Concrete - Field Evaluation Phase," Interim Report, NCHRP Project 4-8 (3)/1 (1978).

3. Lottman, R.P., "Predicting Moisture-Induced Damage to Asphaltic Concrete, Progress Report on Field Evaluation Phase of NCHRP Project 4-8(3)/1, Proc, 66th AASHTO Annual Meeting, Las Vegas, NV (1980), pp. 149-169.

4. Fields, F., Evaluation of Anti-Stripping Additives for Asphaltic Concrete Mixes Effectiveness and Recommended Uses. Presented at CTAA Annual Conference, Halifax, Canada, Nov. 21 to 23, 1977.

5. GHD - 66 (Georgia Highway Department Test Method), Method of Test for Determining Retained Stability of Marshall Specimens and Roadway Cores by Diametral Tensile Splitting. (See Appendix A)

6. GHD - 53 (Georgia Highway Department Test Method), Method of Test for Measurement of Reduction in Marshall Stability of Bituminous Pavements Caused by Immersion in Water

Appendix A

This appendix contains the tentative "Georgia Procedure For the Evaluation of Stripping in Asphaltic Concrete Mixes". This procedure is for evaluating moisture susceptibility (stripping) of Asphaltic Concrete mixtures that have been on the roadway at least one winter. The visual inspection rating for stripping is derived from a method used in Canada (4). The tensile strength (vacuum saturated but no freeze thaw cycle) is derived from work performed by Dr. Robert P. Lottman (1, 2, and 3).

Georgia's method GHD-66 "Method of Test for Determining Retained Stability of Marshall Specimens and Roadway Cores by Diametral Tensile Splitting" is enclosed because the above tentative procedure refers to GHD - 66.

TENTATIVE PROCEDURE FOR THE EVALUATION OF STRIPPING IN ASPHALTIC CONCRETE MIXES

The following procedure shall be used for evaluating stripping of asphalt cement from aggregate particles in asphaltic concrete pavements.

I. Determine the location to be sampled.

A) Record exact location of sample - milepost, station number, lane, distance from edge of pavement, and wheelpath.

B) Observe and record the general condition of the roadway around the core sample location - cut a fill area, pavement failures, etc.

II. Obtain core samples.

A) Cut one 4" diameter roadway core per location.

B) The core edges must be 90^{o} to be tested.

C) Visually inspect drill-cut surface of core for stripping.

D) Immediately seal in a plastic bag and transport to the lab.

E) Measure and record total thickness and each lift thickness.

F) Vacuum saturate each core by GHD - 66.

G) Bring each core to 55^{o}F and break at .065" per minute. Record the height, flattened width, and load.

H) Open the core sample by tension and rate the stripping in accordance with paragraph III.

I) Calculate the stability of each lift of each core in PSI for each location and record.

III. Calculate the stripping rating 'S'.

Calculate the stripping rating 'S' using the equation: $S = \frac{C + F}{2}$, by assigning values to C, and F as shown in the following tables. A # 3 rate is for mixes that are literally falling apart or can be easily crumbled by hand.

Values of C

C	Coarse Aggregate Stripping
1	Less than 10%
2	10% to 40%
3	greater than 40%

Values of F

F	Fine Aggregate Stripping
1	less than 10%
2	10% to 25%
3	greater than 25%

NOTE: The samples must be tested and rated within two days after they are removed from the pavement, since storage may produce increased strength and recoating of the aggregate particles.

IV. Using the calculated stripping rating 'S', evaluate and record the stripping of the core as follows:

'S'	Stripping Evaluation
0	none
1	slight
2	considerable
3	severe